3.5 Seiche

A seiche is an oscillation of the water surface in an enclosed or semi-enclosed body of water initiated by an external cause. Once started, the oscillation may continue for several cycles; however, over time it gradually decays because of friction (NRC, 2011, Section 3.6).

There are two water bodies, The Hudson River and the IPEC Discharge Canal, in the vicinity of IPEC that require evaluation for Probable Maximum Seiche (PMS), as shown in Figure 3.5-1.

3.5.1 Method

The hierarchical hazard assessment (HAA) approach described in NUREG/CR-7046 (NRC, 2011) was used for the evaluation of the effects of seiches on the maximum water surface elevation at IPEC. The HHA approach is a progressively refined, stepwise estimation of site-specific flood hazards, starting with the most conservative simplifying assumptions that maximize flood hazards. If the site is not inundated by the flood mechanism evaluated, a conclusion that the SSCs are not susceptible to flooding is valid and no further analyses were completed (NRC, 2011).

The HHA approach is consistent with the following standards and guidance documents:

1. NRC Standard Review Plan, NUREG-0800, revised March 2007;
2. NRC Office of Standards Development, Regulatory Guides:
    a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976;
    b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977; and
3. American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8 - 1992)

Proximity of IPEC to a large river estuary, the Hudson River, requires evaluation of the PMS for a semi-enclosed basin. This evaluation addresses the PMS at the site with consideration of meteorological, astronomical, and seismic forcing as the causative mechanism for low frequency water surface oscillations or seiche in a semi-enclosed basin, the Hudson River.

The discharge canal at IPEC also requires evaluation of the PMS for both a semi-enclosed basin (in the longitudinal direction) and an enclosed basin (in the transverse direction). This evaluation addresses the PMS at the site with consideration of meteorological, astronomical, and seismic forcing as the causative mechanism for low frequency water surface oscillations or seiche in both a semi-enclosed and enclosed basins, in the discharge canal at IPEC.

With respect to seiche evaluations for both of the water bodies of interest near IPEC, the following steps were used:

1. Estimate the natural period of oscillation (primary seiche mode) of the surface water bodies using Merian's Formula;
2. Analyse measured water level data within the Hudson River to identify the natural periods of the Hudson River, and compare to the periods developed for the river using Merian's Formula;

3. Compare the natural period of the surface water bodies to the periods of potential forcing mechanisms, including meteorological, astronomical and seismic conditions, to determine the potential for resonance; and
4. Evaluate flood levels where resonance could occur.

3.5.1.1 Natural Period of the Surface Water Bodies

As noted above, the natural periods of oscillation of the surface water bodies were determined by calculation using Merian's formula and evaluation of water level data. The dimensions of the water bodies were developed using ArcMap 10.0 in a UTM Zone 18 Projection and IPEC drawings.

Merian's Formula provides a method for estimating the natural periods for the seiche modes in enclosed and semi-enclosed basins (Scheffner, 2008). The Hudson River and the discharge canal (in the longitudinal direction) are semi-enclosed basins and the primary seiche modes are defined by Equation 1:

$$T = \frac{4l}{(1+2n)\sqrt{gh}} \qquad \text{(Equation 1)}$$

where:

T is the period

l is the length of the basin

g is the acceleration due to gravity

h is the average depth of the basin.

$\sqrt{gh}$ is the shallow water wave speed

n = the number of nodes along the axis of the basin (0 for the primary mode of a semi-enclosed basin), n=0,1,2…

In the transverse direction the discharge canal is considered an enclosed basin since it is bounded by a steel sheet pile bulkhead on the west side and a riprap slope on the east side (See Figure 3.5-12). Merian's Formula for an enclosed basin is presented in Equation 2, below (Sheffner, 2008):

$$T = \frac{2l}{n\sqrt{gh}} \qquad \text{(Equation 2)}$$

where:

T is the period

l is the length of the basin

g is the acceleration due to gravity

h is the average depth of the basin.

$\sqrt{gh}$ is the shallow water wave speed

n = the number of nodes along the axis of the basin, 1 for the primary mode in an enclosed basin

A spectral analysis of water level data was also performed to compare the estimated period of the primary seiche mode with the observed periodicity of the Hudson River using six-minute water level

data at USGS water level stations on the Hudson River, and one NOAA water level station near the Hudson River (See Figures 3.5-5 and 3.5-6) . The software MATLAB R2011b was used to perform the Fast Fourier Transform on each water level station data set.

3.5.1.2 Periods of External Forcing Mechanisms

Applicable forcing mechanisms include meteorological, astronomical, and seismic events. The fundamental periods of the forcing mechanisms have to match the natural frequency of the surface water bodies (resonance) for significant seiche amplification to occur. The periods of the external forcing mechanisms were developed based on typical, published ranges for these mechanisms.

3.5.2 Results (AREVA, 2013)

3.5.2.1 Natural Period in the Hudson River using Merian's Formula

Longitudinal Direction

The Hudson River was evaluated as a semi-enclosed basin in the longitudinal direction. The resonant modes of a semi-enclosed basin are shown in Figure 3.5-4. The period of the primary mode is determined by setting n=0 in Merian's Equation 1. In a riverine environment, the location of the anti-node is at the head of the basin, defined as a physical barrier or a rapidly diminishing river cross section that impedes upstream flow and reflects the wave energy. In the case of IPEC, the Federal Dam at Tory acts as an anti-node for seiches. The mouth of the river basin acts as a node.

The Hudson River measures approximately 820,850 feet from the mouth at the Battery to the Federal Dam and has an average depth of approximately 43 feet. Figures 3.5-2 and 3.5-3 show a plan view of depth in meters below NAVD88 along the length of the river. All measurements were performed using ArcMap 10.0 in a UTM Zone 18 Projection. Merian's formula provides an estimated period for the primary mode of approximately 24.6 hours, as provided in Table 3.5-1.

Transverse Direction

The Hudson River was evaluated as an enclosed basin in the transverse direction since both ends of the basin in this direction are closed. The primary mode is determined by Equation 2 with n=1. The length of the basin (i.e., width of the river) is approximately 5,000 feet. The primary period was calculated using two depths of 30 and 50 feet based on the range on measured water depth. The estimated period for the primary mode in Hudson River in the transverse direction is 4 to 6 minutes, as provided in Table 3.5-1.

3.5.2.2 Observed Natural Period in the Hudson River

The natural periods of the river were also evaluated by performing a spectral analysis of measured water level data. The locations of the four USGS river stations and the NOAA Sandy Hook station are shown in Figure 3.5-6. The spectral analysis of the fifteen minute water level data for the four USGS water level stations was carried out on eight to eighteen years of data from the station, and on ten years of data from the 6 minute NOAA station using the software Matlab™ (Release 2011b). The analysis was performed by applying a discreet Fast Fourier Transform (FFT) to the water level data.

The results of the spectral analysis are presented in Figures 3.5-7 through 3.5-11. The semi-diurnal tides form the largest peaks in the power spectra at all five observation locations, followed by the

diurnal tides, which have a relative power about one order of magnitude lower than the semi-diurnal tides for the river water level stations. The tidal harmonic constituents for the Battery, Haverstraw Bay and Albany are shown in Table 3.5-3.

Tidal forcing from the principle astronomical components causes these strong spectral peaks near 1 and 2 cycles per day and match well with the natural period estimated using Merian's Formula. In shallow waters such as the Hudson River the observed tidal spectrum has additional harmonic peaks, known as overtides, caused by transfer of energy to higher frequencies due to non-linear interactions between the tides and the local bathymetry. The observed peaks at frequencies of three cycles per day and higher are all known constituents which occur to some degree in all shallow tidal estuaries.

The spectral analysis of the observed water level shows no direct evidence of a seiche. All of the spectral peaks in Figures 3.5-7 through 3.5-11 are known tidal constituents observed in most coastal and estuarine environments. Further, the power in the peaks for the principle components show no sign of amplification toward the head of the river. Based on the observation data, there is no sign of seiche dynamics in these time series for the Hudson River system.

3.5.2.3 External Forcing Mechanisms in the Hudson River

The periods of external forcing mechanisms were evaluated to determine if resonance with the periods of Hudson River is likely. Possible external forcing mechanisms include astronomical tides, earthquakes, and meteorological conditions including wind gusts and storms.

Earthquakes

The typical frequency content of earthquakes falls outside this range (approximately 24 hours as calculated in section 3.5.2.1). The typical range of ground motion periods from earthquakes do not normally exceed ten seconds.

Meteorological

Meteorological forcing does not have sufficient energy at this frequency to drive a seiche in the Hudson River. Local convection drives wind gusts with a period of about one minute, and diurnal heating and cooling also drive weak periodic motions. In temperate regions, the synoptic scale is the spatial and temporal scale or temperate weather systems, which in the U.S. is about three to seven days (Wells, 1997). The synoptic variability is too long to force a seiche in the Hudson River.

Astronomical

The tides in the Hudson River estuary drive a surface height oscillation of about 3.3 feet near IPEC. The period of diurnal tides fall within the range of seiche periods calculated in the Hudson River and are a potential source of resonance. Overall the diurnal and semi-diurnal tidal constituents diminish from The Sandy Hook to Haverstraw Bay, see Table 3.5-4 (NOAAa-d, 2005) and Figure 3.5-5 for these locations. The minor increases in the tidal constituent amplitudes is not evidence to support a conclusion that there is a resonant seiche due to astronomical tides in the Hudson River. The absence of a resonant seiche is likely due in part to frictional dissipation of the river system.

3.5.2.4 Natural Periods of the Discharge Canal

The canal length was determined by measuring the canal length on the site topographic plan (Sanborn, 2013). The channel depth and width are based on the typical section presented in Figure 3.5-12 (IPEC, 1970). The discharge channel length measures approximately 530 feet long, 72 feet wide and is 20

feet deep relative to MSL. The transverse calculation for the discharge is based on an average width of 62 feet.

Based on the channel dimensions in the longitudinal direction, the period of the primary mode of the discharge canal is 84 seconds. The primary mode in the transverse direction is approximately 5 seconds. An overview of the channel dimensions and associated periods is provided in Table 3.5-2.

3.5.2.5 External Forcing Mechanisms in the Intake and Discharge Canals

Earthquakes

The frequency content of earthquakes can vary by earthquake and by distance of the site from the earthquake epicenter, however it is typically less than ten seconds. The natural period of the canal in the transverse direction falls within the range possible for earthquakes; therefore, resonance in this direction is possible and would result in "sloshing" of the canal water in the transverse direction. Due to the configuration of the discharge canal, see Figure 3.5-12, the height of the seiche in the transverse direction is limited by the elevation of the outboard steel sheet pile bulkhead which is approximately 8.0 feet MSL. Therefore, if a seiche were to occur in the transverse direction, it would be limited to the height of the bulkhead and would not overtop the landward bank of the canal at elevation 15 feet MSL, see Figure 3.5-12.

Meteorological

Meteorological forcing such as wind gusts, typically have a period of approximately 1 minute (Wells, 1997), and therefore do not have a period in alignment with period of the primary seiche mode in the transverse direction (about 5 seconds). The period of primary mode on the longitudinal direction (84 seconds) is close to that of wind gusts. However, wind gusts are not expected to create an initial significant wind set-up or consistent oscillating source of resonant forcing of sufficient strength on the spatial and temporal scales of the discharge canal to cause a significant seiche. A single storm event might cause a large storm surge; however, for resonance to occur, for either reflection point, storm forcing would have to be periodic at approximately once per day.

Astronomical

The astronomical tides in the Hudson River have periods that are several orders of magnitude large than the longitudinal period of the discharge canal and do not cause resonance.

Waves

Although wind-generated waves could occur with periods in the range of the transverse discharge canal period, the geometry of the discharge canal does not allow waves to enter the discharge canal in the transverse direction. The primary mode of the seiche in the longitudinal direction is 84 seconds, and is outside the range of wind-generated wave periods.

3.5.3 Conclusions

The following summarizes the results and conclusions:

1. Two water bodies were identified as being susceptible to seiches and potentially creating a flood hazard at IPEC:
    a. The Hudson River;
    b. The discharge canal at IPEC
2. Seiche motion is strongly damped in the Hudson River and is therefore a seiche is not a threat to the Indian Point Energy Center;

3. Merian's formula, and statistical analysis indicate that the Hudson River has a primary seiche mode with a period of approximately 24 hours in the longitudinal direction and 4 to 6 minutes in the transverse direction;
4. Merian's formula indicates that the IPEC discharge canal has a primary seiche mode with a period of approximately 84 seconds in the longitudinal direction and 5 seconds in the transverse direction;

No further analysis or modeling is required due to the direct observational evidence that resonant modes are damped in the Hudson River and that potential seiches in the discharge channel will not impact SCCs important to safety.

3.5.4 References

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IPEC, 1970. "Indian Point Sheet Piling for Extension of Discharge Canal", Drawing No. BI82027-0, Consolidated Edison, Co., June, 1970, See AREVA Document No. 38-9193643-000.

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Table 3.5-1: Seiche Parameters for the Hudson River

Direction	Length (feet)	Depth (feet)	Period
Longitudinal	820,850	43	24.6 hours
Transverse	5000	30	5.2 minutes
Transverse	5000	50	4.2 minutes

Table 3.5-2: Seiche Parameters for the IPEC Discharge Canal

Discharge Canal	Length (feet)	Depth (feet)	Period (Seconds)
Longitudinal	530	20	84
Transverse	62	20	5

Table 3.5-3: Tidal constituent at three sites on the Hudson River from NOAA

				Amplitude (Feet)			
Name	**Type**	**Period (hours)**	**Frequency (1/days)**	**Sandy Hook**	**The Battery**	**Haverstraw Bay**	**Albany**
M2	Semi-diurnal	12.4	0.5	2.3	2.2	1.5	2.3
N2	Semi-diurnal	12.7	0.5	0.5	0.5	0.3	0.4
S2	Semi-diurnal	12	0.5	0.4	0.4	0.3	0.3
K1	Diurnal	23.9	1.0	0.3	0.3	0.3	0.4
O1	Diurnal	25.8	1.1	0.2	0.2	0.2	0.2
K2	Semi-diurnal	11.9	0.5	0.1	0.1	0.1	0.1
P1	Diurnal	24.1	1.0	0.1	0.1	0.1	0.1
Q1	Diurnal	26.9	1.1	0.03	0.04	0.0	0.0

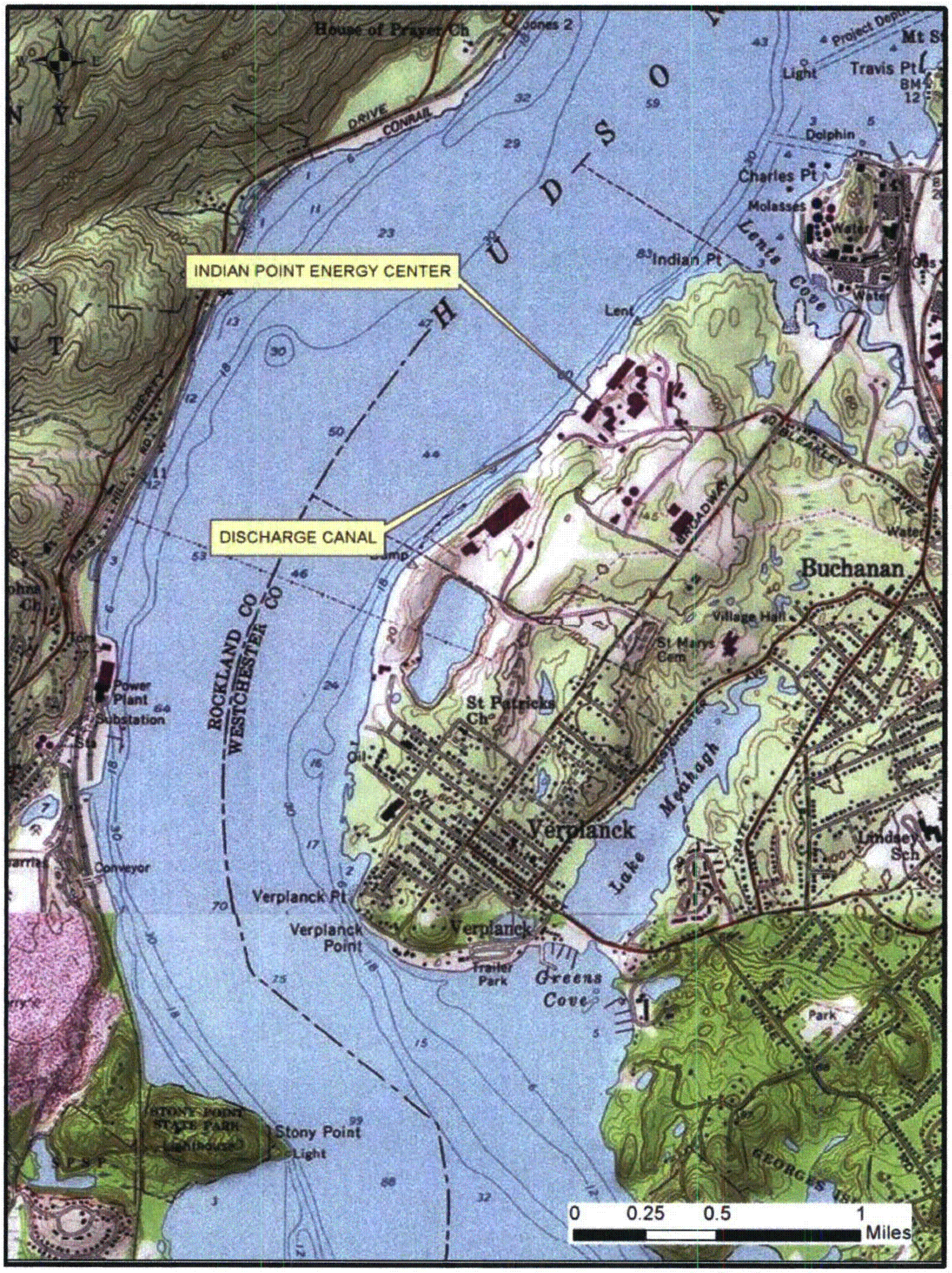

Figure 3.5-1: Site Locus Map with location of the IPEC discharge canal (ESRI, 2012).

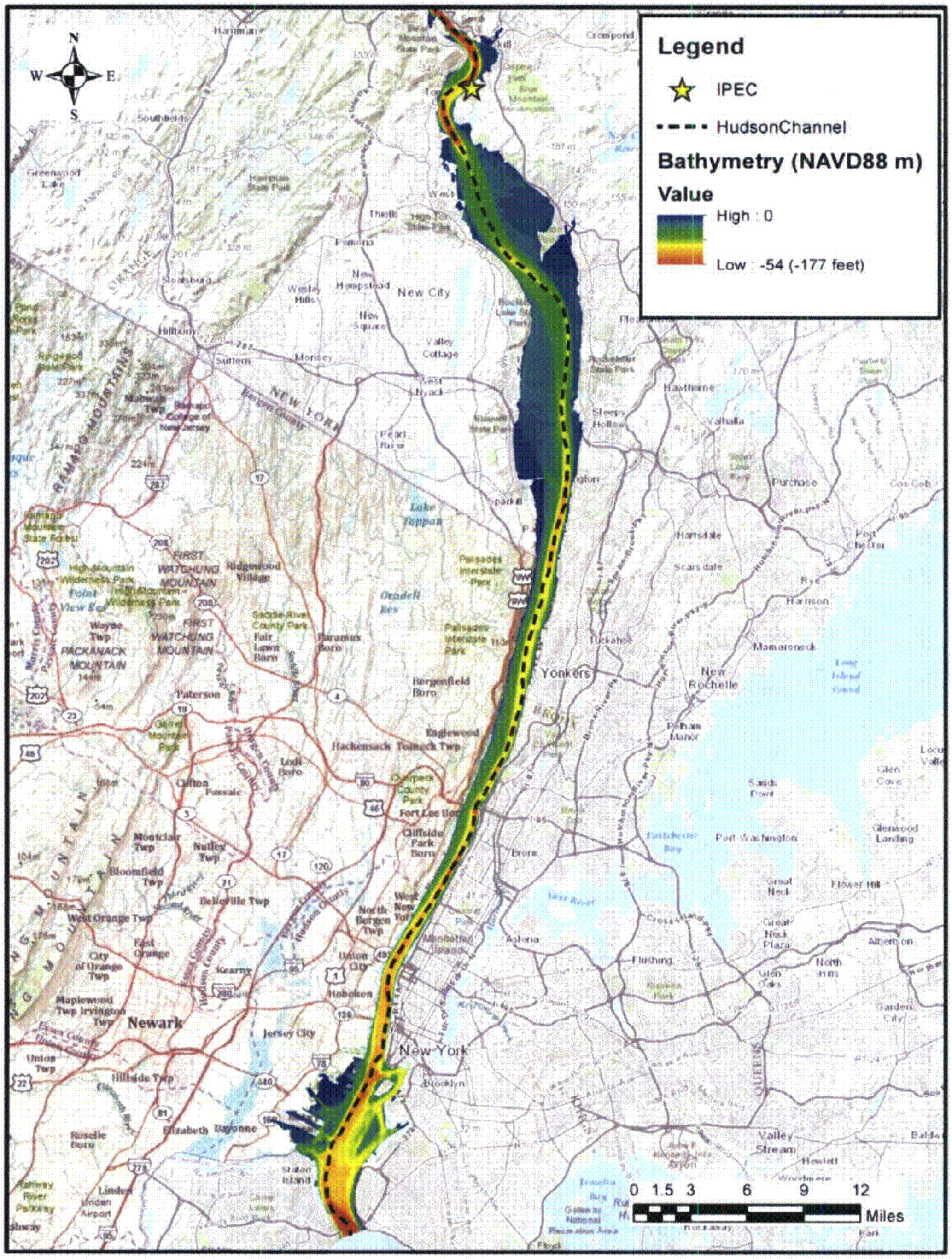

Figure 3.5-2: NYS DEC Lower Hudson River Estuary Program bathymetry.

The dashed line is the approximate location of the deepest channel. The horizontal datum is NAD83 and the vertical datum is NAVD88 (NYS, 2009).

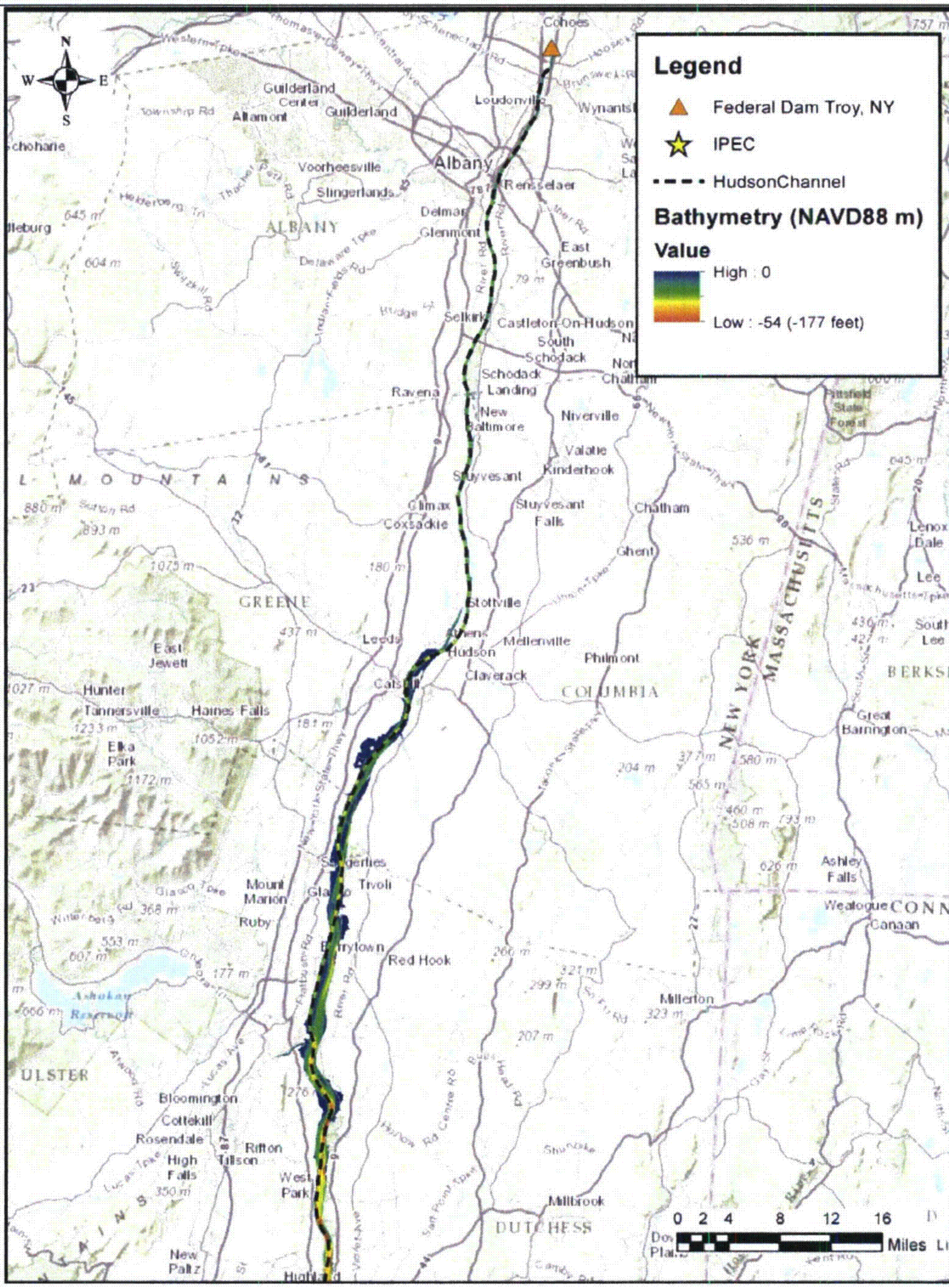

Figure 3.5-3: NYS DEC Upper Hudson River Estuary Program bathymetry.

The dashed line is the approximate location of the deepest channel. The horizontal datum is NAD83 and the vertical datum is NAVD88. Note this portion of the Hudson River is approximately 45 miles upstream of IPEC (NYS, 2009).

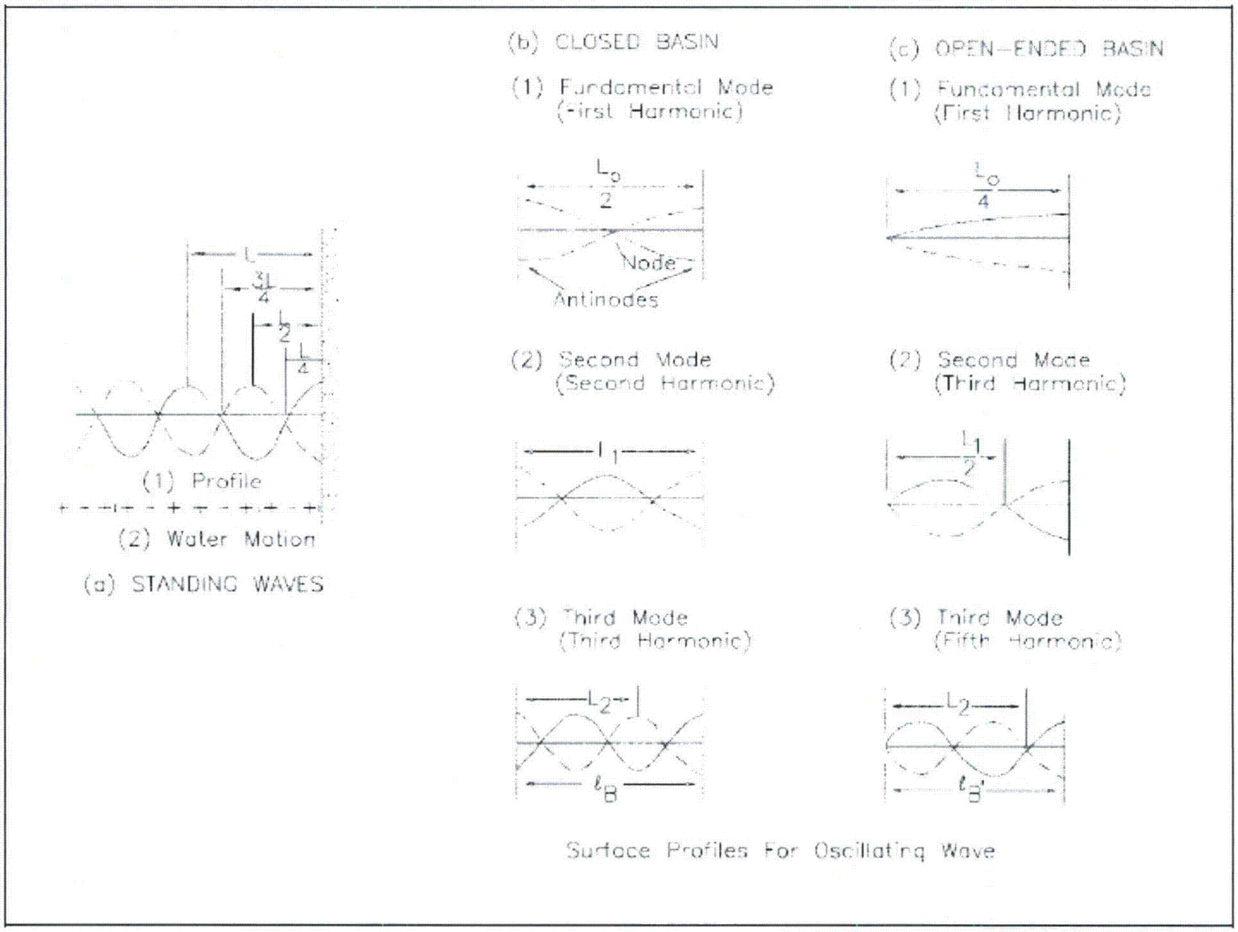

Figure 3.5-4: Diagram of the first three resonant modes of an enclosed (left) and semi-enclosed (right) basin from Scheffner 2008.

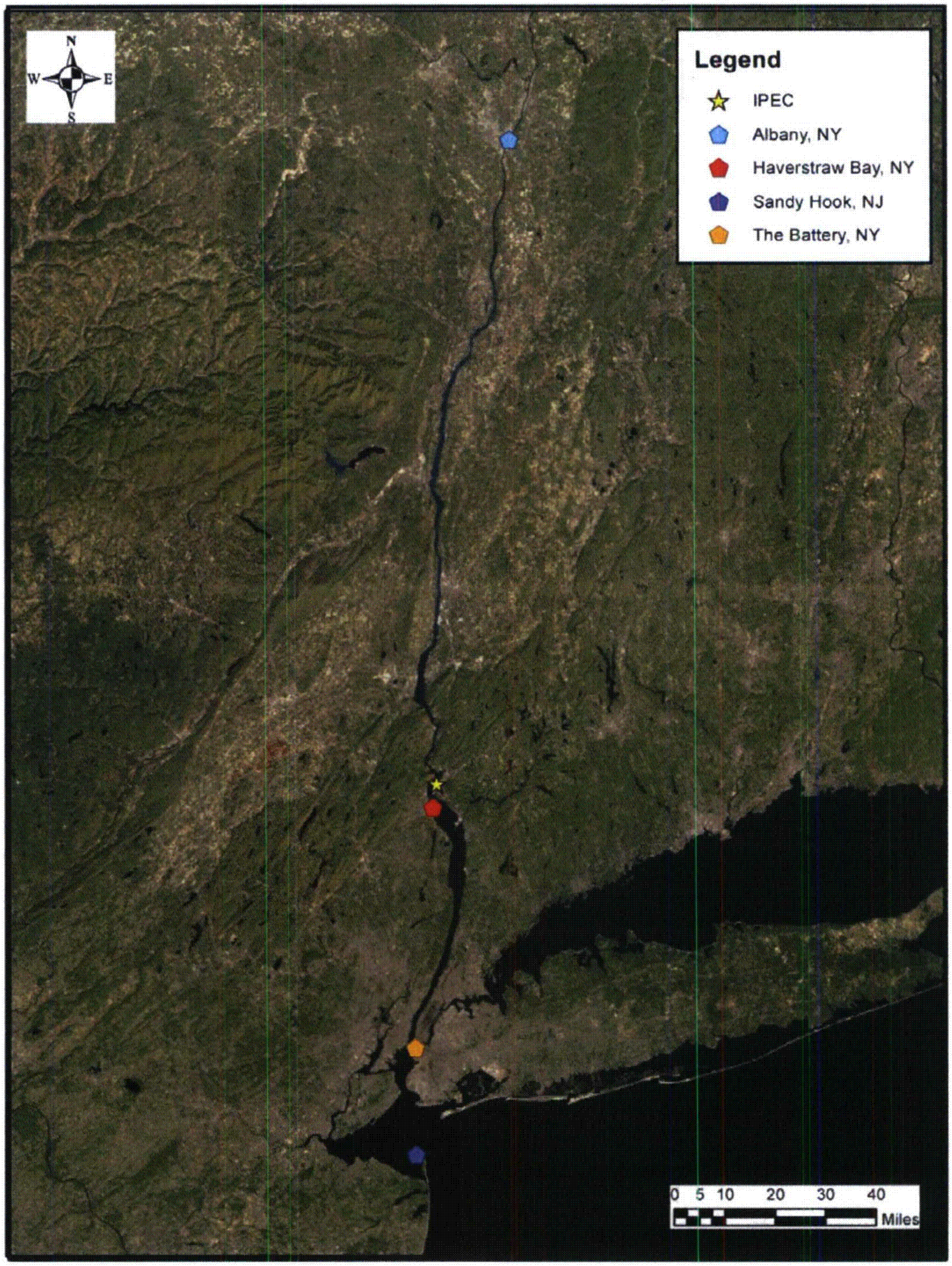

Figure 3.5-5: Map of the Hudson River including the location of IPEC and NOAA Tidal Constituents Stations (NOAA a-d, 2005).

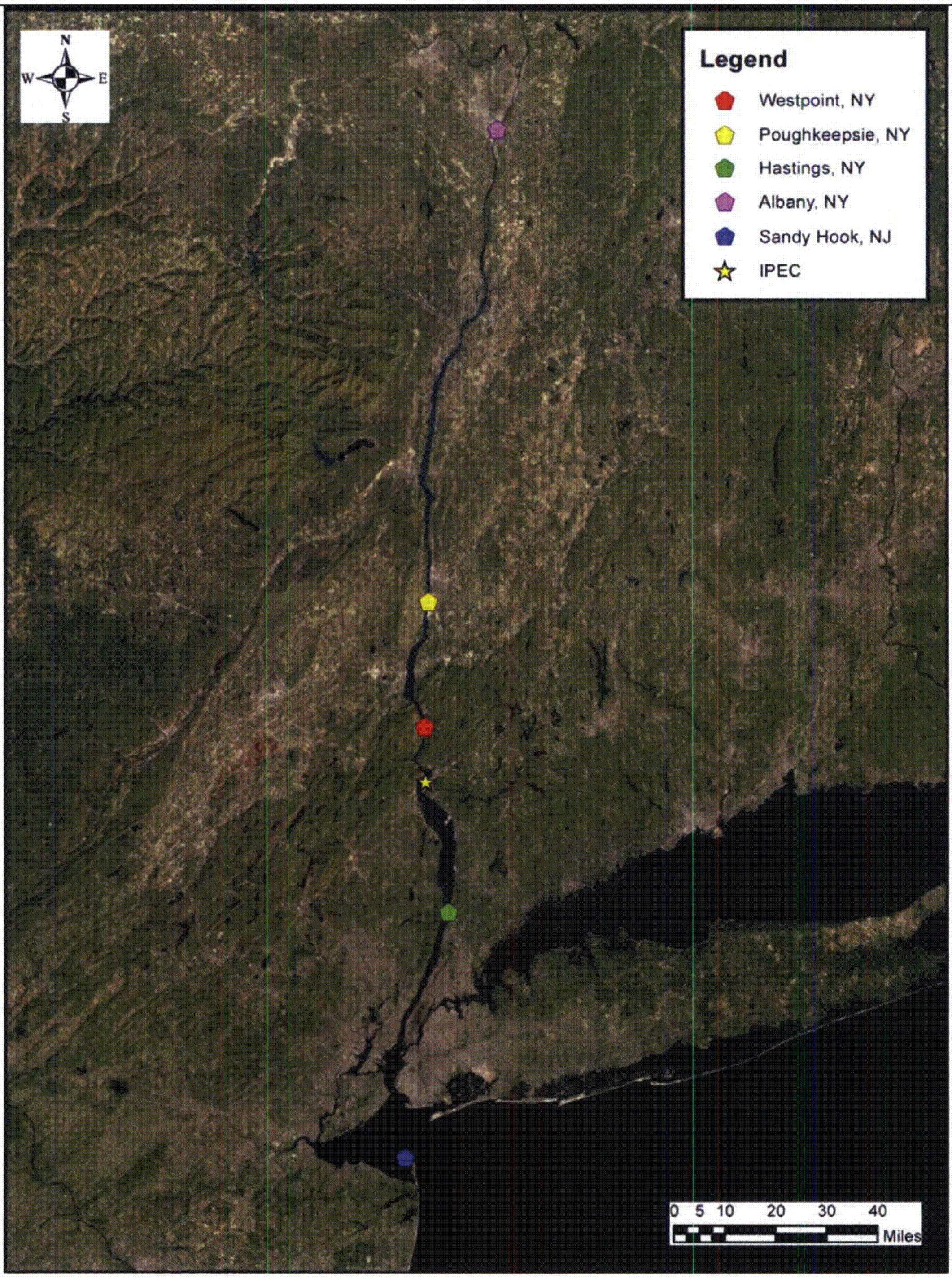

Figure 3.5-6: Map of the Hudson River including the location of IPEC and USGS Stream Gages used for the FFT (USGS, 1991; NOAAa, 2012).

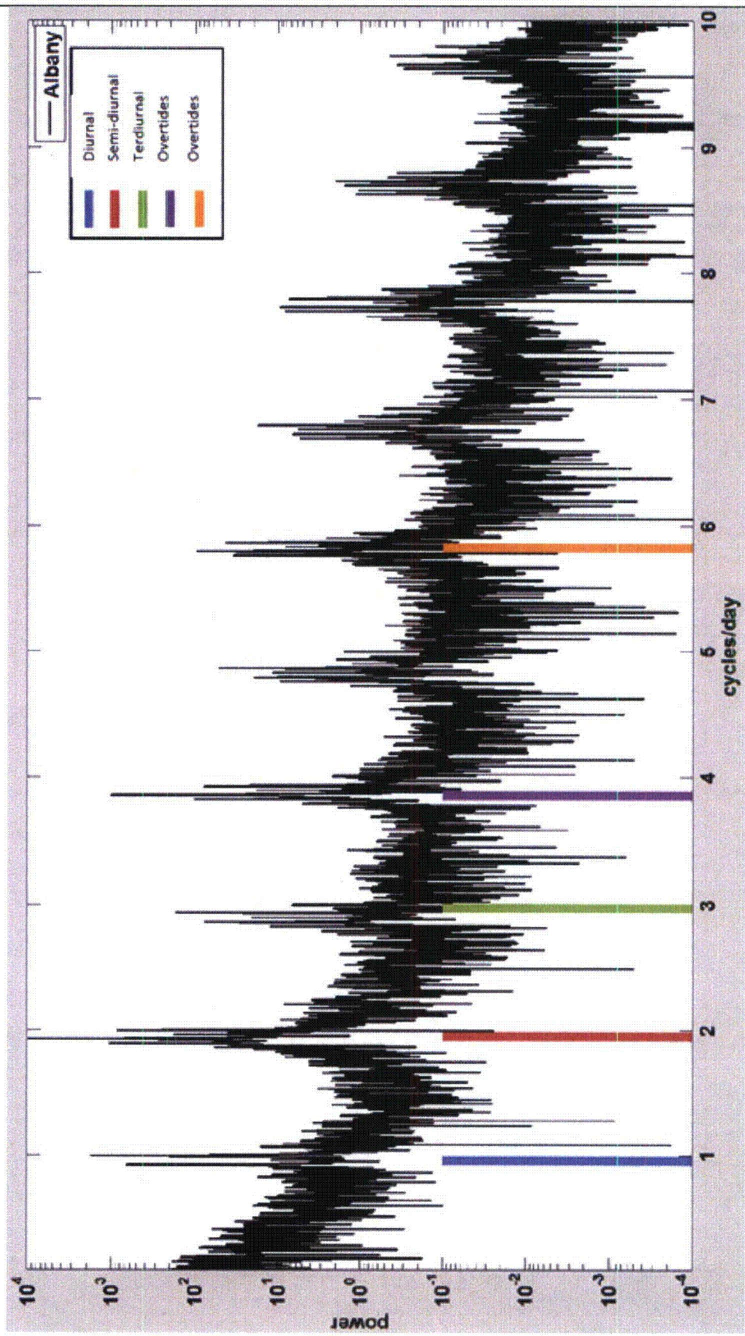

Figure 3.5-7: Spectral Analysis of surface height time series at Albany on the Hudson River.

The y-axis has units of spectral power in this case proportional to feet squared. The x-axis has units of frequency in days or cycles per day.

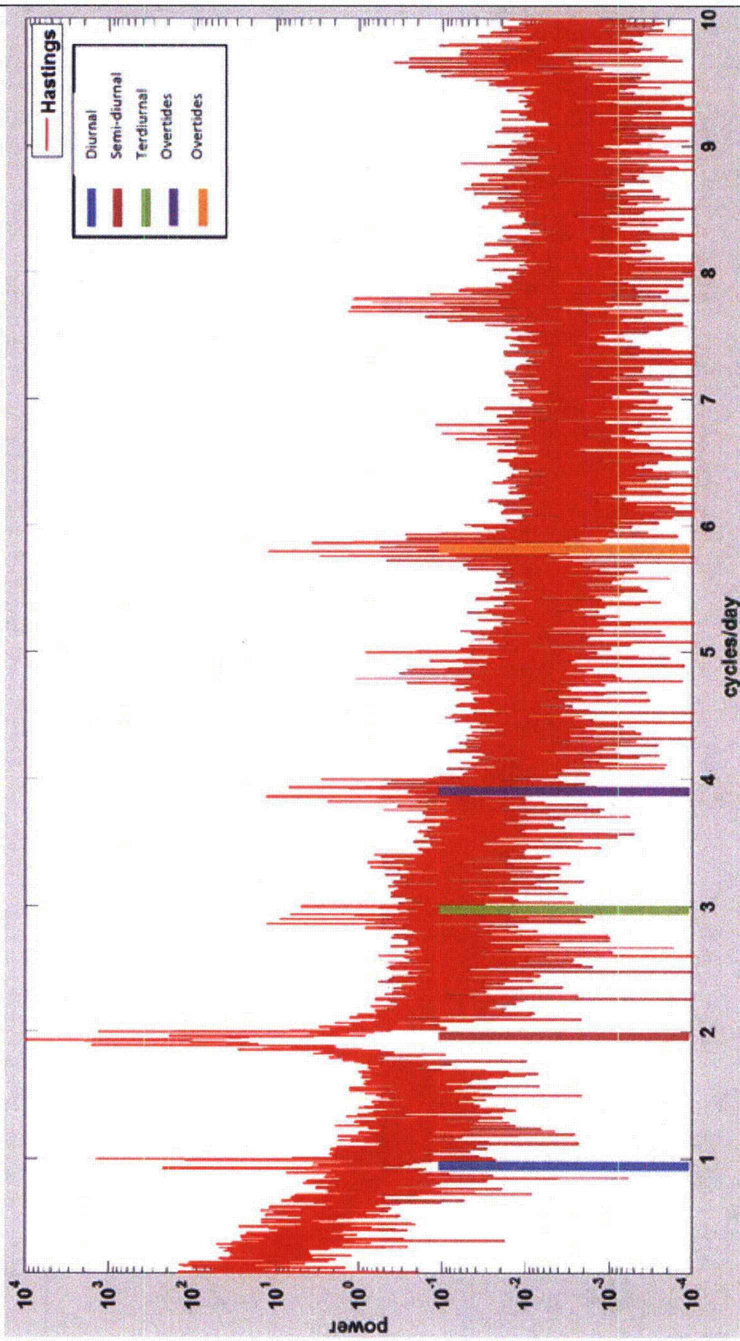

Figure 3.5-8: Spectral Analysis of surface height time series at Hastings on the Hudson River.

The y-axis has units of spectral power in this case proportional to feet squared. The x-axis has units of frequency in days or cycles per day.

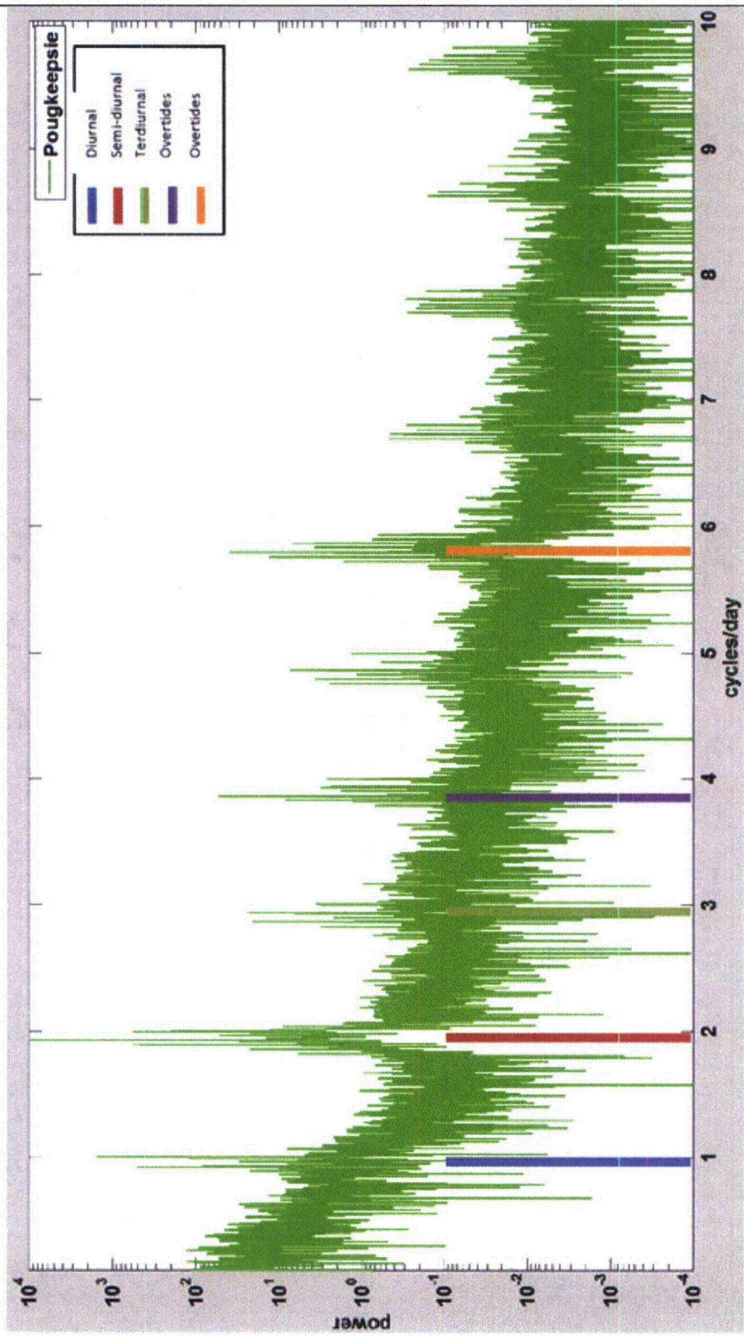

Figure 3.5-9: Spectral Analysis of surface height time series at Poughkeepsie on the Hudson River.

The y-axis has units of spectral power in this case proportional to feet squared. The x-axis has units of frequency in days or cycles per day.

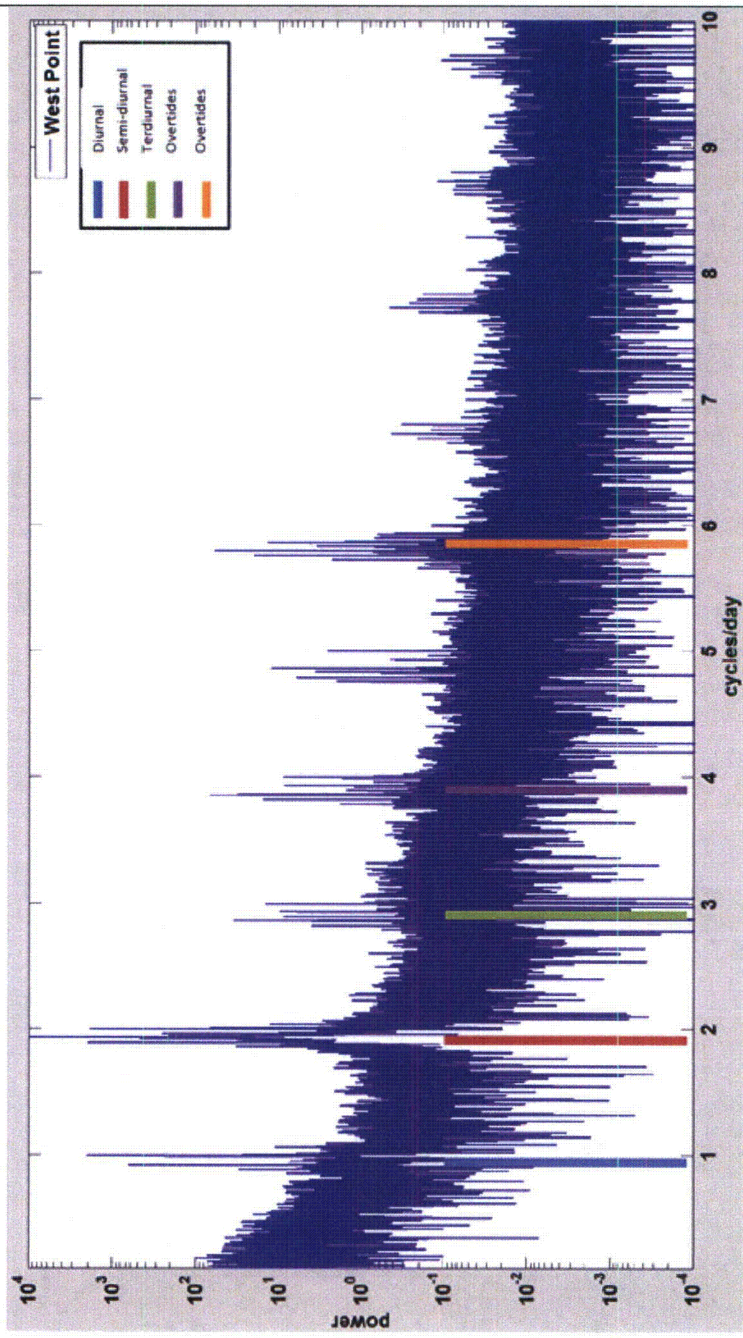

Figure 3.5-10: Spectral Analysis of surface height time series at West Point on the Hudson River.

The y-axis has units of spectral power in this case proportional to feet squared. The x-axis has units of frequency in days or cycles per day.

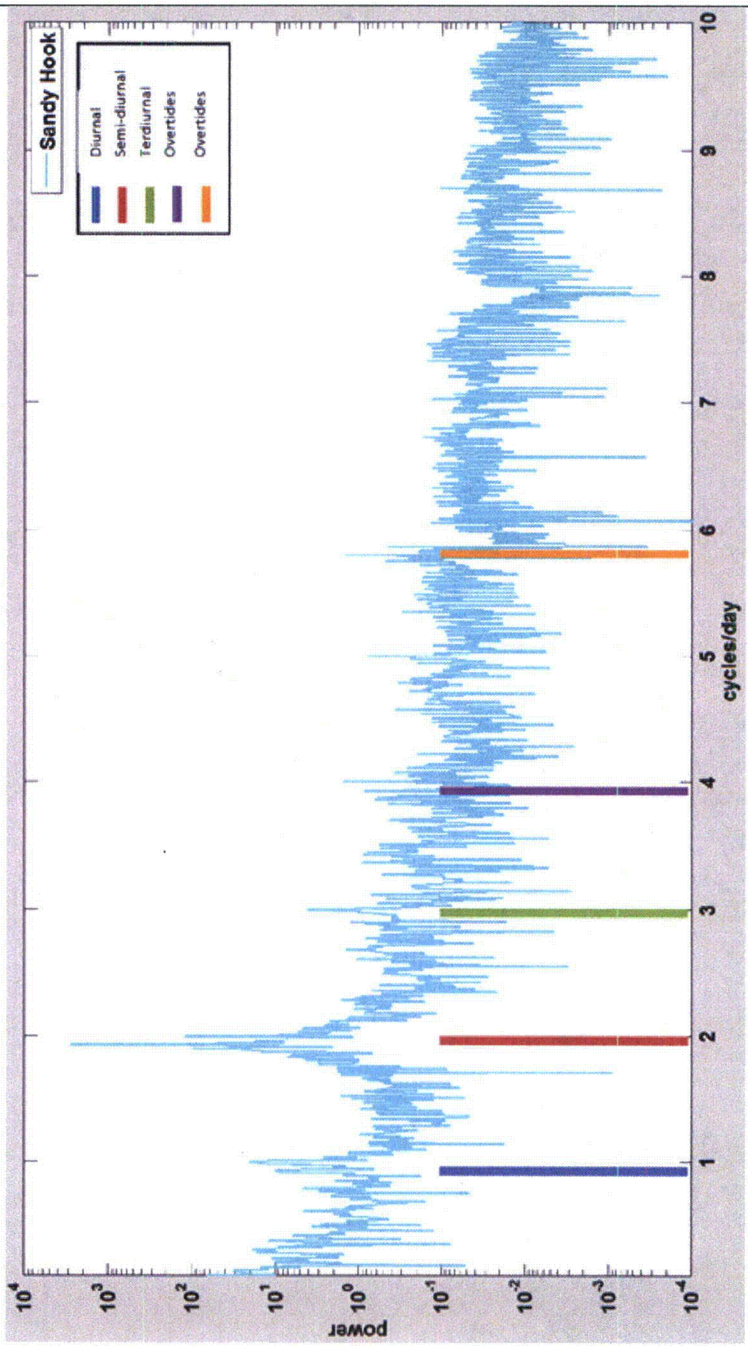

Figure 3.5-11: Spectral Analysis of surface height time series at Sandy Hook on the Hudson River.

The y-axis has units of spectral power in this case proportional to meters squared. The x-axis has units of frequency in days or cycles per day.

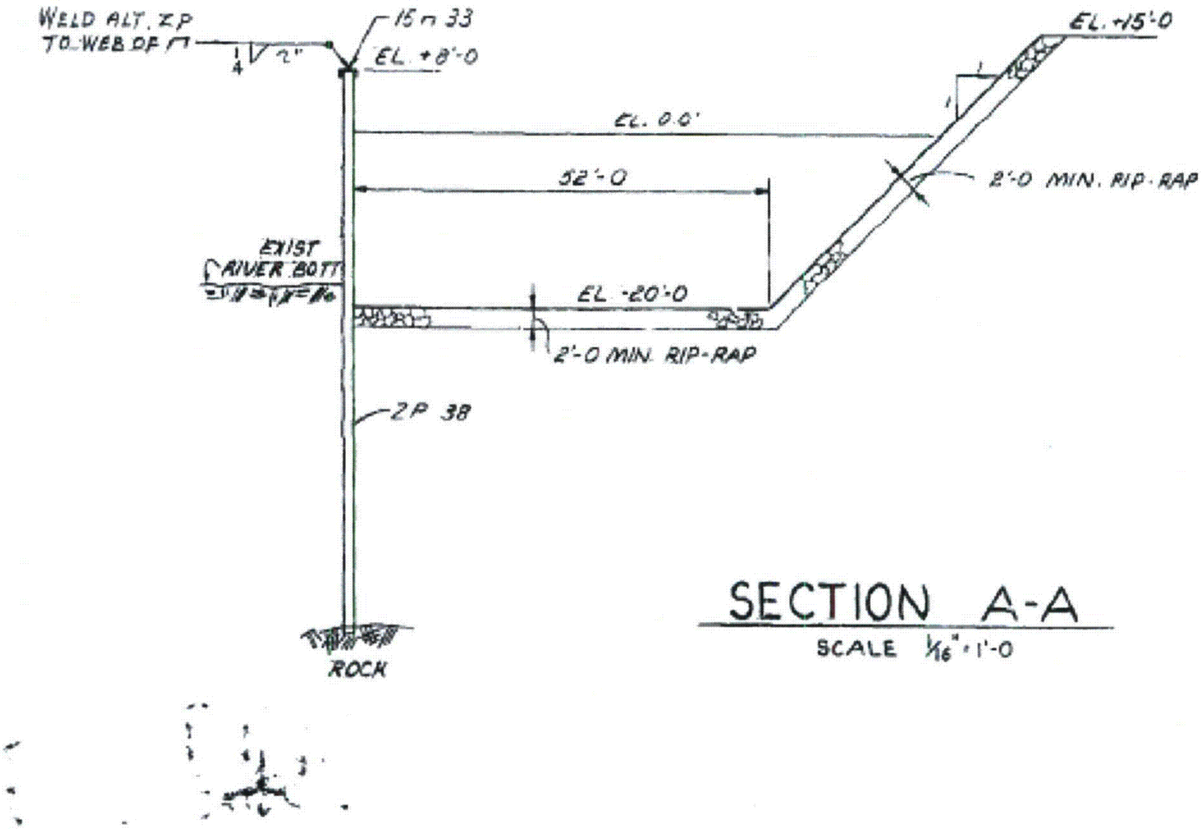

Figure 3.5-12 Profile view of the IPEC discharge canal, (Consolidated, 1970).

3.6 Tsunami

This section addresses the potential for flooding at Indian Point Energy Center (IPEC) due to the probable maximum tsunami (PMT) on the Hudson River.

As defined in NUREG/CR-6966 (U.S. NRC, 2009), the PMT is that tsunami for which the impact at the site is derived from the use of the best available scientific information to arrive at the set of scenarios reasonably expected to affect the nuclear power plant site, taking into account:

1) appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity and period of time in which the historical data have been accumulated;

2) appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena; and

3) the importance of safety functions to be performed.

The Indian Point Energy Center (IPEC) is located approximately 43 mi upstream of the Battery and approximately 60 mi upstream from the ocean coast (See Figure 3.6-1). Due to its location (about 60 miles from the coast), direct inundation of IPEC from a tsunami will not occur. However, given its location on a tidal river which is connected to the ocean, IPEC could potentially experience bores induced by a coastal tsunami propagating up the Hudson River; therefore, evaluation of both the coastal tsunami hazard and likelihood of impact to IPEC was performed.

3.6.1 Method

The approach and methodology used in the tsunami described below. Unless noted otherwise, the approach used in this report is consistent with the following standards and guidance documents:

1. NRC Standard Review Plan, NUREG-0800, revised March 2007 (NRC, 2007);

2. NRC Office of Standards Development, Regulatory Guides:

    a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976 (NRC, 1976);

    b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977 (NRC, 1977);

3. NUREG/CR-7046 – Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, dated November 2011 (NRC, 2011);

4. American National Standard for Determining Design Basis Flooding at Power Reactor Sites, ANSI/ANS 2.8 – 1992 (ANSI/ANS-2.8, 1992); and

5. NUREG/CR-6966 "Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America", Final Report (NRC, 2009).

In accordance with the guidelines presented in NUREG/CD-6966 (NRC, 2009), a hierarchical assessment approach is used to evaluate the tsunami hazard. Relative to tsunami hazards, the hierarchical hazard assessment (HHA) approach consists of the following steps:

1. A Regional Screening Test involving an evaluation of the regional hazard based on a review of the historical record and the best available scientific data.

2. For sites where the Regional Screening Test has identified a tsunami hazard, a Site Screening Test is performed to compare the location and elevation of the plant site with the areas affected by tsunamis in the region.

3. A Detailed Tsunami Hazard Assessment is performed if the screening tests do not establish the safety of the plant. A detailed, site-specific tsunami hazard assessment typically involves identification and modeling of applicable (near-field and far-field) tsunamigenic sources, numerical modeling of wave propagation from the tsunamigenic source to the near shore and numerical inundation modeling of the plant site and vicinity.

The methodology to complete the Regional and Site Screening Tests included:

1. Review of the National Geophysical Data Center (NGDC) Tsunami Event Database (NGDC, 2012) and other sources relative to documented historical tsunamis at or near the site;

2. A literature search to identify the near-field and far-field tsunamigenic sources that are considered a risk relative to generation of tsunamis that may impact the site;

3. Review of published first-order modeling performed for the National Oceanic and Atmospheric Administration (NOAA), as part of the National Tsunami Hazard Mitigation Program (NTHMP, 2012), and others, for the United States (U.S.) East Coast; and

4. Consultation with tsunami subject matter expert, Dr. Stéphan Grilli, PhD, PE. Dr. Grilli is a Professor of Ocean Engineering at the University of Rhode Island and is currently under contract with NOAA-NTHMP to evaluate the tsunami hazard risk along the U.S. East Coast, including performance of the above-referenced modeling and on-going, detailed tsunami hazard assessments of selected areas of the East Coast.

A simplified hydraulic analysis to assess (as part of the Site Screening) whether or not the hydraulic conditions of the Hudson River appear conducive to propagation of bores of significant amplitude to IPEC.

3.6.2 Results (AREVA, 2013a)

3.6.2.1 Regional Screening Test

The regional screening test evaluated the coastal (i.e., near the mouth of the Hudson River) tsunami hazard from both near-field and far-field tsunamigenic sources. The industry technical literature, reflecting the available scientific data on tsunami hazards in the Atlantic Ocean, were compiled and reviewed. That included the NGDC tsunami and earthquake databases, work completed as part of NOAA's National Tsunami Hazard Mitigation Program (NTHMP), and a comprehensive study of tsunami hazard in the Atlantic published by the Atlantic and Gulf of Mexico Tsunami Hazard

Assessment Group (AGMTHAG) and a detailed literature review of tsunamigenic sources affecting tsunami hazard along the U.S. East coast (Grilli et al, 2011).

3.6.2.1.1 Historic Tsunami Record

The NOAA NGDC tsunami database (NGDC, 2012) was queried to identify historic tsunami run-up and source events along the U.S. and Canadian Atlantic coasts. The database query results are summarized in Table 3.6-1. The tsunami runup heights as measured by tide gages along the U.S east coast ranged in height from 0.2 to 2.2 feet (NGDC, 2012).

3.6.2.1.2 Tsunamigenic Sources

Grilli et al, 2011 assesses the regional tsunami hazard for the U.S. East Coast and contains results from relevant tsunami inundation models applied in the region. In addition, during 2007 and 2008, the AGMTHAG, 2008 performed additional evaluation of potential tsunamigenic sources capable of impacting the U.S. Atlantic and the Gulf of Mexico coasts. The findings of both Grilli at al, 2011 and AGMTHAG, 2008 were that the primary tsunamigenic sources defining the tsunami hazard along the U.S. northeast coast are:

- Near-field submarine mass failures (SMFs) (i.e., landslides) along the U.S. Atlantic continental margin;
- Far-field, transoceanic subaerial landslides (the Canary Island's Cumbre Vieja Volcano collapse);
- Far-field earthquakes along the Azores-Gibraltar Plate Boundary (convergence zone); and
- Far-field earthquakes along the Caribbean-North America Plate Boundary (Puerto Rico Trench subduction zone).

Figure 3.6-2 identifies the locations of these sources.

3.6.2.1.3 Near-field Tsunamigenic Sources

Submarine mass failures (SMFs) along the U.S. Atlantic continental margin have potential to generate local tsunamis that impact nearby coastlines. These landslides occurred historically offshore of New England and Long Island, outward of major ancient rivers in the mid-Atlantic, and in the salt dome province offshore of North Carolina (Twichell et al, 2009). Figure 3.6-3 shows the mapped distribution of submarine landslides along the U.S. Atlantic continental slope and rise (AGMTHAG, 2008).

The potential magnitude of landslide-generated tsunamis is extremely site-specific and depends on detailed geometry, location, and slide volume, as well as the mode of rupture. Most of these parameters are poorly understood for observed landslide scars (Grilli et al, 2011). The only documented historic submarine landslide tsunami to impact the North American coastline occurred as a result of the 1929 Laurentian Slope Earthquake near the Grand Banks (magnitude of M_w=7.2). Historic data and tectonics both suggest that similar large tsunamigenic earthquakes are uncommon in the region, and this event represents, to date, the largest earthquake ever recorded in the North American coastal regions of the Atlantic basin (Grilli et al, 2011; Whitmore et al, 2009).

Recent work (Grilli et al, 2011; Grilli et al, 2009 and Grilli et al, 2012) presents a regional assessment of the U.S. East Coast tsunami hazard based on on-going work for NOAA's NTMHP program. First order screening analyses of both near-field and far-field sources are complete and the results, given in terms of near shore breaking wave height and runup, some with defined return periods, are presented below.

Grilli et al, 2009 developed a probabilistic Monte Carlo Simulation (MCS) approach to determine SMF tsunami hazard along the upper northeast U.S. coast, which was refined and extended to the entire U.S. East coast by Krause, 2011 and Grilli et al, 2012 as part of the NOAA-NTHMP project. In this analysis, the tsunami hazard was assessed from local SMF sources triggered on the continental slope by moderate seismic activity, by performing simplified stochastic slope stability and tsunami generation/propagation analyses. The results of this preliminary, screening-level analysis provide first-order estimates of wave amplitude and runup. Over 15,000 stochastic stability analyses of submerged slopes were performed along each of the 45 actual shelf transects in the Northeast and then again along the additional 46 transects (See Figure 3.6-4) that were added for the remaining portion of the coast. The slope stability transect locations were defined in areas with homogeneous topographic features and the slope stability analysis used actual bathymetry for each transect. Grilli et al, 2009 concluded that, for most locations, the overall coastal hazard resulting from 100-year and 500-year submarine landslide generated tsunamis was relatively low and similar to a typical 100-year storm surge in the region, roughly 13 to 16.5 feet in the study region (USACE, 2002). For the 500-year return period tsunami, Grilli et al, 2009 presented two regions of relatively elevated hazard:

1) near Long Island, NY with a peak run-up of 10 ft (3 m); and

2) near the New Jersey coast, with a peak run-up of 13 feet and peak breaking wave height of 20 to 23 feet at breaking distances (from the coastline) of about 800 to 1000 feet.

The most notable submarine landslide complex along the U.S. Atlantic continental margin is the Currituck Landslide, located off the coast of North Carolina. Geist et al, 2009 describes the complex as one of the "largest landslides along the North American Atlantic offshore margin Geist et al, 2009 modeled the Currituck Landslide tsunami event in detail. Results of this modeling showed large variation in both the maximum runup and near shore wave amplitude at a location broadside of Currituck Landslide. The maximum wave runup at Currituck Banks ranged from about 4 to 29 feet, while the near shore wave amplitude at the same location ranged from approximately 9.8 to 22.9 feet. Results from the three different slide scenarios at a shore location are shown in Figure 3.6-5.

Owing to both its similarity in mechanism and nature to most of the other SMFs observed along the North American Atlantic margin (See Figure 3.6-3), the Currituck Landslide is a relevant SMF tsunami hazard assessment for the coastal region near the Hudson River mouth. In the absence of more accurate site-specific field data, the Currituck Landslide is typically used by NOAA as a proxy for the deterministic source of maximum probable SMF tsunami, to compute tsunami hazard maps along the upper U.S. East coast.

3.6.2.1.4 3.2.2.1.2.2 Far-field Tsunamigenic Sources

Due to the location of the Hudson River mouth on the U.S. northeast coast, IPEC may be impacted by tsunamis generated by far-field sources. Grilli et al, 2011 identified the following discussions of far-field tsunamigenic sources that could impact the Hudson River mouth.

Subaerial and Submarine Landslides: The most notable far-field landslide source results from the possible large scale flank collapse of the Cumbre Vieja, a volcano on La Palma in the Canary Islands. Abadie et al. (Abadie et al 2009, 2012, 2011) and Harris et al. (Harris et al, 2012), as part of the work completed for NOAA-NTHMP, recomputed both the tsunami generation and near-field and far-field propagation to the US East Coast, for Cumbre Vieja source. The maximum tsunami elevations for the U.S. East Coast, from a 19 mi^3 slide scenario, presented in Figure 3.6-6, show some of the highest tsunami elevations (on the order of 10 feet) occurring off of northern New Jersey and Long Island. They found that, for the extreme case of 108 mi^3, waves of 26 to 33 feet in height could reach the continental shelf at some locations in the general area of the Hudson River mouth. Propagation over the shelf, however, which is site-specific by nature, caused significant dissipation through breaking of these waves, reducing their coastal impact.

Azores-Gibraltar Convergence Zone: The Azores-Gibraltar convergence zone is another significant tsunamigenic source in the Atlantic basin. There are numerous potentially active faults within the convergence zone, including the Gorringe Bank Fault, the Marque de Pombal Fault, the St. Vincente Fault, and the Horseshoe fault. Those faults are presently active and collectively a source of some of the largest historical earthquakes and tsunamis in the Atlantic Ocean, including the 1755 Great Lisbon Earthquake and tsunami (Grilli et al, 2011).

From historical records, it is known that the 1755 Great Lisbon Earthquake, along with its ensuing tsunami, caused waves of 16.4 to 49.2 feet along the southwest coast of Iberia and Morocco. Runup from the trans-oceanic tsunami was documented in the Caribbean, Brazil, and Newfoundland. There were no reports documenting flooding or runup along the U.S. East Coast, despite the presence of low lying population centers along the U.S. and Canadian Atlantic coasts at the time (Grilli et al, 2009; NGDC, 2012). AGMTHAG, 2008 suggests that the lack of historic runup reports along the U.S. East Coast may be due to the broad continental shelf, which contributed to dissipation. Additionally, the Gorringe Bank and the Madeira-Tore Rise (MTR) appear to have acted as topographic scatters, protecting most of the U.S. East Coast (AGMTHAG, 2008). Mader (Mader, 2001) employed the nonlinear SWAN model to simulate the 1755 Lisbon earthquake using a location near the Gorringe Bank. Mader's model simulations showed that the U.S. East Coast could have received maximum deep water tsunami wave amplitudes of 6.6 feet with a runup of approximately 10 feet.

Barkan et al. (Barkan, 2009) performed additional simulations to assess tsunami hazard to the U.S. East Coast from potential future earthquakes along the Azores-Iberia plate boundary west of the Madeira-Tore Rise (MTR) or in the Gulf of Cádiz would only produce deep water tsunami amplitudes of roughly 0.26 feet off the coast of New York (Barkan, 2009). Furthermore, tsunamis generated by earthquakes in the Gulf of Cádiz would produce maximum wave amplitudes less than 1.3 feet (0.4 m) along the New Jersey and New York coastlines (AGMTHAG, 2008).

Caribbean Subduction Zone: The Caribbean region is characterized as an active seismic area and is associated with a large number of past tsunamis. The Caribbean plate is bounded to the north by a major transform fault at the boundary with the North American plate and to east by the large and subducting South American plate. Its movement with respect to the North American plate causes volcanic eruptions and earthquakes in the region (Grilli et al, 2011; Zahibo et al, 2001). The Puerto Rico Trench (PRT) is the Caribbean fault that is most likely to cause tsunamis that could reach the U.S. Atlantic coast because of its location and east-west orientation (Grilli et al, 2011). The lack of a large earthquake in the PRT in recent history and evidence of internal stress-build up in the nearby subduction zone indicate that a large and potentially tsunamigenic earthquake in the trench may be imminent (Grilli et al, 2011).

Due to potential for a large event, Knight, 2006 developed a first-order estimate of the most extreme earthquake that could occur in the PRT, a magnitude 9.1 earthquake resulting in an average slip of 39 feet (Grilli et al, 2011). Knight also assessed the resulting potential tsunami hazard. He demonstrated that wave arrivals resulting from an earthquake occurring in the Puerto Rico trench along the Atlantic coast have leading elevations with amplitudes up to 5 feet in Atlantic City, NJ and 2.2 feet in Montauk, NY. Knight noted that the far-field tsunamis generated from earthquakes in the Caribbean Sea cause higher amplitudes along the Gulf coast than the Atlantic coast due to dissipation through the Greater Antilles Islands. Grilli et al, 2010 performed a study for both an extreme (magnitude 9.1) and a smaller (magnitude 8.7) earthquake (which, based on limited historical records, were estimated to correspond to 600 and 300-year return periods, respectively) along the Puerto Rican Trench. Their results suggest that a tsunami resulting from the extreme 9.1 magnitude earthquake could cause a maximum runup of 6.6 to 9.8 feet in most locations along the upper U.S. East Coast. Grilli et al, 2012 also examine a similar extreme 9.0 magnitude earthquake and also show the results on a low resolution regional grid, which is shown to cause a maximum tsunami elevation up to about 3 feet along the continental margin off of New York.

In summary, the results of the Regional Screening Test did not identify any historical tsunami impacts (runup heights greater than 2 to 3 feet). Several near field and several far field sources were identified that, under very conservative assumptions, could result in tsunamis that affect the coastal area in the vicinity of northern NJ and Long Island. Preliminary, deterministic first-order models, conservatively assuming "worst case" failures at each of these sources indicated the following:

For near-field tsunamigenic sources, slope stability calculations for the 500-year return period tsunami, Grilli et al, 2009 identified two regions of relatively elevated hazard due to submarine mass failures (SMF) along the continental shelf:

1) near Long Island, NY with a peak run-up of 10 ft; and

2) near the New Jersey coast,

with a peak run-up of 13 feet and peak breaking wave height of 20 to 23 feet at breaking distances (from the coastline) of about 800 to 1000 feet. These regions were also identified in Krause, 2011 as having elevated hazard; however runup was estimated on the order of 16 to 20 feet. The most notable submarine landslide complex along the U.S. Atlantic continental margin is the Currituck Landslide, located off the coast of North Carolina. The Currituck Landslide is often conservatively used by NOAA as a source for the deterministic analysis of SMF tsunami impact at specific sites along the U.S. East Coast, even if they are not located across from the actual Currituck Landslide. The maximum modeled wave runup at Currituck Banks ranged from about 4 to 29 feet, while the near shore wave amplitude at the same location ranged from approximately 9.8 to 22.9 feet.

The Hudson Canyon is a major feature of the continental slope and rise offshore of New York. Due to the Hudson River Canyon bathymetry, long tsunami waves from the far-field sources are expected to significantly refract away from the Hudson River Canyon, reducing their propagation toward the Hudson River (Harris et al, 2012), and thereby the tsunami risk to the Hudson River.

For far-field tsunamigenic sources the following potential sources were identified:

a. <u>Cumbre Vieja Volcano (Canary Islands)</u>: The maximum modeled tsunami elevations ranged from about 10 feet to 33 feet occurring offshore of northern New Jersey and Long Island

(Abadie et al, 2012). The models indicated that propagation over the continental shelf caused significant dissipation through breaking of these waves, reducing their coastal impact.

b. Azores-Gibraltar Convergence Zone: Modeled tsunami elevations and runup are insignificant in the vicinity of northern NJ and Long Island (Abadie et al, 2010).

c. Caribbean Subduction Zone: Modeled tsunami elevations and runup are insignificant in the vicinity of northern NJ and Long Island (Grilli et al, 2012).

3.6.2.2 Site Screening Test

The results of a simplified, probabilistic first-order modeling of the SMFs along the continental shelf identified the potential for coastal impact with estimated peak breaking wave heights of 20 to 30 feet at breaking distances (from the coastline) of about 800 to 1,000 feet. Due to the significant distance of IPEC from the coast (about 60 miles) as well as the significant topographic relief present between IPEC and the coast, inundation in the vicinity of IPEC will not occur due to coastal tsunami run-up. Also, by inspection of the river geometry and IPEC's distance from the coast, it appears that significant attenuation of waves (or bores), should any propagate up the river, will occur before reaching IPEC.

To evaluate whether the hydraulic conditions of the river appear conducive to bore propagation up to the site, a simple, unsteady hydraulic analysis was performed as part of the Site Screening using the USACE unsteady, one-dimensional model HEC-RAS Version 4.1. The unsteady component of the HEC-RAS modeling system uses numerical, finite difference analysis and is capable of simulating one-dimensional unsteady flow through a full network of open channels. The unsteady flow equation performs mixed flow regime (subcritical, supercritical, hydraulic jumps, and drawdowns) unsteady flow calculations. Model details and calibration are presented in AREVA Document No. 32-9196316-000 "Probable Maximum Flood on Hudson River – Hydraulics" (AREVA, 2013b). The HEC-RAS analysis was not used to specifically model tsunami bore propagation since it is not a dispersive model and the model physics do not capture all the components of tsunami bore propagation and tidal interaction. The hydraulic behavior of the river was evaluated by applying a series of five 12-minute gravity waves at the river mouth (input as temporally-varying, elevated water levels).

The model boundary condition water level ("wave" series) was input at the mouth of the Hudson River in Lower Bay with amplitudes of 20 feet and periods of 18 minutes. The first three waves were 20 feet high followed by two smaller waves of 18 feet and 12 feet. These amplitudes and periods correspond to those presented in Figure 3.6-5. The model included normal river flow (tidally-influenced) prior to and during propagation of the gravity wave. The downstream boundary condition was adjusted by superimposing the wave height upon the natural tidal cycle, such that the maximum water surface elevation coincided with high tide at IPEC. The locations of the cross sections used in the HEC-RAS analysis are presented in Figure 3.6-7.

The model results are presented in Table 3.6-2. The model results indicated that that the "wave" amplitudes will likely be significantly attenuated before reaching IPEC. The model also indicated that supercritical flow conditions (required for significant amplification of bores within rivers) will not likely occur.

3.6.3 Conclusions

A probable maximum tsunami event is not considered to be a significant flood hazard for IPEC for the following reasons:

- The results of the Regional Screening Test indicate that the likelihood of a significant tsunami along the U.S. East Coast is low. Review of the NOAA NGDC tsunami database (NGDC, 2012) did not identify any documented, historic tsunami events along the U.S. East Coast that resulted in significant, historical tsunami impacts (runup heights greater than 2 to 3 feet).

- Due to the presence of the Hudson Canyon , tsunami waves from the far-field sources are expected to significantly refract away from the Hudson River Canyon, reducing their propagation toward the Hudson River and thereby the tsunami risk to the Hudson River. As a result, the event resulting in the probable maximum tsunami will likely be a near-field SMF along the continental shelf.

- Due to the significant distance of IPEC from the coast (about 60 miles) as well as the significant topographic relief present between IPEC and the coast, inundation in the vicinity of IPEC will not occur due to coastal tsunami run-up.

- By inspection of the river geometry and IPEC's distance from the coast, it appears that significant attenuation of waves (or bores), should any propagate up the river, will occur before reaching IPEC.

- The results of a simple, unsteady hydraulic analysis indicated that that "wave" amplitudes will likely be significantly attenuated before reaching IPEC, to levels that would not impact IPEC SSCs.

3.6.4 References

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Table 3.6-1: Historic Tsunami Run-up events along the U.S. East coast
(Reference NGDC, 2012)

Date	Time	Validity	Source	Source Location	Run-up Location along the U.S. East Coast	Measurement Type	Run-up Height (m)	Maximum Run-up Location	Maximum Run-up Height (m)
11/01/1755	8:50	Definite	Earthquake	Lisbon, Portugal	-	-	-	Lagos, Portugal (30.1°N,8.667°W)	30
09/24/1848	-	Probable	Landslide	Fishing Ships Harbor, Newfoundland, Canada	-	-	-	-	-
06/27/1864	22:30	Probable	Earthquake	SW Avalon Peninsula, Newfoundland, Canada	-	-	-	-	-
09/01/1886	2:51	Definite	Earthquake	Charleston, SC	Jacksonville, FL (30.317°N,81.65°W)	Water height measurement	-	-	-
					Mayport, FL (30.39°N,81.43°W)				
					Copper River, SC (32.87°N,79.93°W)				
09/01/1895	11:09	Probable	Earthquake	High Bridge, NJ	Long Island, NY (40.591°N,73.796°W)	Water height measurement	-	-	-
12/06/1917[2]	13:04	Definite	Explosion	Halifax, Nova Scotia	Bedford, Nova Scotia (44.729°N, 63.664°W)	-	-	Halifax, Nova Scotia (44.633°N, 63.583°W)	7
					Dartmouth, Nova Scotia (44.667°N, 63.583°W)	-	-		
					Halifax, Nova Scotia (44.633°N, 63.583°W)	-	7		
					Tufts Cove, Nova Scotia (44.68°N, 63.6°W)	-	-		
					Turtle Grove, Nova Scotia (44.68°N, 63.6°W)	-	-		
10/11/1918	14:14	Definite	Earthquake	Puerto Rico (PR), Mona Passage	Atlantic City, NJ (39.364°N,74.423°W)	Tide-gage measurement	0.06	Punta Agujereada, PR (18.51°N,67.167°W)	6.1
11/18/1929	20:32	Definite	Earthquake & Landslide	Grand Banks, Newfoundland, Canada	Ocean City, MD (38.333°N,75.083°W)	Tide-gage measurement	0.3	Taylor's Bay, Newfoundland (46.883°N,54.267°W)	7
					Atlantic City, NJ (39.35°N,74.417°W)		0.68		
					Charleston, SC (32.75°N,79.916°W)		0.12		

[2] Although the tsunami of 1917 is considered definite by the NGDC, it was not caused naturally. The tsunami was the result of an explosion that occurred from the collision of two ships in the Narrows of Halifax Harbour.

Date	Time	Validity	Source	Source Location	Run-up Location along the U.S. East Coast	Measurement Type	Run-up Height (m)	Maximum Run-up Location	Maximum Run-up Height (m)
08/04/1946	17:51	Definite	Earthquake	Northeastern Coast, Dominican Republic (DR)	Daytona Beach, FL (29.20°N,81.017°W)	Tide-gage measurement	-	Rio Boba, DR (19.467°N,69.867 °W)	5
					Atlantic City, NJ (39.364°N,74.423°W)			Nagua, DR (19.417°N,69.817 °W)	5
08/08/1946	13:28	Definite	Earthquake	Northeastern Coast, Dominican Republic (DR)	Daytona Beach, FL (29.21°N,81.02°W)	Tide-gage measurement	-	San Juan, PR (18.483°N,66.133 °W)	0.6
					Atlantic City, NJ (39.364°N,74.423°W)				
05/19/1964	0:00	Probable	Landslide	Long Island, NY	Montauk, NY (41.033°N,71.950°W)	Tide-gage measurement	0.1	Plum Island, NY (41.181°N,72.194 °W)	0.28
					Plum Island, NY (41.181°N,72.194°W)		0.28		
					Willetts Point, NY (40.683°N,73.283°W)		0.1		
					Newport, RI (41.493°N,71.327°W)		0.1		
12/26/2004	0:58	Definite	Earthquake	Off Sumatra, Indonesia	Trident Pier, FL (28.415°N,80.593°W)	Tide-gage measurement	0.17	Indonesia (entire country)	50.9
					Atlantic City, NJ (39.35°N,74.417°W)		0.11		
					Cape May, NJ (38.97°N,74.96°W)		0.06		
10/28/2008	-	Probable	Unknown	Maine	Boothbay Harbor, ME (43.8523°N, 69.6281°W)	Eyewitness Measurement	3.6	Boothbay Harbor, ME	3.6
					Bristol, ME (43.95°N, 69.5°W)	-			
					Portland, ME (43.642°N, 70.285°W)	Tide-gage measurement	0.1		
					Southport, ME (43.833°N, 69.65°W)	-			
					Fort Point, NH (43.072°W, 70.712°W)	Tide-gage measurement	0.06		

Table 3.6-2: Lower Hudson River Hydraulics HEC-RAS Model Results

Reach	River Station	Profile	Flow Total	Minimum Channel Elevation	Water Surface Elevation*	Critical Water Surface	Energy Grade. Elevation*	Energy Grade Slope	Velocity Channel	Flow Area	Top Width	Froude # Channel
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Lower	236858.9	Max WS	22920	-72.3	9.64		9.64	0	0.13	182015.7	4221.86	0
Lower	233196	Max WS	21293.29	-54.9	9.64		9.64	0	0.12	176696.5	3691.68	0
Lower	229934	Max WS	19850.81	-51.9	9.63		9.63	0	0.11	186401.6	4290.34	0
Lower	226446.1	Max WS	18057.31	-48.3	9.62		9.62	0	0.09	202581.1	5147.89	0
Lower	222734.2	Max WS	15807.82	-45	9.6		9.6	0	0.07	218648.1	6063.97	0
Lower	218967.3	Max WS	13545.88	-64.7	9.57		9.57	0	0.06	208575.8	5392.66	0
Lower	215585.1	Max WS	11676.6	-41.7	9.54		9.54	0	0.05	222531.5	5802.97	0
Lower	212259	Max WS	9909.7	-41.7	9.5		9.5	0	0.05	211778.6	5465.71	0
Lower	208904.3	Max WS	8315.04	-45	9.46		9.46	0	0.04	198049.3	5233.76	0
Lower	205342	Max WS	6655.95	-45	9.4		9.4	0	0.03	211906.8	6129.64	0
Lower	201621.5	Max WS	4692.04	-41.7	9.34		9.34	0	0.02	232562.1	8046.68	0
Lower	198246.2	Max WS	2930.92	-38.4	9.28		9.28	0	0.01	238053.8	7441.91	0
Lower	194792.2	Max WS	1394.11	-41.7	9.21		9.21	0	0.01	248317.9	7811.07	0
Lower	190959.9	Max WS	-240.65	-41.7	9.13		9.13	0	0	267504	10181.65	0
Lower	188707.5	Max WS	-1022.03	-51.6	9.07		9.07	0	0	236511.9	7199.92	0

Reach	River Station	Profile	Flow Total	Minimum Channel Elevation	Water Surface Elevation*	Critical Water Surface	Energy Grade. Elevation*	Energy Grade Slope	Velocity Channel	Flow Area	Top Width	Froude # Channel
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Lower	186051.7	Max WS	-1552.21	-71.2	9		9	0	-0.01	200710.7	5084.27	0
Lower	182609.9	Max WS	-1854.51	-81.1	8.87		8.87	0	-0.01	160282	2729.92	0
Lower	179515.2	Max WS	-83380.8	-58.1	8.74		8.74	0	-0.51	164898.1	3358.47	0.01
Lower	176336.8	Max WS	-85255.5	-119.3	8.6		8.61	0	-0.55	155710.8	2160.9	0.01
Lower	174008.8	Max WS	-86339.4	-69.2	8.5		8.51	0	-0.54	159222.1	2877.41	0.01
Lower	170788.1	Max WS	-88304.6	-82.9	8.37		8.37	0	-0.49	179102.3	5089.46	0.01
Lower	166929.8	Max WS	-89936.3	-143.4	8.2		8.21	0	-0.54	168451.3	1808.73	0.01
Lower	162829	Max WS	-90373.7	-169.8	8.03		8.03	0	-0.41	222155.6	3425.83	0.01
Lower	159704.5	Max WS	-90627.9	-103.7	7.89		7.89	0	-0.61	149646.9	1929.46	0.01
Lower	156907.2	Max WS	-90801.6	-67.1	7.74		7.75	0	-0.6	150270.6	2654.79	0.01
Lower	153365.6	Max WS	-180552	-81.7	7.56		7.59	0.000001	-1.3	138366.7	1886.98	0.03
Lower	149596.9	Max WS	-181168	-89.7	7.39		7.41	0.000008	-1.27	143146.8	1867.23	0.03
Lower	146100.7	Max WS	-266097	-110.6	7.25		7.3	0.000002	-1.85	143948.7	1481	0.03
Lower	142654.1	Max WS	-343625	-77.2	7.13		7.2	0.000004	-2.11	162889.8	2719.26	0.05
Lower	138897.5	Max WS	-412829	-116.7	6.98		7.11	0.000005	-2.84	145136.9	1740.54	0.05
Lower	135386	Max WS	-472089	-110	6.9		7.05	0.000006	-3.08	153035.5	1936.26	0.06
Lower	131782.5	Max WS	-520933	-110.6	6.86		7.01	0.000006	-3.11	167470.7	2120.02	0.06

Reach	River Station	Profile	Flow Total	Minimum Channel Elevation	Water Surface Elevation*	Critical Water Surface	Energy Grade. Elevation*	Energy Grade Slope	Velocity Channel	Flow Area	Top Width	Froude # Channel
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Lower	129680.2	Max WS	-521054	-131.5	6.87		6.97	0.000005	-2.47	211246.8	3173.55	0.05
Lower	126279.4	Max WS	-559486	-113.8	6.8		6.98	0.000006	-3.39	165019.5	1686.54	0.06
Lower	124418.2	Max WS	-559622	-124.9	6.81		6.94	0.000008	-2.93	190909.8	3069.34	0.07
Lower	121686.4	Max WS	-588161	-90.6	6.79		6.97	0.00001	-3.42	172021.5	2656.96	0.07
Lower	117165.6	Max WS	-589342	-77.8	6.89		6.98	0.00001	-2.41	244058.3	6627.45	0.07
Lower	113298.7	Max WS	-614548	-67.9	6.88		6.99	0.00001	-2.75	223442.9	4949.77	0.07
Lower	110904.6	Max WS	-616266	-77.2	6.86		7	0.000011	-2.97	207454.2	4177.1	0.07
Lower	109695.3	Max WS	-617030	-57.3	6.87		7.01	0.000014	-3.01	204871.3	4893.32	0.08
Lower	105945.1	Max WS	-618940	-51.2	6.85		7.02	0.000014	-3.35	184923.9	3717.25	0.08
Lower	102156.2	Max WS	-619985	-67.9	6.86		7.03	0.000012	-3.35	185283.2	3471.81	0.08
Lower	99374.4	Max WS	-620352	-71.2	6.86		7.03	0.000012	-3.34	185502.8	3496.37	0.08
Lower	97997	Max WS	-620427	-83	6.84		7.03	0.000012	-3.52	176289.3	2972.51	0.08
Lower	94687.2	Max WS	-650372	-68.6	6.95		7.05	0.000011	-2.61	250358.3	6823.71	0.07
Lower	91859	Max WS	-654413	-45.1	6.95		7.07	0.000019	-2.77	248623	11445.61	0.09
Lower	88511.5	Max WS	-659899	-41.7	7		7.09	0.000017	-2.4	288382.9	13563.39	0.08
Lower	84840.1	Max WS	-664801	-38.4	7.03		7.1	0.000014	-2.09	321683.9	14564	0.07
Lower	80885.8	Max WS	-668229	-37.7	7.03		7.09	0.000012	-1.9	352351.3	14931.16	0.07

Reach	River Station	Profile	Flow Total	Minimum Channel Elevation	Water Surface Elevation*	Critical Water Surface	Energy Grade. Elevation*	Energy Grade Slope	Velocity Channel	Flow Area	Top Width	Froude # Channel
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Lower	77548.5	Max WS	-669535	-35.1	7.02		7.07	0.000012	-1.76	381302.4	18655.87	0.07
Lower	73946.2	Max WS	-758633	-35.1	7.01		7.07	0.000014	-1.95	389124.1	18188.32	0.07
Lower	69814.5	Max WS	-770918	-41.7	6.93		7.04	0.000018	-2.62	302083.3	13747.34	0.09
Lower	65769.6	Max WS	-778247	-44.9	6.88		6.99	0.000017	-2.58	301466.8	13697.46	0.1
Lower	63761.2	Max WS	-780170	-44.9	6.87		6.96	0.000021	-2.45	318881.3	14145.03	0.09
Lower	59968.6	Max WS	-903933	-41.7	6.82		6.96	0.000028	-2.96	305875.1	12909.08	0.11
Lower	56614.9	Max WS	-907974	-38.4	6.8		6.94	0.000025	-3	302387.4	11478.74	0.1
Lower	52958.7	Max WS	-1035033	-38.9	6.76		6.96	0.000038	-3.63	284955.8	10986.43	0.13
Lower	48988.7	Max WS	-1156166	-45	6.77		7.03	0.000047	-4.11	281381.7	10667.81	0.14
Lower	45278.9	Max WS	-1266357	-45	6.86		7.15	0.000052	-4.31	294005.3	11240.42	0.15
Lower	41284	Max WS	-1364954	-41.7	7		7.31	0.00006	-4.42	308643.7	12563.7	0.16
Lower	37430.8	Max WS	-1368908	-38.4	7.14		7.45	0.000062	-4.52	303343.3	12267.65	0.16
Lower	34096.5	Max WS	-1461649	-41.7	7.28		7.65	0.00007	-4.93	296342.5	11494.91	0.17
Lower	30265	Max WS	-1548240	-45	7.49		7.87	0.000077	-4.96	311887.7	12818.36	0.18
Lower	26618.8	Max WS	-1558868	-44.9	7.74		8.07	0.00007	-4.61	338463.2	14537.41	0.17
Lower	22684.4	Max WS	-1564076	-47.2	7.91		8.27	0.000074	-4.79	326295.9	13814.13	0.17
Lower	18693.6	Max WS	-1671704	-48.2	8.13		8.54	0.00008	-5.14	325115.1	13170.86	0.18

Reach	River Station	Profile	Flow Total	Minimum Channel Elevation	Water Surface Elevation*	Critical Water Surface	Energy Grade. Elevation*	Energy Grade Slope	Velocity Channel	Flow Area	Top Width	Froude # Channel
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Lower	14875.8	Max WS	-1679934	-54.4	8.18		8.8	0.000111	-6.32	265633.3	11344.1	0.23
Lower	11282.2	Max WS	-1793479	-58.1	8.38		9.23	0.000149	-7.4	242521.8	9032.28	0.25
Lower	7496	Max WS	-1799968	-54.8	8.62		9.62	0.000127	-8.03	224195.1	6966.41	0.25
Lower	3965.9	Max WS	-1900199	-54.8	8.8		10.13	0.000157	-9.26	205229.6	5722.51	0.27
Lower	0	Max WS	-1972926	-54.8	9.2		10.75	0.000165	-9.97	197908.4	5087.15	0.28
Lower	-4740.*	Max WS	-1978718	-52.67	9.96		11.44	0.000157	-9.78	202450	5159.56	0.27
Lower	-9480.*	Max WS	-2027113	-50.55	10.73		12.21	0.000154	-9.78	207260.1	5212.41	0.27
Lower	-14220.*	Max WS	-2034567	-48.42	11.47		12.9	0.000146	-9.6	212055.3	5277.25	0.27
Lower	-18960	Max WS	-2073356	-46.3	12.14		13.56	0.00014	-9.57	216751.2	5243.34	0.26
Lower	-24127.*	Max WS	-2092113	-46.78	12.88		14.27	0.00014	-9.46	221185.5	5435.74	0.26
Lower	-29295.*	Max WS	-2105780	-47.25	13.56		14.92	0.000135	-9.35	225306.9	5486.63	0.26
Lower	-34462.*	Max WS	-2113347	-47.72	14.17		15.49	0.000129	-9.25	228590.3	5487.33	0.25
Lower	-39630	Max WS	-2202140	-48.2	14.74		16.15	0.000135	-9.53	231176.1	5491.68	0.26
Lower	-44780.*	Max WS	-2222565	-49.83	15.33		16.7	0.000127	-9.4	236530.2	5505.34	0.25
Lower	-49930.*	Max WS	-2235298	-51.45	15.84		17.18	0.00012	-9.28	241192.7	5514.3	0.25
Lower	-55080.*	Max WS	-2239811	-53.08	16.28		17.58	0.000114	-9.16	245199.5	5526.24	0.24
Lower	-60230	Max WS	-2384242	-54.7	16.77		18.2	0.000123	-9.6	249988.3	5796.67	0.25

Reach	River Station	Profile	Flow Total	Minimum Channel Elevation	Water Surface Elevation*	Critical Water Surface	Energy Grade. Elevation*	Energy Grade Slope	Velocity Channel	Flow Area	Top Width	Froude # Channel
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Lower	-65485.*	Max WS	-2400371	-55.95	17.08		18.64	0.000127	-10.02	240547.5	5181.96	0.26
Lower	-70740.*	Max WS	-2406621	-57.2	17.36		19.05	0.00013	-10.45	231219.8	4779.48	0.26
Lower	-75995.*	Max WS	-2572018	-58.45	17.71		19.81	0.000151	-11.65	221831.8	4380.5	0.28
Lower	-81250	Max WS	-2582338	-59.7	18.04		20.37	0.000156	-12.24	212021.8	4060.77	0.29
Lower	-86507.*	Max WS	-2723378	-60.08	18.93		21.18	0.00015	-12.05	228406.1	4546.17	0.28
Lower	-91765.*	Max WS	-2734911	-60.45	19.76		21.74	0.00013	-11.3	246089.9	5031.85	0.27
Lower	-97022.*	Max WS	-2857299	-60.83	20.45		22.34	0.000123	-11.04	264904.2	5711.33	0.26
Lower	-102280	Max WS	-2887344	-61.2	21.09		22.78	0.000108	-10.44	288928.7	8027.53	0.24
Lower	-106636*	Max WS	-2913390	-61.2	22.2		22.92	0.000052	-6.83	437910.4	9721.9	0.17
Lower	-110992*	Max WS	-2941139	-61.2	22.56		22.98	0.000033	-5.22	575147.4	13062.44	0.13
Lower	-115348*	Max WS	-2977086	-61.2	22.7		22.99	0.000024	-4.33	699503.2	16277.22	0.11
Lower	-119704*	Max WS	-3023009	-61.2	22.74		22.96	0.00002	-3.78	811240.9	19456.03	0.1
Lower	-124060	Max WS	-3075728	-61.2	22.71		22.89	0.000017	-3.42	910028.5	22630.38	0.09
Lower	-129310*	Max WS	-3121718	-64.47	22.51		22.72	0.000016	-3.61	874643.8	19075.42	0.09
Lower	-134560*	Max WS	-4074711	-67.75	22.32		22.73	0.000028	-5.15	798568.1	15367.2	0.12
Lower	-139810*	Max WS	-4445190	-71.03	22.2		22.87	0.000039	-6.59	683343.5	11793.19	0.15
Lower	-145060	Max WS	-4822219	-74.3	21.86		23.23	0.000068	-9.39	524185.9	8434.9	0.2

Reach	River Station	Profile	Flow Total	Minimum Channel Elevation	Water Surface Elevation*	Critical Water Surface	Energy Grade. Elevation*	Energy Grade Slope	Velocity Channel	Flow Area	Top Width	Froude # Channel
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Lower	-150047*	Max WS	-5349690	-68.03	22.82		23.56	0.000044	-6.9	788184.2	14140	0.16
Lower	-155035*	Max WS	-1.1E+07	-61.75	23.67		25.63	0.000138	-11.26	995073.1	21139.76	0.27
Lower	-160022*	Max WS	-1.2E+07	-55.48	23.39		25.26	0.00016	-10.97	1133177	29886.46	0.28
Lower	-165010	Max WS	-1.7E+07	-49.2	23.2	5.48	26.45	0.000354	-14.62	1381348	48305.79	0.41

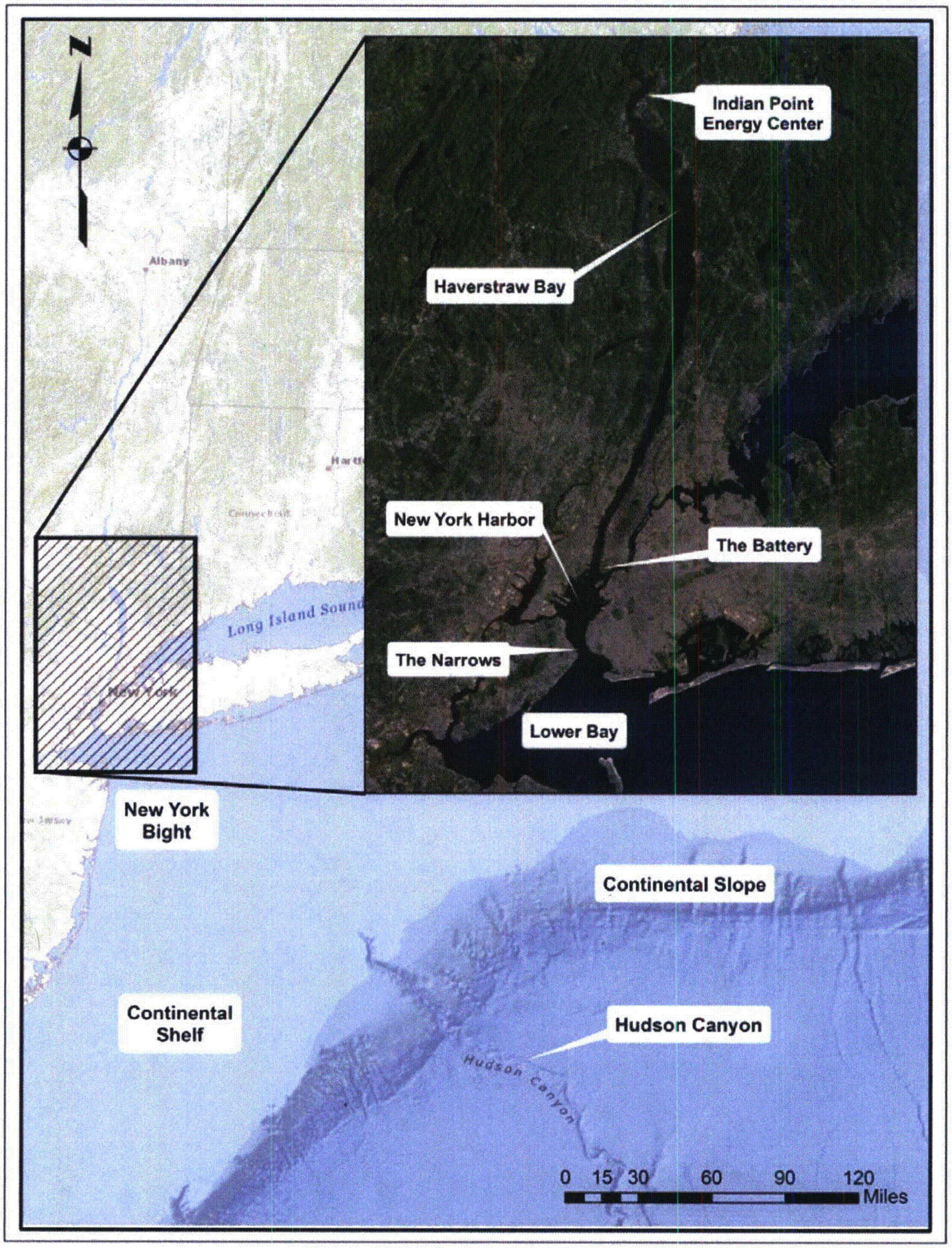

Figure 3.6-1: Location of IPEC relative to Hudson River features and the continental slope

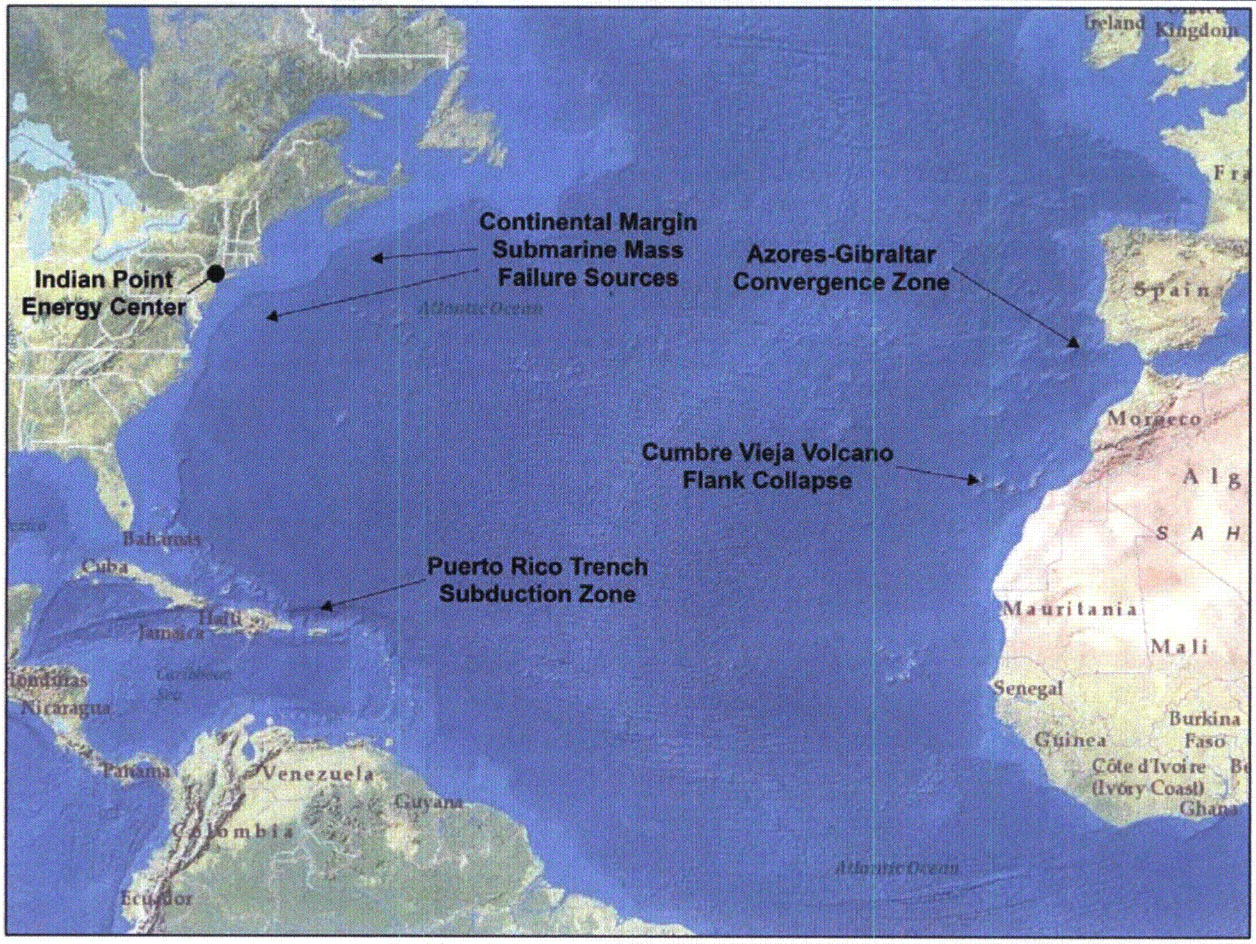

Figure 3.6-2: Potential tsunami source locations for the U.S. East coast in the North Atlantic Ocean basin

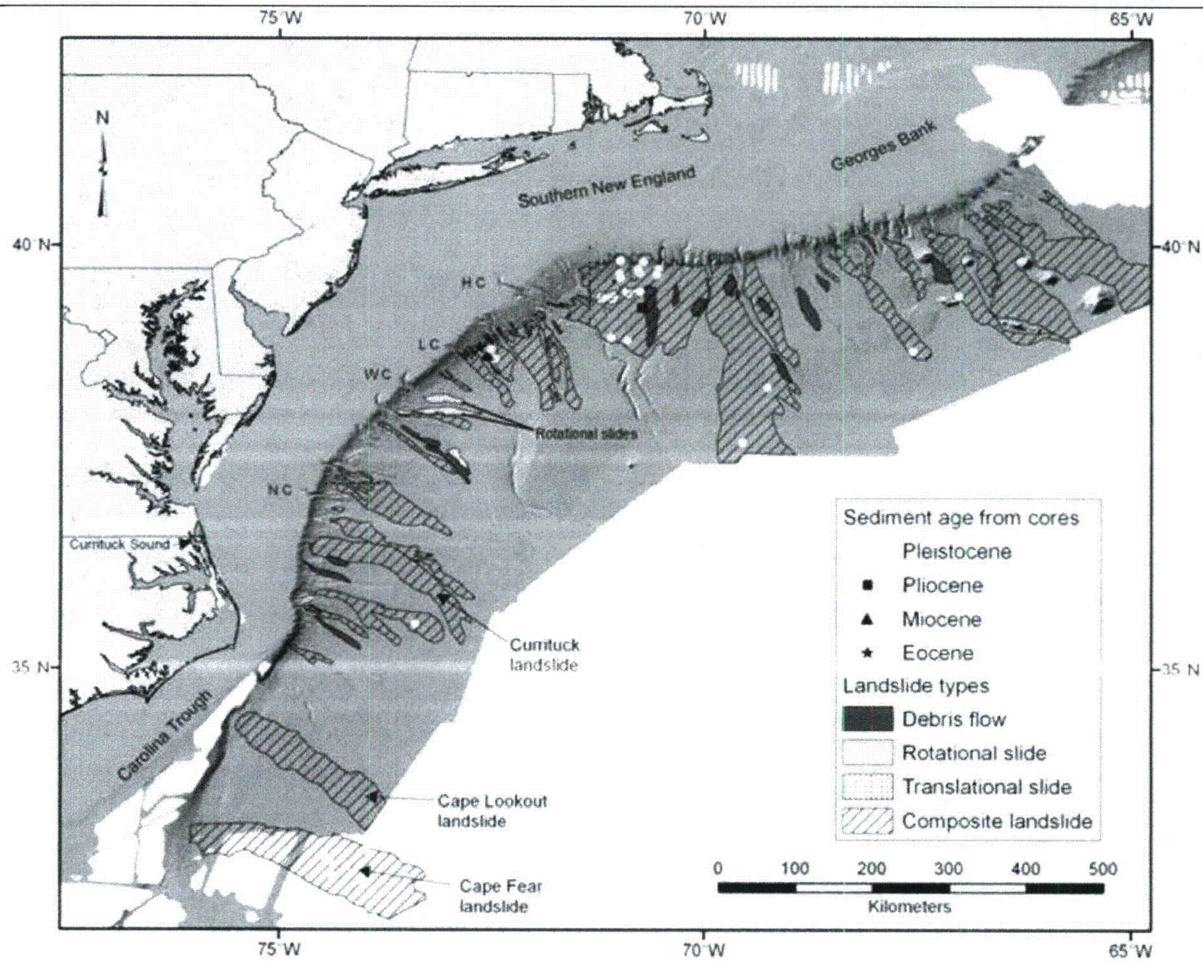

Figure 3.6-3: Shaded relief map of the East Coast of the U.S. with interpretation showing the 48 landslide areas mapped by AGMTHAG (AGMTHAG, 2008)

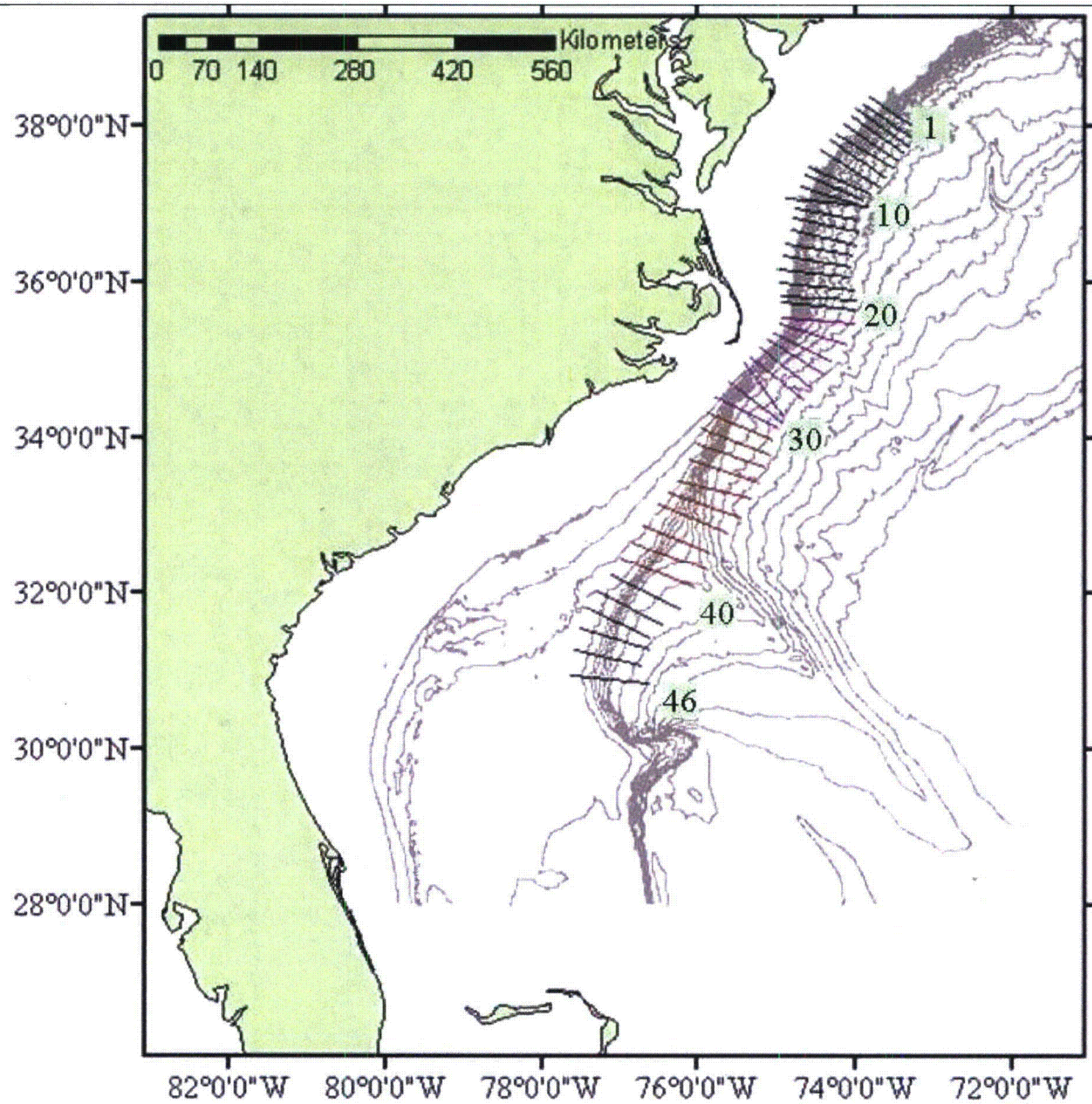

Figure 3.6-4: The additional 46 shelf transects used in Krause (Krause, 2011) each used for 15,000 stochastic stability analyses of submerged slopes

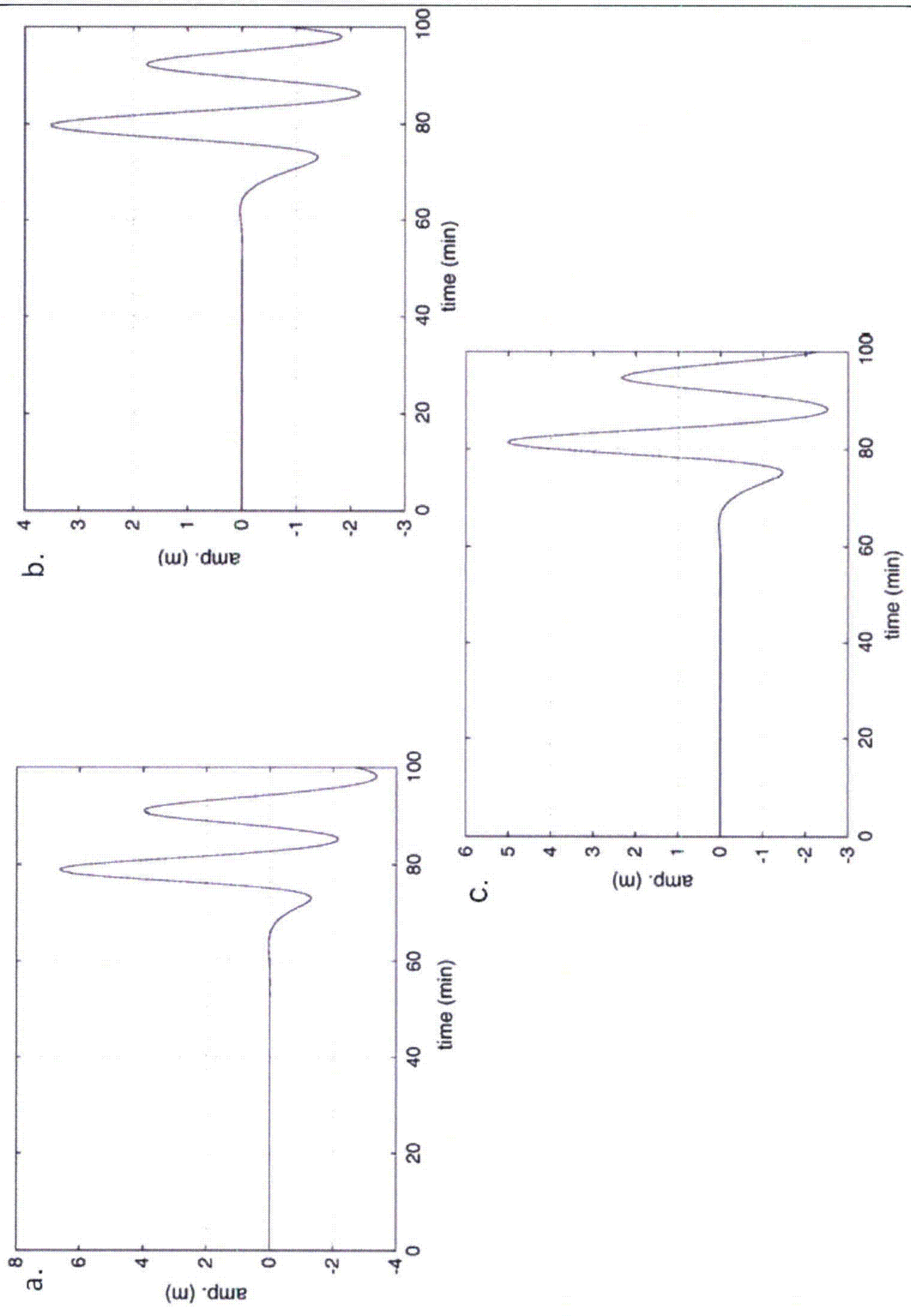

Figure 3.6-5: Time series of tsunami amplitude at a nearshore location broadside of the Currituck Landslide for the three different slide scenarios.

(a) volume =108 km3 ; (b) volume =57 km3 ; (c) volume =128-168 km3 (Geist et al, 2009)

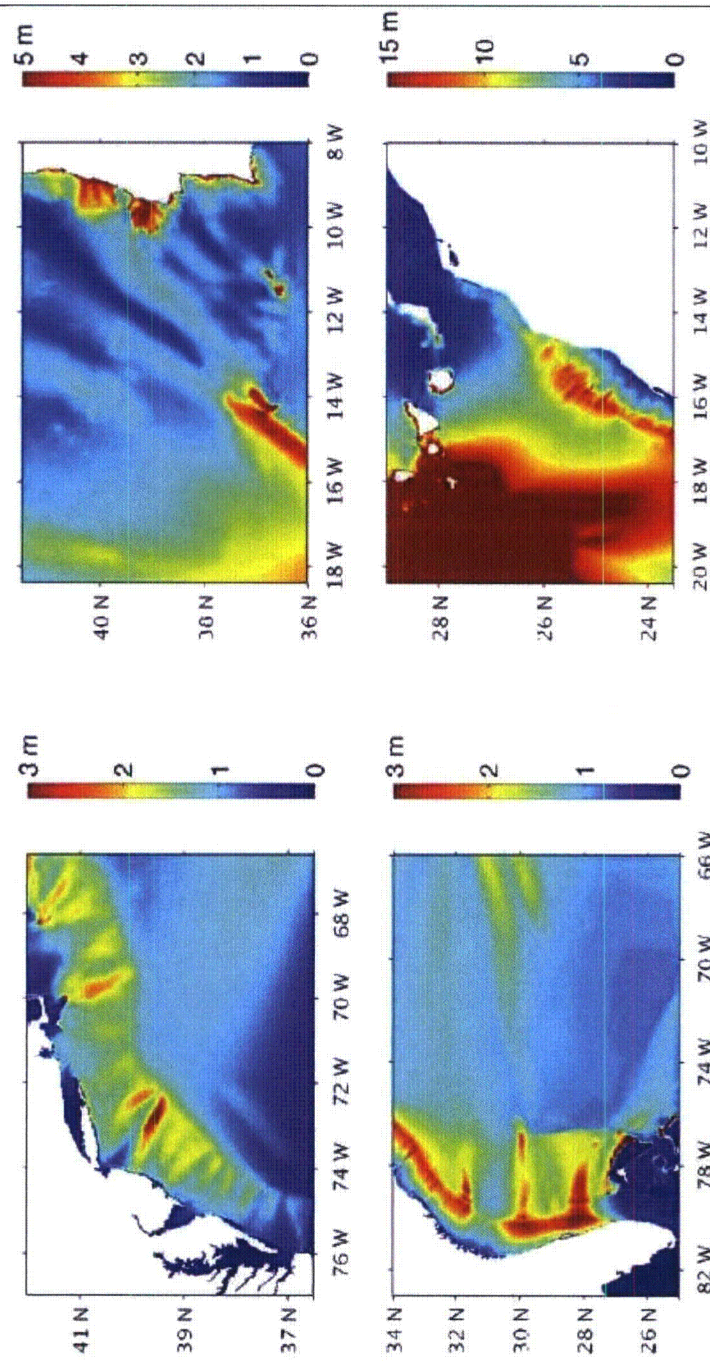

Figure 3.6-6: Maximum surface elevation computed with FUNWAVE in 30" regional grids for the 19 mi^3 CVV case (Harris et al, 2012).

The regional grids include N. North America (top left), S. North America (bottom left), Western Europe (top right), and Western Africa (bottom right)

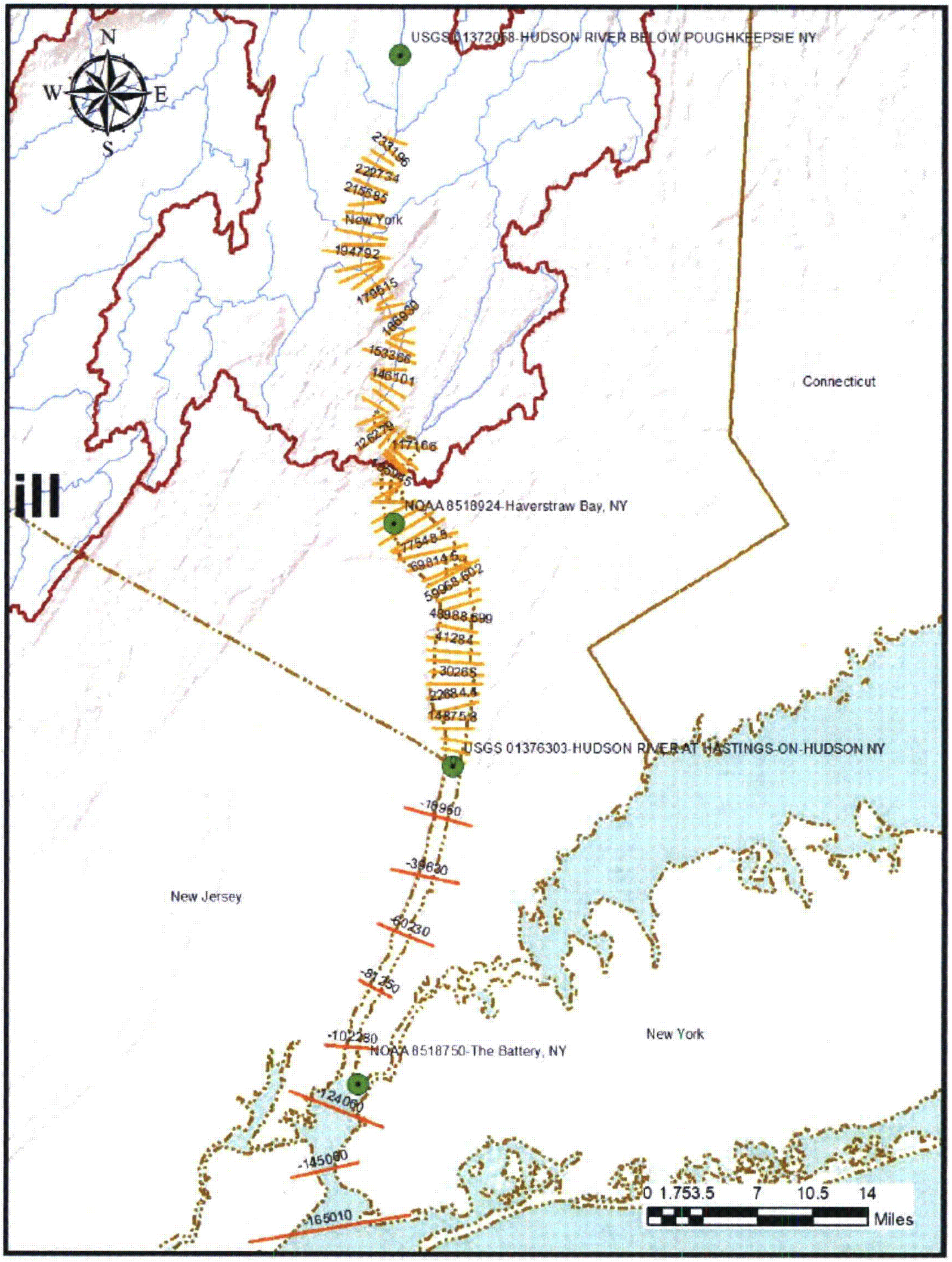

Figure 3.6-7: HEC-RAS Cross Sections used in the Hudson

3.7 Ice-Induced Flooding

This section addresses the effects of ice-induced flooding on the water surface elevation at Indian Point Energy Center (IPEC).

The Hudson River forms the western boundary of the site. IPEC is located on the east bank of the Hudson River in Buchanan, New York, approximately 2.2 miles southwest of Peekskill, New York, and 40 miles north of New York City

3.7.1 Method

The hierarchical hazard assessment (HAA) approach described in NUREG/CR-7046 (NRC, 2011) was used for the evaluation of the effects of ice-induced flooding on water surface elevation at IPEC. The HHA approach is a progressively refined, stepwise estimation of site-specific flood hazards, starting with the most conservative simplifying assumptions that maximize flood hazards. If the site is not inundated by the flood mechanism evaluated, a conclusion that the SSCs are not susceptible to flooding is valid and no further analyses were completed (NRC, 2011). Thus, the flood elevations herein are conservative and intended to demonstrate the safety of SSCs against ice-induced flooding, and not intended to represent the most realistic estimation of flood elevations due to ice-induced flooding.

The HHA approach is consistent with the following standards and guidance documents:

1. NRC Standard Review Plan, NUREG-0800, revised March 2007;
2. NRC Office of Standards Development, Regulatory Guides:
    a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976;
    b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977; and
3. American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8 - 1992)

The criteria for ice-induced flooding is provided in NUREG/CR-7046, Appendix D (NRC, 2011). Two ice-induced events may lead to flooding at the site and are recommended and discussed in NUREG/CR-7046, Appendix D including:

a. Ice jams or dams that form upstream of a site that collapse, causing a flood wave; and

b. Ice jams or dams that form downstream of a site that result in backwater flooding.

With respect to ice-induced flooding on the Hudson River, the HHA used the following steps:

1. Identify largest historic ice-induced flooding event and calculate water depth

2. Conservatively calculate peak water surface elevation resulting from failure of upstream ice jam

3. Conservatively calculate peak water surface elevation from backwater effects resulting from a downstream ice jam

3.7.2 Results

3.7.2.1 Ice-Induced Flooding (AREVA, 2013a)

3.7.2.1.1 Identify largest historic ice-induced flooding event and calculate water depth

Records of historic ice jam flood stages were downloaded from the U.S. Army Corps of Engineers (USACE) ice jam database (USACE, 2013). The largest historic ice-induced flooding event occurred March 23, 1948 at Hadley, NY, 180 miles upstream of IPEC. The resultant calculated flood stage water depth behind the ice jam was 11.35 feet.

3.7.2.1.2 Calculate peak water surface elevation resulting from failure of upstream ice jam

The peak water surface elevation at IPEC was calculated for an ice jam forming and breaching at Bear Mountain Bridge, the first bridge upstream of IPEC. The peak flood wave height was conservatively kept constant (i.e., did not allow for attenuation as the flood wave traveled 4.32 river miles southerly or downstream to IPEC). The resultant peak water surface elevation at IPEC was considered to be equal to the mean tide elevation at IPEC combined with the water depth of the peak flood wave from the upstream ice jam failure.

The ice jam at Bear Mountain Bridge was conservatively considered to be equivalent to the largest historic ice jam recorded at Hadley, New York (11.35 feet high). This is conservative because there are no historic ice jams south of Albany, NY, which is approximately 100 miles upstream of IPEC.

The mean tide elevation was selected as the reference / antecedent water surface elevation because under high tide conditions the Hudson River flows south to north and would convey ice northward.

Mean tide elevation was conservatively selected over mean low tide elevations. The mean tide elevation at IPEC was approximately 1.13 feet. The peak water surface elevation at IPEC resulting from the upstream ice jam breach was therefore calculated to be 11.35 feet plus 1.13 feet which equals 12.48 feet, or 2.77 feet below the maximum allowable surface water elevation at IPEC. (IPEC, 2010 and IPEC, 2011)

3.7.2.1.1 Estimate peak water surface elevation from backwater effects resulting from a downstream ice jam

Peak water elevation was estimated for an ice jam forming at Tappan Zee Bridge, the first bridge downstream of IPEC (about 15.6 river miles south or downstream of IPEC). Peak flood height at IPEC was considered to be equal to the top elevation of the downstream ice jam. The resultant peak water surface elevation at IPEC was equal to the mean tide elevation at IPEC (described above) combined with the water depth of the backwater from the ice jam.

Similar to Section 3.7.2.1.2, the ice jam at Bear Mountain Bridge was conservatively considered to be equivalent to the largest historic ice jam recorded at Hadley, New York (11.35 feet high).

The peak water surface elevation at IPEC resulting from backwater as the result of a downstream ice jam was 12.48 feet, or 2.77 feet below the maximum allowable surface water elevation at IPEC. (IPEC, 2010 and IPEC, 2011).

3.7.3 Conclusions

The peak water surface elevation at IPEC, resulting from the upstream ice jam / ice dam breach was conservatively calculated to be 12.48 feet, 2.77 feet below site grade. This was bounded by the PMF peak water surface elevation of the Hudson River at IPEC of 14.6 feet (AREVA, 2013b).

The peak water surface elevation at IPEC as a result from backwater caused by the ice jam was calculated to be 12.48 feet, 2.77 feet below site grade. This was bounded by the PMF peak water surface elevation on the Hudson River at IPEC of 14.6 feet (AREVA, 2013b).

These results presented above are considered to be conservative in that they are based on an historic ice jam which occurred far to the north and it is unlikely that a larger ice jam would occur on the tidal portion of the Lower Hudson River. The potential for any ice jam in the vicinity of IPEC is judged to be remote because the closest reported ice jam occurred in Albany approximately 100 miles upstream in 1857.

Ice-induced flooding is not specifically included as a mechanism to be combined with other extreme events as per NURGE/CR-7046 (NRC, 2011).

3.7.4 References

AREVA, 2013a. "Ice-Induced Flooding", AREVA Document No. 32-9196322-000, 2013.

AREVA, 2013b. AREVA Document No. 32-9196316-000, "Probable Maximum Flood on Hudson River – Hydraulics", January, 2013.

IPEC, 2010. "Indian Point 2 Updated Final Safety Analysis Report Revision 22", Entergy, 2010, See AREVA Document No. 38-9193643-000.

IPEC, 2011. "Indian Point 3 Updated Final Safety Analysis Report Revision 04", Entergy, 2011, See AREVA Document No. 38-9193643-000.

NRC, 2011. "Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046", U.S. Nuclear Regulatory Commission, November 2011. (ADAMS Accession No. ML11321A195)

USACE, 2013. U.S. Army Corps of Engineers (USACE), Ice Engineering Research Group, Cold Regions Research and Engineering Laboratory, Website: http:icejams.crrel.usace.army.mil/, accessed January 2013, See AREVA Document No. 32-9196322-000.

3.8 Channel Migration or Diversion

Natural river channels may migrate or divert either away from or toward the site. The relevant event for flooding is diversion of water towards the site. There are no well-established predictive models for channel diversions. Therefore, it is not possible to postulate a probable maximum channel diversion event. Instead, historical records and hydrogeomorphological data should be used to determine whether an adjacent channel, stream, or river has exhibited the tendency to meander towards the site. (NRC, 2011).

3.8.1 Method

The channel migration and diversion flooding evaluation followed the HHA approach described in NUREGCR-7046, Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America (NRC, 2011). The HHA approach used is consistent with the following standards and guidance documents:

1. NRC Standard Review Plan, NUREG-0800, revised March 2007;
2. NRC Office of Standards Development, Regulatory Guides:
    a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976;
    b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977;
3. NUREG/CR-7046 – Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, dated November 2011; and

With respect to channel migration and diversion, the HHA used the following two steps:

1. Review historical records and hydrogeologic data to assess whether the Hudson River exhibits the tendency to migrate towards the site.
2. Evaluate present-day channel maintenance measures in place to mitigate channel migration of the Hudson River.

3.8.2 Results (AREVA, 2013)

3.8.2.1 Review of Historical Records

A literature review did not yield evidence suggesting there have been significant historical diversions of the Hudson River near Indian Point Energy Center (IPEC) over the last century. A comparison of 1892 (UNH, 2012) and 1981 (NY GIS, 2012b) USGS Topographic maps illustrates continuity of the river course over the last 120 years.

The Lower Hudson Valley has been subjected to repeated glacial advance and retreat, creating typical glacial morphology of main and tributary valleys and bedrock ledges (Sirkin, 2006). The glaciers have controlled the deposition of unconsolidated deposits in the region, although these are absent locally due to erosion and excavation (GZA, 2008). Glacial till lies directly on the bedrock surface and is generally less than 10 feet thick, but is locally thicker against steep north-facing bedrock slopes (GZA,

2008). The Lower Hudson River Valley exhibits characteristics of a fjord within the high rock walls of the highlands (Sirkin, 2006).

The site slopes downward relatively steeply towards the Hudson River and is characterized by ground surface elevations ranging from approximately 15 to 60 feet MSL. Much of the critical shoreline at the IPEC site is composed of vertical steel sheet pile, but some portions of the southern shore are relatively steep, riprap covered slopes. As illustrated on Figure 6.18 from the Hydrogeologic Site Investigation (GZA, 2008), the majority of the critical structures are founded on bedrock; however, several structures including the intake structures are located along the shoreline in the overburden material. This section of shoreline is protected from the river with the vertical steel sheet pile walls. It is therefore unlikely that the shoreline would erode and the Hudson River divert through the site.

The steep slopes and glacial lake clay soil deposits of the land surrounding the Hudson River valley near IPEC offer the potential for landslides. The area is considered to be an area of high landslide incidence (NYSOEM, 2011). The Hudson River is approximately 3,500 feet wide about 2.5 miles upstream of IPEC. The maximum river depth in this area is approximately 70 feet (NY GIS, 2012a). While a landslide near IPEC is possible, a large landslide involving the substantial volume of material needed to cutoff and divert the Hudson River is unlikely. No evidence of landslide induced channel diversion near IPEC was found during the literature review.

3.8.2.2 Evaluation of Present Day Channel Maintenance

The U.S. Army Corps of Engineers, New York District performs maintenance dredging of the Hudson River federal navigation project, as necessary to maintain the authorized project dimensions, thereby assuring safe and economical use of the Hudson River by shipping interests. The navigational channel near IPEC is 600 feet wide, (from New York City to Kingston), with depths of 32 feet in soft material and 34 feet in rock (USACE, 2012).

3.8.3 Conclusions

A review of historical data indicates that the Hudson River has not exhibited a tendency to meander towards the site. The Hudson River is a maintained, navigable waterway near IPEC and the U.S. Army Corps of Engineers, New York District is responsible for maintaining navigable conditions through periodic dredging. Much of the critical shoreline at the IPEC site is composed of vertical steel sheet pile wall, but some portions of the southern shore are relatively steep, riprap covered slopes. As per NUREG / CR-7046, stream channels that are steeply incised, have limited flood plains, and are located in geologic formations relatively resistant to erosion would not be expected to be susceptible to channel diversion. Given these conditions, channel migration is not considered to be a potential contributor to flooding at IPEC.

3.8.4 References

AREVA, 2013. "Indian Point Energy Center Flood Hazard Re-evaluation – Channel Migration or Diversion", AREVA Document No. 51-9196309-000, 2013.

GZA, 2008. "Hydrogeologic Site Investigation", GZA GeoEnvironmental, Inc., January 2008.

NRC, 2011. "Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046", U.S. Nuclear Regulatory Commission, November 2011.

NY GIS, 2012a. Hudson River Estuary Bathymetry Polyline Contours, New York State GIS Clearinghouse, http://gis.ny.gov/gisdata/inventories/details.cfm?DSID=1136, accessed 2012, see AREVA Document No. 51-9196309-000.

NY GIS, 2012b. "1981 Peekskill Quadrangle, 1:24,000 Quadrangle, U.S. Geologic Survey, New York State GIS Clearinghouse, http://gis.ny.gov/gisdata/quads/drg24/index_quadname.htm#W, accessed 2012, see AREVA Document No. 51-9196309-000.

NYSOEM, 2011. "2011 New York State Standard Multi-Hazard Mitigation Plan – Chapter 3.13 Landslide hazard Profile", New York State Office of Emergency Management, 2011.

Sirkin, 2006. "The Hudson River Estuary – Chapter 2 – The Hudson River Valley Geological History, Landforms, and Resources", 2006.

UNH, 2012. "Historic USGS Maps of New England & NY", 1892 West Point, NY Quadrangle, http://docs.unh.edu/nhtopos/nhtopos.htm, accessed 2012, see AREVA Document No. 51-9196309-000.

USACE, 2012. "Hudson River, New York North Germantown Reach Federal Navigation Project Maintenance Dredging" – New York District Public Notice, U.S. Army Corps of Engineers, March 13, 2012.

3.9 Combined-Effect Flood

This section addresses the potential for flooding at Indian Point Energy Center (IPEC) due to combined effect flood events.

3.9.1 Method

The hierarchical hazard assessment (HHA) approach described in NUREG/CR-7046 (NRC, 2011) was used for the evaluation of the effects of combined flood events on water surface elevation at IPEC. The HHA approach is a progressively refined, stepwise estimation of site-specific flood hazards, starting with the most conservative simplifying assumptions that maximize flood hazards. If the site is not inundated by the flood mechanism evaluated, a conclusion that the SSCs are not susceptible to flooding is valid and no further analyses were completed (NRC, 2011). Thus, the flood elevations herein are conservative and intended to demonstrate the safety of SSCs against combined events flooding, and not intended to represent the most realistic estimation of flood elevations due to combined events flooding.

1. The HHA approach is consistent with the following standards and guidance documents:
2. NRC Standard Review Plan, NUREG-0800, revised March 2007;
3. NRC Office of Standards Development, Regulatory Guides:
   a. RG 1.102 – Flood Protection for Nuclear Power Plants, Revision 1, dated September 1976;
   b. RG 1.59 – Design Basis Floods for Nuclear Power Plants, Revision 2, dated August 1977; and
4. American National Standard for Determining Design Basis Flooding at Power Reactor Sites (ANSI/ANS 2.8 - 1992).

The criteria for combined events floods are provided in NUREG/CR-7046, Appendix H and consist of considering the following scenarios:

1. <u>Floods Caused by Precipitation Events</u> (NUREG/CR-7046, Appendix H, Section H.1)
2. <u>Floods Caused by Seismic Dam Failures</u>(NUREG/CR-7046, Appendix H, Section H.2)
3. <u>Floods along the Shores of Open and Semi-Enclosed Bodies of Water</u>(NUREG/CR-7046, Appendix H, Section H.3)
4. <u>Floods Caused by Tsunamis</u> (NUREG/CR-7046, Appendix H, Section H.5)

   Tsunamis have been screened out as a potential flood mechanism at IPEC and therefore are not subject to combined event flood analysis (AREVA, 2013c).

The combined event evaluation for IPEC was conducted for the above applicable scenarios as described in AREVA, 2013a for Floods Caused By Precipitation Events and Seismic Dam Failures and

Section 3.4.2.5 and AREVA 2013d for Floods Along the Shores of Open and Semi-Enclosed Bodies of Water and used the following steps:

- Calculate the wind wave effects and wave runup on the Hudson River at IPEC.
- Calculate the Probable Maximum Water Elevation on the Hudson River at IPEC resulting from the combined-effect flood.

3.9.2 Results

The results of the combined effect flood evaluation are summarized in Table 3.9-1. Floods along the Shores of Open and Semi-Enclosed Bodies of Water (due to coastal storm surge) resulted in the highest calculated combined flood elevation. The potential combined effect scenarios are discussed below.

Floods Caused by Precipitation Events (NUREG/CR-7046, Appendix H, Section H.1)

Three alternatives for Floods Caused by Precipitation Events were evaluated in AREVA, 2013a. The highest flood level results from the peak water surface elevation on the Hudson River at IPEC due to the Probable Maximum Flood (PMF) including hydrologic upstream dam failure, as described in AREVA,2013b. The bounding stillwater elevation for flood events is 14.9 feet NGVD29. This scenario, with the inclusions of wave runup induced by 2-year wind speeds, is described as Alternative 3 of Section H.1 of Appendix H of NUREG/CR-7046.

Floods Caused by Seismic Dam Failures(NUREG/CR-7046, Appendix H, Section H.2)

The results of the dam failure calculation, (AREVA,2013b),indicate that flood elevations from seismic dam failures (NUREG/CR-7046, Appendix H.2) resulted in a peak stillwater elevation of 13.5 feet at IPEC and is bounded by PMF with coincident hydrologic dam failure on the Hudson River that resulted in a peak stillwater elevation of 14.9 feet at IPEC. No overtopping of the shoreline bulkhead will occur. Therefore, further calculations to address NUREG/CR-7046 Appendix H.2 are not necessary.

Floods along the Shores of Open and Semi-Enclosed Bodies of Water(NUREG/CR-7046, Appendix H, Section H.3)

As described in Section 3.4.2.5, Alternative 3 of Section H.3 of NUREG/CR-7046 (the combination of the 25-year flood in the stream, the probable maximum storm surge and seiche with wind-wave activity, and the antecedent 10 percent exceedance high tide) resulted in the highest flood elevation.

Probabilistic analysis of storm surge indicates that the storm surge flood elevation at IPEC associated with a $2x10^{-6}$ annual exceedance probability, including storm surge, tidal flow, projected sea level rise and steady-state river flow associated with a 25-year return period in the Hudson River, is elevation 14.8 feet NAVD88 (15.8 feet NGVD29). The wave crest elevation is calculated to be 16.3 feet. The limit of wave run-up elevation is calculated to be 17.7 feet.

3.9.3 Conclusions

At IPEC, the bounding condition for impacts to the site from combined event on the Hudson River combined with wind generated waves are summarized below.

The wind generated wave crest applied along the critical direction on the Hudson River at IPEC was calculated to be 0.5 feet (elevation 16.3 ft NGVD29); and wave runup was calculated to be 1.9 feet (elevation 17.7 ft NGVD29).

The bounding still water elevation for coastal flood events (the combination of the 25-year flood in the stream, the probable maximum storm surge and seiche with wind-wave activity, and the antecedent 10 percent exceedance high tide) is calculated to be 15.8 feet (AREVA, 2013d). This scenario, with the inclusions of wave runup induced by coincident wind speeds, is described as Alternative 3 of Section H.3 of Appendix H of NUREG/CR-7046.

The Maximum Water Elevation resulting from combined events on the Hudson River at IPEC was calculated as the sum of the bounding flood event (still water elevation 15.8 feet) and the coincident wind wave runup (0.5 feet). This Maximum Water Elevation resulting from combined coastal events (was calculated to be elevation 17.7 feet.

The results indicate that the combined effects flood maximum water elevation on the Hudson River at IPEC resulting from combined-effect flood is above the plant grade elevation of 15.0 feet (IPEC, 2010).

3.9.4 References

AREVA, 2013a. "Combined Events Flood Analysis – Riverine", AREVA Document No. 32-9196323-000, 2013.

AREVA, 2013b. "Dam Failures", AREVA Document No. 32-9196316-000, 2013.

AREVA, 2013c. "Tsunami Screening Evaluation", AREVA Document No. 51-9196310-000, 2013.

AREVA, 2013d. "Flood Hazard Re-evaluation – Combined Effect Floods – Coastal Processes for Indian Point Energy Center", AREVA Document No. 32-9193356-000, 2013.

IPEC, 2010. "Indian Point 2 Updated Final Safety Analysis Report Revision 22", Entergy, 2010, See AREVA Document No. 38-9193643-000.

NRC, 2011. "Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants - NUREG/CR-7046", U.S. Nuclear Regulatory Commission, November 2011, (ADAMS Accession No. ML11321A195).

Table 3.9-1: Combined Effect Flood Scenario Summary Results

Scenario	Stillwater (ft)	Wave (ft)(e)	Combined (ft)
Floods Caused by Precipitation Events(e)	14.9	1.7	16.6
Floods Caused by Seismic Dam Failures	13.5	(a)	(a)
Floods along the Shores of Open and Semi-Enclosed Bodies of Water	15.8(b)	0.5	16.3/17.7 (c)
Floods Caused by Tsunamis	(d)	(d)	(d)

Notes:

(a) Combined Event analysis not required. Dam Failure is included in Flood Caused by Precipitation Events

(b) Based on estimated annual exceedance probability $2x10^{-6}$ storm surge event

(c) Wave elevation/Limit of wave run-up

(d) Effects of tsunamis were screened out as a controlling flood mechanism at IPEC

(e) Waves for Precipitation Event are based on 2-year wind and are essentially deepwater waves for purposes of comparison with the waves calculated for the coastal surge which are based on hurricane winds.

4.0 COMPARISON OF CURRENT AND REEVALUATED FLOOD CAUSING MECHANISMS

4.1 Summary of Current Licensing Basis and Flood Reevaluation Results

This section compares the current and reevaluated flood-causing mechanisms. It provides a comparison of the CLB flood elevation to the reevaluated flood elevation for each applicable flood-causing mechanism. A comparison of the CLB elevations and the reevaluated flood elevations is provided in Table 4.1-1.

Screened mechanisms have been evaluated at a high level and determined to not be applicable to the flooding hazard for IPEC.

4.1.1 Local Intense Precipitation

4.1.1.1 Current Licensing Basis

The CLB does not include an evaluation of LIP. Localized PMP was evaluated as part of IPEEE for Unit 2 and Unit 3, utilizing storm parameters presented in HMR-51 and HMR-52.

The IPEEE evaluation for Unit 2 concluded that ponding could occur on the IPEC Unit 2 site, however, PMP related flood levels would not exceed the design allowable water buildup at safety-related SSC (IPEEE, 1995, IPEC, 2012a).

The IPEEE evaluation for Unit 3 concluded that there were no pathways in the vicinity of safety-related SSC that would allow precipitation runoff to penetrate any buildings (IPEEE,1997).

4.1.1.2 Reevaluated Flood Elevation

In the immediate vicinity of Unit 2, the maximum water surface elevations are up to elevation 19.0 feet in the transformer yard. The calculated flood elevation varies between 0 and 0.2 feet higher than building entrances abutting the Unit 2 Transformer Yard.

In the immediate vicinity Unit 3 the maximum water surface elevations are up to elevation 19.2 feet in the transformer yard. The calculated flood elevation does not exceed building entrances abutting the Unit 3 Transformer Yard.

Details of this evaluation are provided in Section 3.1.

4.1.1.3 Comparison

As the CLB does not include localized PMP effects, direct comparison is not possible. The new evaluation results in higher flood levels than the IPEEE evaluation, and results in impacts to safety-related SSC at Unit 2.

4.1.2 Flooding on Rivers and Streams

4.1.2.1 Current Licensing Basis

The PMF in the Hudson River with no tidal influences is defined in the Unit 2 FSAR as 1,100,000 cfs, resulting in a water surface elevation of 12.7 ft (IPEC, 2010).

4.1.2.2 Reevaluated Flood Elevation

The PMF in the Hudson River at IPEC after hydraulic routing within HEC-RAS is conservatively calculated at 1,185,300 cfs. Historical records do not indicate flooding in excess of this PMF flow.

The peak PMF water surface elevation at Indian Point Energy Center is 14.6 feet, which is 0.4 feet below the plant grade elevation of 15.0 feet (IPEC, 2010 and IPEC, 2011).

Based on the re-evaluated peak PMF elevation on the Hudson River at IPEC, the peak PMF water surface elevation from the Hudson River flood is below the plant grade elevation.

Details of this evaluation are provided in Section 3.2.

4.1.2.3 Comparison

The results of the new evaluation are 1.9 ft higher than the CLB for PMF on the Hudson River. The resulting PMF elevation remains below the site grade of 15.0 ft.

4.1.3 Dam Breaches and Failures

4.1.3.1 Current Licensing Basis

The CLB reports a failure of the Ashokan Dam coincident with a Standard Project Flood on the Hudson River to have a resulting flow rate of 705,000 cfs. The peak water elevation associated with this event at IPEC is 7.2 ft (IPEC, 2010).

4.1.3.2 Reevaluated Flood Elevation

The controlling dam failure scenario is the hydrologic event (PMF) with the forced failure of Conklingville Dam. Combination of the dam breach outflow and PMF at IPEC after hydraulic routing is calculated to be 1,217,700 cfs. The resultant peak water surface elevation from the combined dam breach peak outflow and PMF in the Hudson River at IPEC is 14.9 feet, which is 0.1 feet below the plant grade elevation of 15.0 feet.

Details of this evaluation are provided in Section 3.3.

4.1.3.3 Comparison

The new evaluation uses a different dam and a higher base river flow. The resulting flood elevation is 7.7 ft higher than the CLB. The resulting flood elevation is below the site grade of 15.0 ft.

4.1.4 Storm Surge

4.1.4.1 Current Licensing Basis

The CLB does not include a separate evaluation of flooding solely due to storm surge. The IPEC CLB of elevation 15 feet NGVD29 provides plant storm surge protection.

4.1.4.2 Reevaluated Flood Elevation

The reevaluated storm surge due to a hurricane with tidal flow and sea level rise is calculated to be 15.8 feet based on an annual exceedance probability of 2×10^{-6}.

4.1.4.3 Comparison

The reevaluated storm surge elevation exceeds the CLB by 2.3 ft (15.8 ft vs. 13.5 ft).

4.1.5 Seiche

4.1.5.1 Current Licensing Basis

Probable Maximum Seiche caused flooding was not evaluated as part of the CLB.

4.1.6 Tsunami

4.1.6.1 Current Licensing Basis

Flooding from tsunamis was not evaluated quantitatively because flood from a tsunami was not screened out by other flood mechanisms (IPEC, 2011, Section 1.3.1).

4.1.6.2 Reevaluated Flood Elevation

The Probable Maximum Tsunami is not considered to be a significant flood hazard for IPEC due to the inland location of the site, the necessary geometry of a tsunami wave to enter the Hudson River, and the predicted attenuation of a tsunami wave traversing the Hudson River.

Details of this evaluation are provided in Section 3.6.

4.1.6.3 Comparison

Tsunamis were not evaluated as part of the CLB, so no direct comparison is possible. Based on the new evaluation, the probable maximum tsunami event would not impact safety-related SSC at IPEC.

4.1.7 Ice-Induced Flooding

4.1.7.1 Current Licensing Basis

Ice-induced flooding was not evaluated as part of the CLB.

4.1.7.2 Reevaluated Flood Elevation

The peak water surface elevation at IPEC, resulting from the upstream ice jam / ice dam breach was conservatively calculated to be 12.48 feet, 2.77 feet below site grade. This was bounded by the PMF peak water surface elevation of the Hudson River at IPEC of 14.6 feet.

The peak water surface elevation at IPEC as a result from backwater caused by the ice jam was calculated to be 12.48 feet, 2.77 feet below site grade. This was bounded by the PMF peak water surface elevation on the Hudson River at IPEC of 14.6 feet.

Details of this evaluation are provided in Section 3.7.

4.1.7.3 Comparison

Ice-induced flooding was not evaluated as part of the CLB, so no direct comparison is possible. Based on the new evaluation, ice-induced flooding elevations are bound by the PMF on the Hudson River.

4.1.8 Channel Migration or Diversion

4.1.8.1 Current Licensing Basis

Migration of the Hudson River is not evaluated as part of the CLB.

4.1.8.2 Reevaluated Flood Elevation

A review of historical data indicates that the Hudson River has not exhibited a tendency to meander towards the site. The Hudson River is a maintained, navigable waterway near IPEC and the U.S. Army Corps of Engineers, New York District is responsible for maintaining navigable conditions through periodic dredging.

Details of this evaluation are provided in Section 3.8.

4.1.8.3 Comparison

Channel Migration was not evaluated as part of the CLB, so no direct comparison is possible. Due to the geology of the watershed and man-made controls on the Hudson River, channel migration is not considered a hazard for IPEC.

4.1.9 Combined Effect Flooding

4.1.9.1 Current Licensing Basis

The controlling CLB flood level is the dam failure water level coincident with a probable maximum hurricane, resulting in a stillwater elevation of 14.0 ft. Wave runup was determined to be 1.0 ft, resulting in a maximum water surface elevation of 15.0 ft (IPEC, 2010).

The NRC performed an independent evaluation of combined event flood elevations for Unit 2 (NRC, 1988). The result of this evaluation was a stillwater elevation of 15.0 ft with 3.25 ft of wave runup,

resulting in a maximum water surface elevation of 18.25 ft. This evaluation concluded that the physical separation of safety related SSC from the river bank provided a sufficient attenuation distance to mitigate this wave runup height.

4.1.9.2 Reevaluated Flood Elevation

The Maximum Water Elevation resulting from combined external flood effects, is associated with coastal processes and includes storm surge, 25-year storm-related river flooding, antecedent 10 percent exceedance tide, and coincident wind-generated wave activity.. The flood associated with a storm surge stillwater annual exceedance probability of 2×10^{-6} is calculated to be 15.8 feet. The wave crest elevation is calculated to be 16.3 feet. The limit of wave run-up elevation is calculated to be 17.7 feet.

Details of this evaluation are provided in Section 3.9.

4.1.9.3 Comparison

The 17.7 ft elevation of the new evaluation exceeds the CLB combined event elevation by 2.7 ft. However, the 17.7 ft combined event elevation is bound by the NRC independent evaluation value of 18.25 ft, which was considered to not impact safety-related SSC at IPEC as described in Section 4.1.9.1.

4.2 Summary of Walkdown Findings

The following sections provide summaries of walkdown findings for the two UPEC units from (IPEC, 2012a and IPEC 2012b).

4.2.1 IPEC Unit 2 Walkdown Findings (IPEC, 2012a)

There were some observed conditions of features that did not meet the NEI 12-07 acceptance criteria. These conditions were entered into the plant Corrective Action Program; however, only two of these observations were determined to be deficiencies as defined in NEI 12-07. The operability determinations for these deficient conditions concluded that the features did not pose a threat to operability.

A feature found to be deficient was Item IP2-CTL-001, a double door between the 480V switchgear room and the transformer yard. It was observed that the bottom seal on the east section of Item IP2-CTL-001 was degraded. Light could be seen and air cold be felt coming through the seal area. A work order was created to repair the deficient condition.

An existing observation in the CAP identified an incident of water intrusion into the 480V Switchgear room in the Control Building during a volumetric data test of the #22 moat. In-leakage was observed near the north east corner of this room. Repairs to the moat have been made and the latest moat test showed no indication of water leakage into the room..

There were several conduits whose internal seals are currently not part of a preventive maintenance program. These include the conduits that run from Manhole 24 to the 480V Switchgear room and those that run from Manhole 21 to the Zurn Strainer Pit.

4.2.2 IPEC Unit 3 Walkdown Findings (IPEC, 2012b)

There were some observed conditions of features that did not meet the NEI 12-07 acceptance criteria. These conditions were entered into the plant Corrective Action Program (CAP); however, only four of these observations were determined to be a deficiency as defined in NEI 12-07. The operability determinations for the deficiencies concluded that the features did not pose a threat to operability.

A manhole near the southwest corner of the Diesel Generator Building, Manhole D1, contains backflow prevention for the building. A 2 ft flapper valve is located in the manhole and prevents any site drainage or river high water level from backing up into the Diesel Generator Building. During a visual inspection, it was noted the valve did not fully close and seal completely. This valve has subsequently been replaced."

During preparations for Hurricane Sandy from 10/27/2012-10/29/2012, sandbagging was initiated around the IP3 Intake Structure Service Water Pump motors and around the Zurn Strainer Pit inside the IP3 Intake Structure. Several issues were identified:

- There does not appear to be a dedicated sand pile solely for use for flooding mitigation to assure there will be enough sand available to meet the requirements of the flood procedure.

- The flood procedure does not cross-reference the maintenance procedure which provides the details and extent of the sandbagging requirements.
- The sandbagging requirements of the maintenance procedure need to be revisited, as it requires the building of a berm six feet tall and two sandbags deep around the perimeter of the service water pumps.
- The maintenance procedure does not address the sandbagging around the Zurn Strainer Pit area.
- It was observed that three persons were only able to fill 300 sandbags over a period of roughly one day. This number of sandbags is not adequate to meet the needs of the plant in a CLB flood event.

It was also observed during the reasonable simulations that the flood procedure did not correctly identify the location of the temporary pumps on the 33 ft Elevation of the Unit 1 Turbine Building and the incorrect valve number for the Zurn strainer pit sump pump discharge isolation valve was referenced in the procedure. These conditions have been corrected by the purchase of pre-filled sandbags staged on pallets, Tiger Dam flood barriers and revision of appropriate procedures.

Water damage was observed during a visual inspection of IP3-CTL-009. Box XV2, located in the 480V Switchgear room of the IP3 Control Building, showed signs of prolonged exposure to water on the terminals.

There were two conduits that penetrate the south wall and one that penetrates the east wall of the Strainer Pit that do not have internal seals and therefore are potential leak paths into the pit. The 4 inch and 2 inch conduits that penetrate the south wall terminate in Manhole 31. Subsequent inspection of Manhole 31 observed that these conduits are not sealed at the manhole either. The termination point of the 2 inch conduit that penetrates the east wall is unknown. Plant procedures have been revised to protect manhole 31 from river flooding by sandbagging around the manhole.

There were several conduits whose internal seals are currently not part of a preventive maintenance program. These include the conduits that run from Manhole 34 to the 480V Switchgear room and those that run from Manhole 31 to the Zurn Strainer Pit. A PM program is now in place to periodically inspect these seals.

4.3 Impacts of Flood Elevations

Based on the results of the new flood evaluation, four flood mechanisms exceed the IPEC CLB for those specific mechanisms.

Flooding due to combined effects from coastal processes exceeds the CLB by 2.7 ft. However the NRC independent evaluation (NRC, 1988), which predicted a combined effect flood level of 18.25 ft, indicated that wave runup would not impact safety-related SSC at IPEC due to sufficient physical separation from the river edge. The NRC evaluation bounds the new flood results, and as a result the new combined effect evaluation will not impact safety-related SSC at IPEC.

The results of the new flooding on streams and rivers is 14.6 ft, compared to 12.7 ft in the CLB for streams and rivers. The results of the new flooding due to dam failures is14.9 ft compared to 7.2 ft in

the CLB for dam failures. Both reevaluations of the flooding on streams and rivers and dam failures result in flood elevations less than site grade or 15 ft.

LIP caused flooding of the Unit 2 transformer yard results in 0.0 ft to 0.2 ft above door entry levels, which could impact safety-related SSC. Maximum water depths in the vicinity of the doors adjacent to the Unit 2 transformer yard are shown in Table 4-2. Pre-filled Sandbags have been staged in the transformer yards and procedures revised to protect all of the vulnerable doors with respect to the beyond-CLB LIP event.

4.4 References

IPEC, 2010. "Indian Point 2 Updated Final Safety Analysis Report Revision 22", Entergy, 2010, See AREVA Document No. 38-9193643-000.

IPEC, 2011. "Indian Point 3 Updated Final Safety Analysis Report Revision 04", Entergy, 2011, See AREVA Document No. 38-9193643-000.

IPEC, 2012a. "Indian Point Energy Center Unit 2 Flooding Walkdown Submittal Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding", Entergy Nuclear, Engineering Report No. IP-RPT-12-00036, November 2012, See AREVA Document No. 38-9193643-000.

IPEC, 2012b. "Indian Point Energy Center Unit 3 Flooding Walkdown Submittal Report for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Flooding", Entergy Nuclear, Engineering Report No. IP-RPT-12-00038, November 2012, See AREVA Document No. 38-9193643-000.

IPEEE, 1995. "Individual Plant Examination of External Events for Indian Point Unit No. 2 Nuclear Generating Station", Consolidated Edison Company of New York, Inc., December 1995, See AREVA Document No. 38-9193643-000.

IPEEE, 1997. "Indian Point Three Nuclear Power Plant – Individual Plant Examination of External Events", New York Power Authority, IP3-RPT-UNSPEC-02182, September 1997, See AREVA Document No. 38-9193643-000.

NRC, 1988. "External Flooding Condition Technical Specifications for Indian Point Nuclear Generating Unit No. 2 (TAC No. 51921)", U.S. Nuclear Regulatory Commission, November 15 1988.

Table 4.1-1: Flood Elevation Comparisons

Mechanism	CLB Elevation (ft) (NGVD29)****	New Elevation (NGVD29) (ft)	Difference (ft)
Local Intense Precipitation			
Unit 2	See Table 4-2	19.0	See Table 4-2
Unit 3	See Section 4.3	19.2	Bound
Probable Maximum Flood (PMF)– Hudson River	12.7	14.6 (Tide Elev. at Battery 5.35 ft.)	+1.9, Bound
PMF Including Local Oscillatory Wave	13.7	NE	N/A
PMF and High Tide	12.4 (Tide Elev. at Battery +2.2 ft.)	14.6 (Tide Elev. at Battery 5.35 ft.)	+2.2, Bound
PMF and High Tide Including Local Oscillatory Wave	13.4	NE	N/A
PMF and Low Tide	13.0	NE	N/A
PMF and Low Tide Including Local Oscillatory Wave	14.0	NE	N/A
Standard Project Flood and Ashokan Dam Failure	7.2	NE	N/A
PMF and Dam Failure	NE	14.9	N/A
Standard Project Flood and Ashokan Dam failure Including Local Oscillatory Wave	8.2	NE	N/A
PMF and Dam Failure with Coincident Wind-Wave Activity	NE	15.8	N/A
Probable Maximum Hurricane (PMH) and Spring High Tide	13.5	NE	N/A
Probable Maximum Hurricane (PMH) and Spring High Tide Including Local Oscillatory Wave	14.5	NE	N/A

Table 4.1-1: Flood Elevation Comparisons

Standard Project Hurricane and Standard Project Flood on Hudson River	13.0	NE	N/A
Standard Project Hurricane and Standard Project Flood on Hudson River Including Local Oscillatory Wave	14.0	NE	N/A
Standard Project Flood plus Standard Project Hurricane plus Dam Failure	14.0	NE	N/A
Standard Project Flood plus Standard Project Hurricane plus Dam Failure Including Local Oscillatory Wave	15.0	NE	N/A
Hurricane resulting in 500,000-Year Surge plus 25-Year Flood on Hudson River and 10 Percent Exceedance High Tide	NE	15.8	N/A
Hurricane resulting in 500,000-Year Surge plus 25-Year Flood on Hudson River and 10 Percent Exceedance High Tide with Coincident Wind-Wave Activity	NE	17.7	N/A
Seiche	N/A	Screened	N/A
Tsunami	N/A	Screened	N/A, Bound
Ice-Induced Flooding	N/A	12.48	N/A, Bound
Channel Migration or Diversion	N/A	Screened	N/A
Combined Effects Flooding	15.0 / 18.25*	17.7	+2.7/-0.5**

* Note: The NRC independently calculated the probable maximum combined effect flood to be a stillwater level of 15.0 ft plus wave runup of 18.25 ft, and this flood level had no impact to safety-related SSC.

** Note: Value conservatively rounded down.

***Note: "Bound" indicates that the flood hazard has increased for a specific flood mechanism, but does not exceed the plant design

****Note: All values are from Table 2.5-1 of Unit 2 UFSAR Rev 22

NGVD29 is equivalent to MSLCLB Combined Effects Flooding is based on Standard Project Flood plus Standard Project Hurricane plus Dam Failure with Coincident Wind-Wave Activity. Reevaluated Combined Effects Flooding based on combination of the 25-year flood in the stream, the hurricane resulting in 500,000-year storm surge with wind-wave activity, and the antecedent 10 percent exceedance high tide

NE indicates mechanism not evaluated.

Table 4-2: Unit 2 Transformer Yard Doorway LIP Flood Impacts

Unit 2 Doorway	Door Elevation (ft)	Flood Elevation (ft) near Door	Water Depth at Door (ft)
U2-PAB-1	18.9	18.9	0.0
U2-ABFP-1	18.6	18.5	-0.1
U2-ABFP-2	18.7	18.8	+0.2
U2-ABFP-2	18.7	18.8	+0.2
U2-CB-1	18.7	18.9	+0.2

5.0 INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED

Storm surge and combined effects resulting from the postulated hurricane is the bounding event that exceeds the Current Licensing Basis (CLB) flood level. These results are shown below in Section 5.1.

Additional results for the new flooding on streams and rivers is 14.6 ft, compared to 12.7 ft in the CLB and the new flooding due to dam failures is14.9 ft compared to 7.2 ft in the CLB for dam failures. Both reevaluations of the flooding on streams and rivers and dam failures result in flood elevations less than site grade or 15 ft and are thus bounded by the storm surge and combined events results.

Although not part of the CLB, Local Intense Precipitation (LIP) based on the new hazard evaluation will produce ponding in the transformer yard areas that will require certain doors to be protected from flooding. This section describes the interim actions to be taken to mitigate the effects of the Surge and LIP.

Activities to mitigate the effects of External Flood events are performed in accordance with Operations and Maintenance procedures. Both Sandbag and Tiger Dam placement is controlled by maintenance procedure 0-MET-402-GEN. The guidance set forth in the maintenance procedure for placement of sandbags is based on guidance contained in literature for good practices. Placement of the Tiger Dams is in accordance to manufacturer's instructions. Since the wave crest elevation is only projected to be 1.3' above the flood stage level, the wall height of the sandbags, only approximately 3', and the placement locations either indoors or in outdoor areas shielding from direct wave action, it is not anticipated that the sandbag walls or Tiger Dams will be adversely affected by the floodwaters. Even conservatively considering wave run up heights estimated to be 2.7' above flood stage, a 3' tall sandbag or Tiger Dam wall has margin. With the exception of the protection of manholes 21, 23, 31,33 & 37, the interim actions described herein have been in place since July of 2011

5.1 Storm Surge and Local Intense Precipitation

The results of the evaluation for Storm Surge in Section 3.4 for a 500,000 year storm determined that the maximum water levels at the waterfront as a result of the postulated hurricane are as follows:

Still Water Elev.	Wave Crest Elev.	Limit of Run-up Elev.
15.8'	16.3'	17.7'

These levels exceed the maximum CLB stillwater Elevation of 15' resulting from the postulated storm surge combined with the 25 year flood, 10% antecedent high water level and wave action. The storm surge results govern the over the LIP for areas located below the 18' elevation.

IPEC currently has actions in place to protect vital SSC's located in the following areas of the facility to flood levels up to El. 17'-11" and the postulated LIP event:

- Unit 2 Intake Structure (Service Water Pumps)
- Unit 3 Intake Structure (Service Water Pumps)
- Unit 2 480 Volt Switchgear Room
- Unit 3 480 Volt Switchgear Room
- Unit 2 Turbine Building for MCC 24A
- Unit 3 Turbine Building for MCC 34
- Unit 3 Emergency Diesel Generator Building
- Unit 2 Service Water Valve Pit
- Unit 3 Service Water Valve Pit

Due to the general elevation of the transformer yard (Elevation 18') and location with respect to the river, river flooding will not affect the transformer yard. As a result, with the stillwater elevation postulated to be 15.8', storm surge will not affect this area. Only ponding resulting from the LIP needs to be considered. This affects the following plant components:

- Door 224 (U2-ABFP-1)
- Door 225 (U2-ABFP-2)
- Door 226 (U2-ABFP-3)
- Door 227 (U2-PAB-1)
- Door 229 (U2-CB-1)
- Door 215 (U3-ABFP-1)
- Door U3-ABFP-2
- Door 212 (U3-ABFP-3)
- Door FDR-11-PA (U3-PAB-1)
- Door 305 (U3-PAB-2)

- Door 208 (U3-CB-1)

Current LIP mitigation strategies for the above doors are in place.

No actions are required for flooding due to mechanisms other than LIP or storm surge events. Floods due to all other mechanisms have peak elevations below the plant's CLB or are bounded by the LIP or storm surge events.

5.1.1 Intake Structure

The Intake Structures at both Unit 2 and Unit 3 are difficult targets to defend from the postulated storm surge levels due to their proximity to the shoreline. The vulnerable components at the Intake Structure are the Service Water Pump Motors which are located on top of the pumps at an approximate bottom elevation of 18'. The pump and motors are protected from direct wave action due to large equipment (traveling screen system) located between them and the river. The maximum still water river level from the 500,000 year storm is 15.8' and including wave run up is 17.7', which are both below the approximate elevation of the pump motors and should not adversely affect pump operation. However, mitigating actions are in place to keep the river flood waters from directly impacting the pumps/motors up to a level of 17'-11" through the placement of sandbags.

5.1.2 Protection of the Unit 2 and Unit 3 480V Switchgear Rooms

Switchgear rooms at Unit 2 & 3 are protected up to a river water elevation of 17'-11".. These areas are away from the river front and their access points are located within the existing Turbine Buildings. Although the Turbine Buildings will be allowed to flood, the buildings will protect the access doors to the Switchgear rooms from wave action. The expected water elevations at the access door locations are equal to the determined still water elevation of 15.8'. As a result, the mitigating actions are sufficient to protect the Switchgear rooms. These actions are summarized below:

- The Unit 2 480 Volt Switchgear Room is located at El. 15'-0" in the Control Building. There are multiple locations that require flood protection. The first is the fire door just outside of Door 234 and the second is Door 235. There are also three floor drains and six hub cleanouts. Tiger Dams and or sand bags will be used for Door 235 since this area is clear of interferences and is relatively flat. Sand bags will be used at Door 234 since there is no water source in the vicinity to fill the Tiger Dam. Inflatable drain plugs will be used to seal the floor drains and hub cleanouts.

    - The Tiger Dam comes in a standard 50' length. A single 50' long tube can be pinched as necessary to reduce the length. Three Tiger Dams are required at Door 235 setup in a pyramid (two tubes followed by one on top). The water source for Door 235 is FP-150 which has a hose assembly that is adequate to reach the Tiger Dam location. An additional coupler is required to attach the fire hose to the Tiger Dam prior to filling with water.

- Considering a sand bag wall that is 2 sand bags deep and 3 foot tall is approximately 12 sand bags per foot. One foot of height is required on the 6'-5" side because a concrete wall already exists which is 25" tall. This results in approximately 4 sandbags per foot.

The Unit 3 480 Volt Switchgear Room is located at El. 15'-0" in the Control Building and already has a flood barrier that is 4'-0" tall (verified via field measurements) located directly outside Fire Door FDR-1-CG. However, this flood barrier is typically removed during outages to allow craft to enter and exit the Switchgear Room more quickly. Therefore, if this flooding event were to occur during an outage, the 480 Volt Switchgear Room would be vulnerable. The flood barrier will be reinstalled in the event that this flood scenario occurs during an outage, or, in the unlikely event that the flood barrier cannot be re-installed, sand bags will be utilized.

In addition to the measures discussed above, manholes 21,23,31,33 and 37 will be protected by the placement of sandbags. These manholes contain conduits that communicate with the Unit 2 & 3 switchgear rooms and require protection from inundation by the storm surge. Due to the potential of groundwater intrusion and some sandbag leakage, and to enhance this protection, these conduits will be sealed as discussed in Section 6 of this report.

5.1.3 Protection of MCC 24A & 34A

These MCCs are located in the Turbine Buildings and provide power to certain EDG auxiliaries, including the EDG Compressors, Jacket Water Heaters and Lube Oil Heaters. As with the doors to the switchgear rooms, the expected water elevations at the MCC locations are equal to the determined still water elevation of 15.8'. The protections set up are adequate to protect these MCC's from the postulated flood. These protection descriptions are summarized below are based on the protection level up to elevation 17'-11".

MCC 24A is located at El. 15' in the Unit 2 Turbine Building. Two existing concrete walls provide adequate flood protection on the north and east sides and a 1 ft tall concrete curb surrounds the south and west sides. In order to provide flood protection up to 17'-11", Tiger Dams will be used on the south and west sides of the MCC.

Three standard Tiger Dams are required around MCC 24A setup in a pyramid (two tubes followed by one on top). The proposed water source for the MCC 24A Tiger Dam is FP-686, which has a hose assembly that is adequate to reach the Tiger Dam location. An additional coupler is required to attach the fire hose to the Tiger Dam prior to filling with water.

- MCC 34 is located at El. 15' in the Unit 3 Turbine Building. There is adequate clearance on the south and west sides to install Tiger Dams. These features will provide flood protection to a height of 17'-11". For the north side sand bags or equivalent barrier will be used since the floor space available is considerably smaller than the other sides. Additional plastic sheeting can be used as well to supplement the sand bags.

Three standard Tiger Dams are required around MCC 34 setup in a pyramid (two tubes followed by one on top). The proposed water source for the MCC 34 Tiger Dam is FP-149, which has a hose assembly that is adequate to reach the Tiger Dam location. An additional coupler is required to attach the fire hose to the Tiger Dam prior to filling with water.

5.1.4 Protection of Unit 3 EDG Diesel Cells

The Unit 3 EDG diesels are located in the Unit 3 Control Building at the 15' elevation. The EDG diesels are located in separate reinforced concrete cells adjacent to the 480V switchgear room. The current 4' tall wall in front of the entrance to the switchgear room will protect the diesel cells from floodwaters entering through the access doors to the room and cells.

The other area of vulnerability for flooding of the diesel cells is through the drain pipes located in the sumps of each cell. These drain lines are connected to a common drain header that discharges to manhole D-1 located outside and south of the Control Building. Manhole D-1 then discharges into the river. To prevent river water from being able to back up into the diesel cells through the common drain header, a flapper valve is installed at the discharge point of the drain header in Manhole D-1. This valve acts as a check valve that is shut against the header pipe when water floods the manhole. This valve was recently replaced as a result of the Fukushima flooding walk down inspections.

Therefore, the Unit 3 EDG cells are currently protected from the new flooding hazard and require no additional interim measures.

5.1.5 Protection of Unit 2 & 3 Service water Valve Pit

The Unit 2 & 3 Service water Valve Pits are located just west and outside of the Turbine Buildings. The openings to these pits are approximately located at the 15' elevation and would be subject to flooding. These pits contain manually operated valves that allow operations personnel to swap service water headers as necessary during plant operations. During the postulated hurricane that produces the storm surge, it is not expected that these valves will need to be operated. As a result, no interim protection measures are required for the service water valve pits.

5.1.6 Flood Warning Times and Durations

Time series plots of flood water elevations during the critical flooding scenarios are provided in the Section 3.1 (Figures and) for LIP. These figures provide timing and duration of the flooding. Warning time will be on the order of hours based on standard information from the National Weather Service for a LIP event. The National Weather Service, Hydrometeorological Prediction Center, produces various guidance forecast products to assist weather and river forecast centers. Quantitative precipitation forecasts (QPFs) are particularly useful in determining when an LIP event might occur. The QPF provides rainfall over the continental U.S. for up to seven days at various intervals. These forecasts

depict isohyets in varying increments of accumulated precipitation expected in each interval. (NOAA 2013) The longer term QPF provides an awareness level as to the potential for a minimum LIP event. Similarly, an alert level requiring standby action could be taken using a shorter period QPF. Again, if the total precipitation estimated in three days is a certain percentage (to be determined) of the minimum LIP.

Current plant operating procedures begin preparations for flooding protections 48 hours prior to a predicted hurricane impact or of a high tide warning with tides expected to exceed 4.5'(Unit 2) or 7' (Unit 3). For LIP type events, the procedures enter into the flood preparations for forecasted 24 hour rainfall that would exceed 5", which is significantly less than the postulated LIP event. With the use of pre-filled sandbags, the staging of the sandbags on pallets in the vicinity of vulnerable areas, and Tiger Dams staged ina trailer, sufficient time will be available to implement the interim measures prior to the arrival of floodwaters Surge duration, a duration for flood waters to be above the CLB elevation of 15 feet (NVGD29) is expected to be approximately 3 hours (AREVA 2013, Figure 6).

5.2 Reference

AREVA, 2013. "Combined Events Flood Analysis – Riverine", AREVA Document No. 32-9196323-000, 2013.

6.0 ADDITIONAL ACTIONS

Currently, mitigation measures are in place at IPEC to provide protection against a river flood level up 17'-11", which is 2'-11" above the current design basis (CLB). As described in Section 5 of this report, the protection measures primarily involve placement of sandbags and or Tiger Dams to hold back floodwaters from adversely affecting critical equipment. This includes placement of sandbags around certain manholes that contain conduits which communicate with the switchgear rooms at both units. If these manholes become inundated with floodwaters, these conduits could provide a leak path into the 480V switchgear rooms of both units.

To enhance the current protection measures and provide additional protection against normal seepage through sandbag walls and the potential of some groundwater intrusion into the manholes, all of the conduits that communicate with the 480V switchgear rooms and originate in manholes will be sealed. These conduits will be sealed with sealant tested and shown to be capable of resisting hydrostatic head pressures that can result from the beyond design basis flood levels discussed in this report. These seals will be installed in accordance to Entergy procedures governing the modification process.

APPENDIX A: ADDITIONAL SIMULATION MODEL USE DESCRIPTIONS

A.1 Additional Computer Software Used for Flood Simulations

All software codes used for IPEC flood analyses are those recommended in NUREG/CR-7046, with one exception: FLO-2D. That program is described below.

A.1.1 FLO-2D for LIP Simulations

The example LIP calculation presented in Appendix B of NUREG/CR-7046 (NRC, 2011) used HEC-HMS and HEC-RAS, developed by Hydrologic Engineering Center of US Army Corps of Engineers. The hydrologic part of the calculation was performed within HEC-HMS, whereas the hydraulic part of the calculation was performed within HEC-RAS. In this flood re-evaluation study, FLO-2D was selected for calculation of the LIP-induced PMF at IPEC. For the LIP calculation, rainfall runoff was calculated internally by FLO-2D and translated into overland flow within FLO-2D.

Appendix A was prepared as per Section 5.3 of NUREG/CR-7046 (NRC, 2011) to describe FLO-2D.

A.1.2 Software Capability

The FLO-2D computer program was developed by FLO-2D Software, Inc., Nutrioso, Arizona. FLO-2D is a combined two-dimensional hydrologic and hydraulic model that is designed to simulate river overbank flows as well as unconfined flows over complex topography and variable roughness, split channel flows, mud/debris flows and urban flooding.

FLO-2D is a physical process model that routes rainfall-runoff and flood hydrographs over unconfined flow surfaces using the dynamic wave approximation to the momentum equation. The model has components to simulate riverine flow including flow through culverts, street flow, buildings and obstructions, levees, sediment transport, spatially variable rainfall and infiltration and floodways. Application of the model requires knowledge of the site, the watershed (and coastal, as appropriate) setting, goals of the study, and engineering judgment. This software will be used to simulate the LIP, propagation of storm surge, seiches, and riverine flow through overland flow and channels to establish stillwater levels at various Flood Hazard Re-evaluation Project sites.

The major design inputs to the FLO-2D computer model are digital terrain model of the land surface, inflow hydrograph and/or rainfall data, Manning's roughness coefficient and Soil hydrologic properties such as the SCS curve number. The digital terrain model of the land surface is used in creating the elevation grid system over which flow is routed. The specific design inputs depend on the modeling purpose and the level of detail desired.

The following executable modules compose the FLO-2D computer program:

*.exe File	Size
FLO.exe	10.76 MB
GDS.exe	6.00 MB
PROFILES.exe	2.84 MB
HYDROG.exe	2.07 MB
Mapper_2009.exe	3.33 MB
MAXPLOT.exe	2.32 MB

FLO.exe is the model code that performs the numerical algorithms for the aforementioned components of the overall FLO-2D computer model.

GDS.exe graphically creates and edits the FLO-2D grid system and attributes and creates the basic FLO-2D data files for rainfall – runoff and overland flow flood simulation. PROFILES.exe displays the channel slope and permits interactive adjustment of the channel properties. HYDROG.exe enables viewing of channel outputs hydrographs and lists average channel hydraulic data for various reaches of river. Mapper_2009.exe and Maxplot.exe enables graphical viewing of model results and inundation mapping.

A description of the major capabilities of FLO-2D which will be used for this project is provided in Section A.1.2 below.

A.1.3 Model Components

Overland Flow Simulation

This FLO-2D component simulates overland flow and computes flow depth, velocities, impact forces, static pressure and specific energy for each grid. Predicted flow depth and velocity between grid elements represent average hydraulic flow conditions computed for a small time step. For unconfined overland flow, FLO-2D applies the equations of motion to compute the average flow velocity across a grid element (cell) boundary. Each cell is defined by 8 sides representing the eight potential flow directions (the four compass directions and the four diagonal directions). The discharge sharing between cells is based on sides or boundaries in the eight directions one direction at a time. At runtime, the model sets up an array of side connections that are only accessed once during a time step instead of the dual algorithm required by searching for available elements. The surface storage area or flow path can be modified for obstructions including buildings and levees. Rainfall and infiltration losses can add or subtract from the flow volume on the floodplain surface.

Channel Flow Simulation

This component simulates channel flow in one-dimension. The channel is represented by natural, rectangular or trapezoidal cross sections. Discharge between channel grid elements are defined by average flow hydraulics of velocity and depth. Flow transition between subcritical and supercritical flow is based on the average conditions between two channel elements. River channel flow is routed with

the dynamic wave approximation to the momentum equation. Channel connections can be simulated by assigning channel confluence elements.

Flood Channel Interface

This FLO-2D component exchanges channel flow with the floodplain grid elements in a separate routine after the channel, street and floodplain flow subroutines have been completed. An overbank discharge is computed when the channel conveyance capacity is exceeded. The channel-floodplain flow exchange is limited by the available exchange volume in the channel or by the available storage volume on the floodplain. Flow exchange between streets and floodplain are also computed during this subroutine. The diffusive wave equation is used to compute the velocity of either the outflow from the channel or the return flow to the channel.

Floodplain Surface Storage Area Modification and Flow Obstruction

This FLO-2D component enhances detail by enabling the simulation of flow problems associated with flow obstructions or loss of flood storage. This is achieved by the application of coefficients (Area reduction factors (ARFs) and width reduction factors (WRFs) that modify the individual grid element surface area storage and flow width. ARFs can be used to reduce the flood volume storage on grid elements due to buildings or topography and WRFs can be assigned to any of the eight flow directions in a grid element to partially or completely obstruct flow paths in all eight directions simulating floodwalls, buildings or berms.

Rainfall – Runoff Simulation

Rainfall can be simulated in FLO-2D. The storm rainfall is discretized as a cumulative percent of the total. This discretization of the storm hyetograph is established through local rainfall data or through regional drainage criteria that defines storm duration, intensity and distribution. Rain is added in the model using an S-curve to define the percent depth over time. The rainfall is uniformly distributed over the grid system and once a certain depth requirement (0.01-0.05 feet) is met, the model begins to route flow.

Hydraulic Structures

Hydraulic structures including bridges and culverts and storm drains may be simulated in FLO-2D Pro. Discharge through round and rectangular culverts with potential for inlet and outlet control can be computed using equations based on experimental and theoretical results from the U.S. Department of Transportation procedures (Hydraulic Design of Highway Culverts; Publication Number FHWA-NHI-01-020 revised May, 2005).

Levees

This FLO-2D component confines flow on the floodplain surface by blocking one of the eight flow directions. A levee crest elevation can be assigned for each of the eight flow directions in a given grid element. The model predicts levee overtopping. When the flow depth exceeds the levee height, the discharge over the levee is computed using the broad-crested weir flow equation with a 2.85 coefficient. Weir flow occurs until the tailwater depth is 85% if the headwater depth. At higher flows, the water is exchanged across the levees using the difference in water surface elevations.

A.1.4 FLO-2D Model Theory

Governing equations and solution algorithm are presented in details in FLO-2D Reference Manual (FLO2D, 2009). The general constitutive fluid equations include the continuity equation and the equation of motion (dynamic wave momentum equation) (FLO-2D, 2009a, Chapter II):

$$\frac{\partial h}{\partial t} + \frac{\partial h V}{\partial x} = i$$

$$S_f = S_o - \frac{\partial h}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t}$$

where

h = flow depth;

V = depth averaged velocity in one of the eight flow directions;

x = one of the eight flow directions;

i = rainfall intensity;

S_f = friction slope based on Manning's equation;

S_0 = bed slope

g = acceleration of gravity

The partial differential equations are solved with a central finite difference numerical scheme, which implies that final results are just approximate solutions to the differential equations. Details on the accuracy of FLO-2D solutions are discussed in FLO-2D Validation Report (FLO-2D, 2011).

A.1.5 Model Inputs and Outputs

Inputs to FLO-2D are entered through a graphical user interface (GUI), which creates ASCII text files used by the FLO-2D model (FLO-2D, 2009b). The ASCII text files can be viewed and edited by other ASCII text editors such as MicroSoft WordPad.

Calculated results from FLO-2D simulations are saved in the ASCII text format in a number of individual files. The results can be viewed with the post-processor programs as follows:

- Mapper – to view grid element results such as elevation, water surface elevation, flow depth and velocity, to create contour maps and to generate shapefiles that can later be used by GIS mapping software such as ArcMap.

- MAXPLOT – to view grid element maximum flood elevation, flow depth, velocity, channel flow depth/elevation/velocity, and levee minimum free board/overtopping.

- HYDROG – to generate hydrographs for channel elements.
- PROFILES – to plot channel water surface and channel bed profiles.

A.1.6 Conclusions

FLO-2D is a FEMA-approved software (FLO-2D, 2011). The model validation report prepared for FEMA and the FLO-2D software certification prepared for Flood Re-evaluation Projects (AREVA, 2012) have demonstrated its modeling capabilities and numerical accuracy. It is therefore judged to be an appropriate modeling tool for the Grand Gulf flood re-evaluation study where 2-dimensional overland flow is predominant.

A.1.7 References

AREVA, 2012. AREVA Document No. 38-9191747-000, Computer Software Certification – FLO-2D Pro, GZA GeoEnvironmental, Inc., October 2012.

FLO-2D, 2009a. FLO-2D Reference Manual, FLO-2D Software, Inc.

FLO-2D, 2009b. FLO-2D Data Input Manual, FLO-2D Software, Inc.

FLO-2D, 2011. FLO-2D Model Validation for Version 2009 and up prepared for FEMA, FLO-2D Software, Inc, June 2011.