2.4 Storm Surge

2.4.1 Methodology

The HHA approach described in NUREGCR-7046 (NRC 2011) was used to calculate the Probable Maximum Storm Surge (PMSS) at NMP Unit 1 and Unit 2. In accordance with the HHA approach, this calculation utilizes the simple Great Lakes Storm Surge Planning Program (SSPP) software program developed by NOAA's Great Lakes Environmental Research Laboratory (GLERL) (Schwab et al. 1987) and the Probable Maximum Wind Storm (PMWS) wind velocity and direction, to conservatively predict the PMSS at NMP Unit 1 and Unit 2.

The methodology to determine the design storm surge at NMP Unit 1 and Unit 2 includes the following steps (as defined in ANSI/ANS 2.8-1992):

- Selection of the historic design storm by performance of a statistical analysis of NOAA one hour and six minute water level data to: a) eliminate long term water level fluctuations; b) identify the short term water level fluctuations; and c) identify the historical storm and storm type resulting in the highest recorded wind speeds and storm surges.
- Development of the PMWS meteorological parameters by modification of the historical storm parameters in accordance with ANSI/ANS-2.8-1992.
- Development of the antecedent water level by comparing the maximum controlled water level elevation on Lake Ontario to the 100-year high water level, and selecting the lesser water elevation.
- Calculation of the PMSS still water elevation using the SSPP.

2.4.2 Results

2.4.2.1 Selection of the Design Storm

Consistent with NUREG/CR-7046 and ANSI/ANS-2.8-1992, identification of the historic storm for development of the PMWS parameters was based on review and analysis of the National Oceanic and Atmospheric Administration (NOAA) National Climatic Data Center (NCDC) (NOAA, 2012c) water level data from nearby NOAA Tides and Currents stations (NOAA, 2012b) and the NOAA National Hurricane Center (HURDAT) data (EC 2012a-d). The storm types evaluated included tropical cyclones (hurricanes), moving squall lines and extra-tropical cyclones.

The physical characteristics of Lake Ontario are indicated on Figure 2.4-1. Lake Ontario has a length of about 195 miles and an average width of about 53 miles, and is oriented with its long dimension trending in a northeast-southwest direction. The bathymetry of the lake offshore in the vicinity of NMP is characterized by a deep basin, with a lake bottom depth of about 800 ft. NMP is located along the southeast lake shoreline.

Water-level fluctuations due to storm surges on the Great Lakes are generally caused by one of several types of strong storms, including: 1) tropical systems that move north from the Gulf Region and Mid-

Atlantic; 2) non-convective storms (extra-tropical cyclones) that originate in Canada and move to the east through the lakes region (Alberta Low) or originate in the southern and central Rockies (Colorado Low) and move east through the lakes region; and 3) convective storms or thunderstorm frontal passages, including moving squall lines (FEMA 2012).

The NOAA Hurricane Research Division historic hurricane data record (HURDAT) was evaluated relative to tropical systems originating in the southern latitudes and moving into the Great Lakes area. The review of historic hurricane activity for Lake Ontario identified 17 extra-tropical or tropical cyclones that have tracked over or near Lake Ontario (EC 2012a). The storms are summarized on Table 2.4.1, which presents the storm speed, wind speed (1-minute, 10 meter) and pressure at the storm track latitudes and longitudes near the lake. Storm tracks were typically in an approximate north-south direction. Storm forward speeds ranged from 6 to 58 miles per hour and averaged 28 miles per hour. Wind speeds ranged from 25 to 70 miles per hour and averaged 41 miles per hour. Fifteen of the 17 storms had transitioned into extra-tropical storms before reaching Lake Ontario. The other two remained tropical depressions or storms. Six of the storms are of note for Lake Ontario due to observed flooding, wind or rainfall. Hurricane Hazel in 1954 caused widespread rainfall-induced flooding along the western shore of Lake Ontario (EC 2012b). Hurricane Isabelle, which passed over Lake Erie to the west, in September, 2003 caused large waves in the western portion of the Lake Ontario (EC 2012c). Hurricane Audrey (July 1957), Hurricane Katrina (August 2005), and Hurricane Frances (September 2004) were reported to have caused extreme rainfall in and around Lake Ontario (EC 2012c, EC 2012d). Rainfall from Hurricane Fran (September 1996) caused flooding in the western portion of the lake, however water level records from the southern and eastern portion of the lake show no significant surges (EC 2012d).

Squall lines are also a consideration in the Great Lakes region, particularly along the shores of Lake Michigan. ANSI/ANS-2.8-1992 states that "A moving squall line should be considered for the locations along Lake Michigan where significant surges have been observed because of such a meteorological event" and notes the possible occurrence of squall lines within the other Great Lakes (ANSI/ANS, 1992). Review of the literature indicates that moving squall lines are not the controlling storm event relative to storm surge in Lake Ontario. Specifically:

- Federal Emergency Management Agency (FEMA) publications indicate that while moving squall lines are possible in Lake Ontario, it is generally accepted that these local, fast-moving events can be neglected when assessing extreme water levels as their inclusion has negligible influence on water level statistics (FEMA 2012).
- Analyses by Melby et al. indicate that, in general, neglecting convective events (i.e., moving squall lines) has minimal influence on external water-level statistics (Melby et al. 2012).

The parameters of moving squall lines have been developed previously at NMP and American Nuclear Society (ANS) studies, including:

1. ANS developed probable maximum squall line parameters for the Great Lakes in 1976, which were adopted and included in ANSI/ANS-2.8-1992 (ANSI/ANS 1992). These parameters included a pressure gradient of 8 mb over 11.5 miles and a maximum wind of 75 mph (NMP2 1976).

2. Design parameters for moving squall lines were previously developed for NMP and presented in the Revetment-Ditch system design document for NMP Unit 2 (NMP2 1976). Due to a lack of available historic data, the analysis developed synthetic pressure and wind field utilizing Fujita's squall line model (Fujita 1955), as well as historic recorded data for a 1953 squall line that occurred in Nebraska and was documented by Fujita to have a pressure gradient of 9 mbars over 50 miles and a sustained wind speed of 45 mph (Fujita et al. 1956). In the analysis, the wind speed was conservatively increased to 50 mph to account for the lack of local, supporting historical data (NMP2 1976). The NMP Unit 2 study also used the Fujita (1955) model to construct horizontal pressure and wind fields for squall line parameters defined by the ANS. Since the ANS standards were not yet official in 1976, the NMP Unit 2 study also developed a "severe squall line" in which the historical maximum pressure gradient (from the 1953 Nebraska event) was increased 30 percent from 9 mbars to 12 mbars over 50 miles and the maximum wind speed was increased 50 percent from 50 mph to 75 mph (NMP2 1976). Using these three models of moving squall line pressure and wind fields, a two-dimensional storm surge model was applied. The model predicted a storm surge of no more than 1.8 ft for any of the events modeled (NMP2 1976).

As noted above, most of the strong storms in the Great Lakes are extra-tropical, low-pressure non-convective systems. These non-convective storms typically originate in Canada or the southern or central Rockies and move to the east. The movement of high-pressure systems through the region often precedes or follows the occurrence of the low-pressure system. Low-pressure systems spin counter-clockwise, while high-pressure systems spin the opposite way. Winds on the eastern side or leading edge of a low-pressure system are typically coming from the south, while winds on the eastern side of a high-pressure system are coming from the north. High winds and large atmospheric pressure variations are commonly associated with these storm events, and they can cause elevated water levels, or storm surge, along the lake shoreline (FEMA 2012). This is consistent with findings of Danard (Danard et al. 2003) who concludes that the occurrence of storm surges on Lake Ontario is mainly due to extra-tropical cyclones. The results of a 2003 joint IJC/USACE Lake Ontario Waves study (IJC 2003) indicates that waves generated by extra-tropical events are more intense than those generated by convective events (squall lines). The winter season is characterized by the most severe cyclones. The principal storm track of winter storms in the Erie-Ontario region is to the northeast (Angel 1996).

In accordance with the procedures presented in ANSI/ANS-2.8-1992, an analysis of available data for historic synoptic cyclonic wind storms was performed. Long-term (about 50 years) Lake Ontario wind and water level records were compiled from the NOAA National Climatic Data Center Storm Events Database (NOAA 2012a) and the Tides and Currents Great Lakes Water Level Data (NOAA 2012b). Lake Ontario has four NOAA Tides and Currents Stations (see Figure 2.4-2): Station 9052076, Olcott, New York; Station 9052058, Rochester, New York Station 9052030, Oswego, New York; and Station 9052000, Cape Vincent, New York. Hourly water level data is available for the period of 1961 to 2012. Six minute water level data was evaluated for the period of about 1994 to 2012 and wind data is available for the period of about 1950 to 2012.

Statistical analyses of available hourly and six-minute water level data (two separate data sets) from all four NOAA stations (NOAA 2012b) was performed by processing the data with a frequency domain filter in order to attenuate signals from high frequency events such as lake water level fluctuations associated with the annual hydrologic cycle and to identify water levels associated with major storms. Table 2.4-2 identifies the storms that resulted in the top twenty surges identified by the statistical analysis of the hourly water levels. All but one of the storms were extra-tropical events occurring during the winter months. The highest surge identified was 2.06 ft. at Cape Vincent, NY on November 13, 1992. Wind data is not available for this storm event because this event is not present in the NOAA Storm Events Database (NOAA 2012a). The second highest surge identified was 1.88 ft. at Cape Vincent, NY on February 17, 2006. This extra-tropical storm also had the highest recorded wind speed (80.6 mph, NOAA 2012a) of the surge-causing events.

2.4.2.2 Development of the Design Storm Parameters

The February 17, 2006 storm, identified by the statistical analysis presented above to have had the greatest recorded wind speed and the second largest surge elevation, was selected as the model storm and modified (per ANSI/ANS-2.8-1992) to develop the PMWS parameters.

Three hour (3-h) pressure maps for the February 17, 2006 storm were obtained from the NCDC (NOAA 2006). The 15:00 GMT pressure fields were identified as the most critical based on the observed isobars and resultant pressure gradient. The surface analysis map for 15:00 GMT on February 17, 2006 is shown in Figure 2.4-3. The pressure map was geo-referenced in ArcMap™ to calculate the isobars, distance between isobars, and wind angles affecting the lake. Geo-referencing is the process of referencing geographic data to the earth's surface. In this case, geo-referencing was used to establish a relationship between the pressure maps and a geographic coordinate system in order to determine storm speed. The pressure maps were loaded as raster images into ArcMap™, where the geo-referencing Toolbar was used to apply a geographic coordinate system to the images.

PMWS Storm Track and Speed

The February 17, 2006 storm originated in the west and travelled in an approximately southwest to northeast direction. The storm track of the PMWS follows the primary tracks constructed from the historic extra-tropical cyclone climatology for the Fall, Winter, and Spring seasons. (Angel 1996) Its recorded translational speed was between 40 and 50 mph. The path of the February 17, 2006 storm was smoothed to develop the PMWS storm track and the storm's translational speed was conservatively maintained at a constant steady-state speed of 40 mph, corresponding to the lower range of the recorded storm speed. The PMWS storm track is presented in Figure 2.4-4. The PMWS track is consistent with the tracks of the other representative storms that generated significant surges at the site which have three hour surface pressure maps available. The storm tracks for the February 1, 2002, December 24, 2004, February 17, 2006 and the January 17, 2012 storms compared well with the PMWS storm track. The March 9, 2002 storm track varied significantly from the other storm tracks affecting Lake Ontario. The track for this storm shows that the storm was impacted by a secondary cold front that appears to have steered the storm to the north but also generated the winds which resulted in the high surge. Based on the storm track data, the PMWS track represents a conservative track for the generation of winds on Lake Ontario which is deemed to have a reasonable probability of occurrence.

PMWS Wind Field

Lake Ontario was divided into three zones: Z1–Western, Z2–Central, and Z3–Eastern to develop a spatially varying wind field (see Figure 2.4-5). The isobar patterns as the storm moved along the storm path were used to calculate the PMWS time varying pressure, wind speed and wind direction at the eastern and western ends of each zone using the methods presented in the U.S. Army Corps of Engineers (USACE) Coastal Engineering Manual (CEM) (Resio et al. 2008). Wind direction in each zone was estimated from the orientation of the isobars. Wind directions were calculated to be at an angle of 10 degrees across the isobars (as specified by ANSI/ANS-2.8-1992 for the Great Lakes region).

In order for the maximum wind speed to reach 100 mph (ANSI/ANS 1992) in each zone, the wind speeds were scaled up by a factor equal to the ratio of 100:maximum wind speed: 1.4 in Zone 1, 1.5 in Zone 2, and 1.3 in Zone 3. In order for the minimum pressure to reach 950 mbar (ANSI/ANS 1992), the minimum pressure of the storm, 992 mbar, was scaled down to 950 mbar. The PMWS parameters are presented in Table 2.4-3.

2.4.2.3 Development of the Antecedent Water Level

As defined in Appendix H of NUREG/CR-7046 (U.S NRC 2011) for enclosed bodies of water, the lesser of the 100-year level or the maximum controlled water level should be used for the evaluation of flood levels from storm surges.

Lake Ontario has been regulated by the International Joint Committee (IJC) (formerly the International St. Lawrence River Board of Control) under Plan 1958-D since 1960. Actual control of the lake elevations started in 1963. The current regulated water level of Lake Ontario under Plan 1958-D, defined as the regulated monthly mean level, is Elevation 247.3 ft IGLD85 (IJC 2012a). Proposals for Plan Bv7 2011 (IJC, 2012a) to modify the regulated water levels are currently under study and review. As part of the evaluation process for the adoption the lake water regulation plan, the IJC analyzed 101 years of water supply data to estimate lake water levels. Under the proposed Plan Bv7 (IJC 2012a), the regulated monthly level is exceeded about 5 percent of the time (during the Late Fall and Winter Months; September through March).

A frequency analysis of the monthly mean water level data from NOAA Station 9052030 (Oswego, NY), for the period of time corresponding to the period that the Lake Ontario water level has been regulated and controlled (1963 to 2012), was performed using a Log-Pearson III statistical analysis to calculate the 100-year high water level. The 100-year high water level was calculated to be Elevation 248.4 ft IGLD85 as part of this calculation using methods from reference U.S. Dept. of the Interior, 1982.

Since the maximum controlled water level elevation level of 247.3 (IGLD85) (IJC 2012a) is less than the 100-year water level elevation of 248.4 (IGLD85), it is selected as the ambient water level for the PMSS calculation.

2.4.2.4 GLERL SSPP Storm Surge Model

The SSPP was used to predict the surge elevation due to the PMWS. Appendix A provides a description of the program. The SSPP automatically calculates the maximum and minimum water level during the 12 hours following the onset of the wind at 15 defined points (see Figure 2.4-6) along the southern and eastern shore of Lake Ontario. As shown in Figure 2.4-6, NMP is represented by point 10. The SSPP model was run for a sustained wind speed of 100 mph. The wind direction was varied in 10 degree increments between 250 and 300 degrees to determine the wind direction which results in the greatest surge elevation at NMP. The results of the sensitivity analysis are presented below:

Wind Direction (degrees)	Set-up at NMP(ft)	Surge Elevation (ft; IGLD85)
310	4.1	251.4
300	4.5	251.8
290	4.7	252.0
280	4.8	252.1
270	4.7	252.0
260	4.7	252.0
250	4.5	251.8

The SSPP predicts a maximum still water level increase of 4.8 ft at NMP resulting from an extra-tropical cyclone with sustained maximum winds of 100 mph, a storm track resulting in the maximum winds occurring parallel to the long axis of the lake. This surge height corresponds to still water elevations of 252.1 ft (IGLD85).

A comparison of measured water levels to those predicted using the SSPP was also performed as part of the NMP flood re-evaluation and model validation in accordance with Section 5.5 of NUREG/CR-7046 (U.S NRC 2011). The storm surges from nine representative, historic storms were calculated using the SSPP and the results were compared to the measured surge elevations. The historic storms evaluated occurred on February 17, 2006, January 9, 2008, January 18, 2012, December 23, 2004, March 10, 2002, February 1, 2002, September 9, 2008, January 30, 2008 and February 10, 2001. Storm surge elevations at three of the four NOAA Tides and Currents stations along the south shore of Lake Ontario (Olcott, NY, Rochester, NY and Oswego, NY) were extracted from the filtered six-minute water level data. These measured water levels were compared with outputs from the SSPP. The locations of these water level stations relative to the 15 SSPP output points are shown in Figure 2.4-6. Each water level station and the SSPP output point used for verification are indicated below:

NOAA Water Level Station	NOAA SSPP Output Point
Station 9052076: Olcott, NY	2
Station 9052058: Rochester, NY	6
Station 9052030: Oswego, NY	10

Wind data for these storms were extracted from six NOAA National Climate Data Center (NCDC) stations around Lake Ontario, shown in Figure 2.4-7. The distribution of these stations around the lake provides representative winds around the entire lake. The maximum wind speed, and associated wind direction, from each of the six station's time-series was determined. These six winds speeds were then converted to hourly overwater wind speeds in mile per hour (mph) and used to determine a spatially averaged wind speed over the entire lake. For those stations with durations other than 60-minutes, the equations provided in the USACE CEM (Reiso 2008) were used to convert to hourly winds.

The comparison of the predicted to measured water levels for the nine representative extra-tropical storms is presented on Figure 2.4-8. The results show that using the spatially averaged constant wind field over the entire lake as input, the SSPP results reasonably and conservatively predict the surge elevation at Oswego (near NMP Unit 1 and Unit 2) when compared to measured water levels during the storm. A correlation coefficient between predicted and measured values of 0.95 was calculated. The collated trend line is linear and indicates some model bias to under predict surge elevation at low surge values and over predict surge elevation at moderate to high surge values.

2.4.2.5 Storm Surge Effects

NMP Unit 1 and Unit 2 are protected with a shore protection dike (Unit 1) and a revetment ditch system (Unit 2). The tops of the revetments dikes are at Elevation 262.3 ft (IGLD85). The ground surface elevations within the plant area behind the Unit 1 dike and the Unit 2 dike are slightly lower, ranging from about 258.3 ft to 259.3 ft (IGLD85) (C.T. Male, 1999). The predicted PMSS still water elevation is 252.1 ft (IGLD85). Therefore, the existing plant grades are above the predicted PMSS still water elevation and impact to NMP Unit 1 and Unit 2 structures from the rise in lake level during the surge is not expected. The area to the west of Unit 1 is not fronted with a revetment and portions of this area are low-lying, with elevations below the PMSS still water elevation. These areas will be flooded during the PMSS, without impact to Units 1 or 2. Wind-generated waves (which occur with the storm surge) will cause wave run-up to higher elevations than the PMSS surge elevation. The effects of wind-generated waves are discussed in Section 2.9.

2.4.3 Conclusions

Based on the PMSS calculation for the NMP Unit 1 and Unit 2, the following conclusions are reached:

- The controlling storm type is an extra-tropical storm.
- Per ANSI/ANS-2.8-1992, the antecedent water level is the regulated lake water level which is 247.3 ft (IGLD85).
- The predicted PMSS height is 4.8 ft.
- The predicted PMSS elevation is 252.1 ft (IGLD85).
- NMP Unit 1 and Unit 2 are protected with a shore protection dike (Unit 1) and a revetment dike and drainage ditch (Unit 2). The elevations of the top of the revetment dikes, and the plant grade behind the dikes are above the predicted PMSS still water elevation; therefore, impact to NMP Unit 1 and Unit 2 structures due solely to the surge water level are not predicted. The effects of wind-generated waves (which are combined with the storm surge) are discussed in Section 2.9, Combined Effect Flood.

Uncertainty and conservatism was considered in the calculation as per Section 5.4 of NUREG/CR-7046, as follows. The probable maximum wind storm (PMWS) input and output parameters, which are the basis for SSPP model used to calculate the PMSS, were adjusted to provide the most adverse conditions. The adjustments included:

- The PMWS storm track was smoothed and the translation speed of the storm was reduced to 40 mph (which was the minimum recorded storm speed) to increase the effect of the pressure gradients and resulting wind speeds,
- The predicted peak wind speed for the PMWS was increased to reflect a maximum over-water wind speed of 100 mph (as defined in ANSI/ANS-2.8-1992). This resulted in a 21% increase in the calculated peak wind speed determined for the synthetic storm used to evaluate the PMWS.

In analyzing the PMSS, the synthetic storm wind speeds and direction are conservatively assumed to be temporally and spatially constant which maximizes the storm surge. A sensitivity analysis was also performed on the peak wind speed direction by analyzing the surge elevations relative to varying wind directions, to determine the wind direction resulting in the maximum surge elevation at the site. The maximum predicted storm surge height of 4.8 feet resulted from a wind speed direction of 280 degrees. Validation of the surge model was performed by comparing predicted water levels to measured water levels for a number of historic lake storm surges; the validation indicates that the model over predicts surge elevations at moderate to high surge values.

2.4.4 References

NOTE: Refer to the Project Manager's approval (on the signature page of this report) verifying that the Constellation Nuclear Energy Group (CENG) references are valid sources of design input created in accordance with the CENG's QA program.

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Table 2.4-1: Summary of Hurricane Parameters

Name	Year	Month	Day	Latitude	Longitude	Direction [degrees]	Forward Speed [mph]	Wind Speed [mph]	Pressure [mb]	Storm Type
N/A	1878	September	13	44.0N	78.5W	10	28	50	n/a	ET
N/A	1893	October	14	42.7N	77.6W	5	46	70	n/a	TS
N/A	1901	September	29	44.2N	76.5W	40	25	30	n/a	ET
N/A	1903	September	17	43.0N	77.0W	335	13	45	n/a	ET
N/A	1915	August	22	43.5N	79.0W	55	17	30	n/a	ET
N/A	1923	October	24-25	43.6N	76.9W	N/A	19.4-22.5	33.4-39	n/a	ET
N/A	1926	August	2	44.0N	78.8W	50	19	30	n/a	ET
Hazel	1954	October	16	45.2N	78.6W	350	48	70	n/a	ET
Audrey	1957	June	29	43.7N	77.1W	35	58	60	n/a	ET
Hugo	1989	September	23	42.2N	80.2W	20	43	40	988	ET
Opal	1995	October	6	43.3N	78.4W	50	23	40	997	ET
Fran	1996	September	8	43.4N	79.9W	15	6	35	999	TD
Dennis	1999	September	8	43.5N	76.5W	90	9	25	1006	ET
Isabelle	2003	September	19	43.9N	80.9W	350	34	35	1000	ET
Frances	2004	September	9	42.8N	77.7W	35	32	40	1001	ET
Katrina	2005	August	31	40.1N	82.9W	50	26	30	996	ET
Ernesto	2006	September	3	43.1N	77.5W	350	20	25	1014	ET

Note 1: Data was selected according to HURDAT data from the latitude and longitudinal coordinates with closest proximity to Lake Ontario.

Note 2: Wind speed is the 1-minute, 10-meter wind speed

Note 3: ET indicates Extra-tropical; TS indicates Tropical Storm and TD indicates Tropical Depression

Table 2.4-2: Top 20 Storm Surges on Lake Ontario (1961 to 2012)

Rank					Cape Vincent			Oswego			Rochester			Olcott		
	Year	Month	Day	Wind Speed [mph]	Hour	setup (m)	(ft)	Hour	setup (m)	(ft)	Hour	setup (m)	(ft)	Hour	setup (m)	(ft)
1	1992	11	13	n/a	7	0.627	2.06	6	0.341	1.12	6	0.009	0.03	6	-0.251	-0.82
2	2006	2	17	80.6	14	0.572	1.88	14	0.308	1.01	14	-0.155	-0.51	14	-0.223	-0.73
3	1979	4	6	n/a	10	0.553	1.81	10	0.207	0.68	10	-0.107	-0.35	10	-0.150	-0.49
4	1979	4	6	n/a	16	0.510	1.67	20	0.334	1.10	20	-0.062	-0.20	20	-0.201	-0.66
5	2008	1	30	59.8	14	0.490	1.61	13	0.201	0.66	13	-0.113	-0.37	13	-0.239	-0.78
6	2012	1	18	59.8	4	0.480	1.57	4	0.203	0.67	4	-0.036	-0.12	4	-0.175	-0.57
7	2002	2	1	63.3	20	0.437	1.43	21	0.216	0.71	21	-0.006	-0.02	21	-0.573	-1.88
8	1991	12	15	57.5	0	0.432	1.42	23	0.280	0.92	22	0.090	0.30	23	-0.211	-0.69
9	1997	2	22	70.2	17	0.430	1.41	16	0.257	0.84	16	-0.041	-0.14	16	-0.169	-0.55
10	2003	11	13	65.6	13	0.426	1.40	17	0.252	0.83	17	0.001	0.00	17	-0.159	-0.52
11	2002	3	10	65.6	5	0.400	1.31	3	0.107	0.35	3	-0.303	-1.00	3	-0.111	-0.37
12	2001	2	10	76	9	0.397	1.30	8	0.222	0.73	8	0.036	0.12	8	-0.146	-0.48
13	1972	1	25	n/a	17	0.380	1.25	16	0.227	0.75	16	0.013	0.04	16	-0.160	-0.52
14	2012	1	29	n/a	1	0.373	1.22	0	0.115	0.38	0	-0.042	-0.14	0	-0.126	-0.41
15	2006	12	2	67.9	6	0.369	1.21	5	0.157	0.51	5	-0.021	-0.07	5	-0.076	-0.25
16	1965	11	17	n/a	9	0.368	1.21	10	0.123	0.40	14	0.104	0.34		(no data)	
17	1967	2	16	n/a	12	0.360	1.18	16	0.242	0.79	15	0.055	0.18	18	0.008	0.02
18	2008	1	9	64.4	15	0.358	1.17	14	0.124	0.41	14	0.093	0.31	14	-0.154	-0.50
19	2012	2	25	59.8	4	0.357	1.17	4	0.170	0.56	4	0.009	0.03	4	0.009	0.03
20	1975	11	10	72.5	21	0.355	1.17	21	0.132	0.43	21	-0.114	-0.38		(no data)	

Note: Wind Speed is the maximum 1 minute sustained wind speed or gust.

Table 2.4-3: PMWS Pressure, Wind Speed, and Wind Direction on Lake Ontario

	Z1				Z2				Z3				
Time (hour)	P1 (mb)	S1 (mph)	S1 (kph)	D1 (deg)	P2 (mb)	S2 (mph)	S2 (kph)	D2 (deg)	P3 (mb)	S3 (mph)	S3 (kph)	D3 (deg)	P4 (mb)
0	971	50	81	120	973	60	96	130	977	44	72	130	979
1	969	52	84	110	972	49	80	110	975	46	74	110	977
2	967	55	89	100	970	49	79	100	973	46	74	90	975
3	965	51	81	110	968	50	80	110	971	47	75	100	973
4	964	44	71	130	966	50	80	120	969	48	78	120	971
5	962	41	66	130	965	47	75	120	967	45	73	110	969
6	961	43	69	110	963	42	68	110	966	44	70	100	967
7	959	45	73	110	962	42	68	110	964	38	60	100	966
8	958	44	71	110	960	45	73	110	963	37	60	110	964
9	956	44	71	100	958	46	74	110	961	39	62	110	963
10	954	46	74	90	957	47	76	110	960	41	66	110	961
11	953	37	60	120	955	48	77	140	958	40	64	140	960
12	952	29	47	140	954	46	73	140	956	41	66	160	958
13	951	27	43	130	953	39	62	160	955	43	69	160	957
14	951	25	40	210	952	28	44	190	954	36	58	180	955
15	951	34	54	190	951	34	55	185	953	27	43	180	954
16	954	58	94	300	951	36	57	190	953	28	45	190	953
17	958	76	122	290	953	36	58	280	952	30	49	190	953
18	961	100	161	300	956	66	106	300	952	47	76	200	950
19	965	95	152	300	960	100	161	300	954	94	151	290	950
20	968	94	152	300	963	83	134	300	958	100	161	290	954
21	971	89	143	300	966	87	141	300	961	84	135	290	958
22	974	81	130	300	970	90	145	300	964	82	131	300	961
23	976	73	117	300	972	88	142	300	967	83	134	300	964
24	978	64	103	300	975	75	121	300	971	84	136	300	967
25	980	56	90	290	977	67	108	290	973	65	104	290	971
26	982	52	84	290	979	62	100	290	976	62	99	290	973
27	983	44	71	300	981	58	93	290	978	58	94	290	975
28	985	39	62	300	983	49	79	300	980	57	91	300	978
29	986	38	61	300	984	38	61	300	982	52	84	300	980
30	987	34	54	300	986	38	62	300	983	36	58	300	982
31	988	30	49	310	987	37	59	300	985	35	56	300	983
32	989	29	46	330	988	29	47	310	986	36	58	300	985
33	990	28	46	350	989	29	47	325	987	27	43	300	986
34	991	26	41	340	990	28	45	320	988	24	39	300	987
35	992	23	38	340	990	28	44	340	989	24	38	320	988
36	992	23	38	340	991	24	39	340	990	22	35	320	989

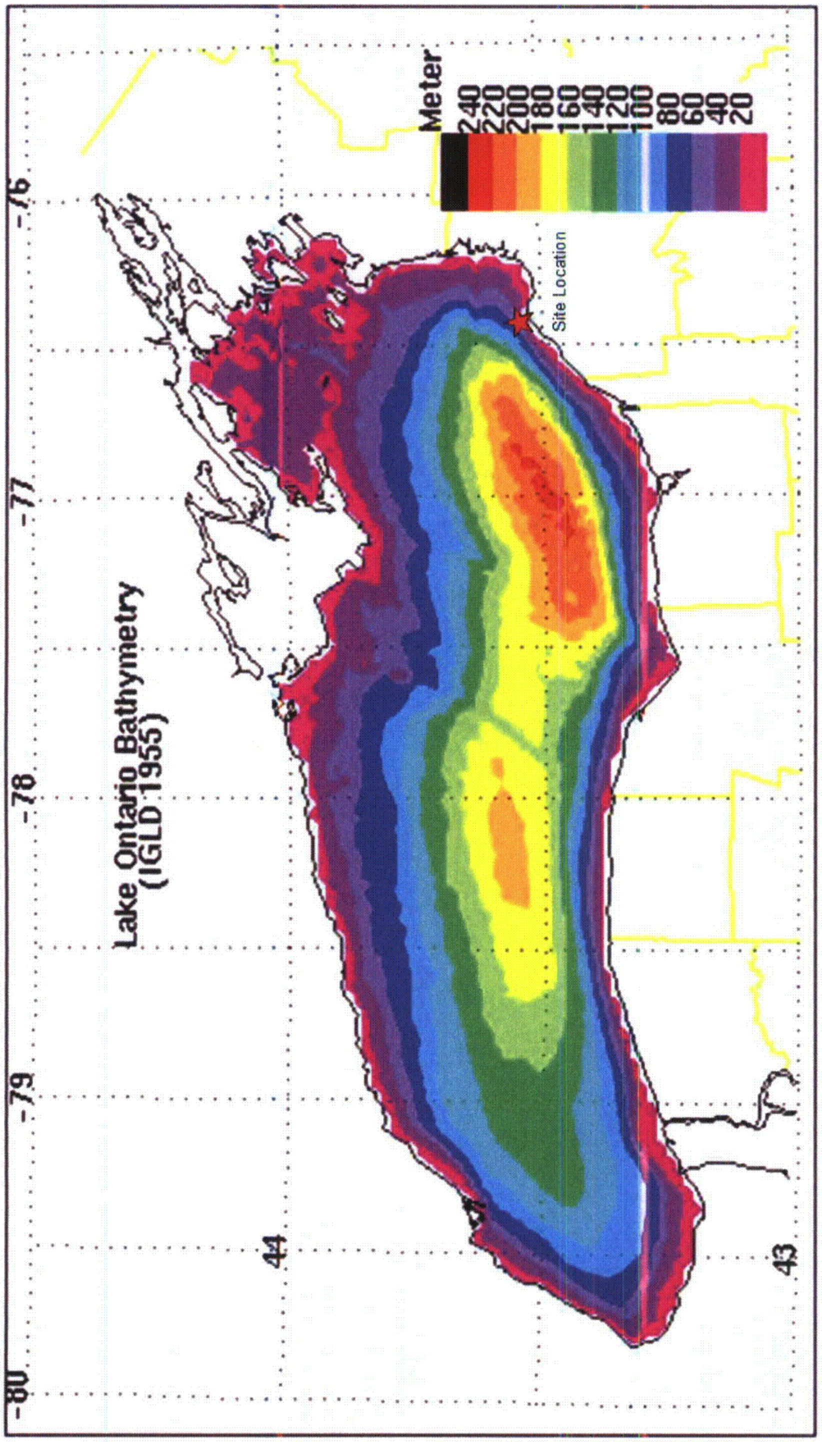

Figure 2.4-1: Lake Ontario Bathymetry

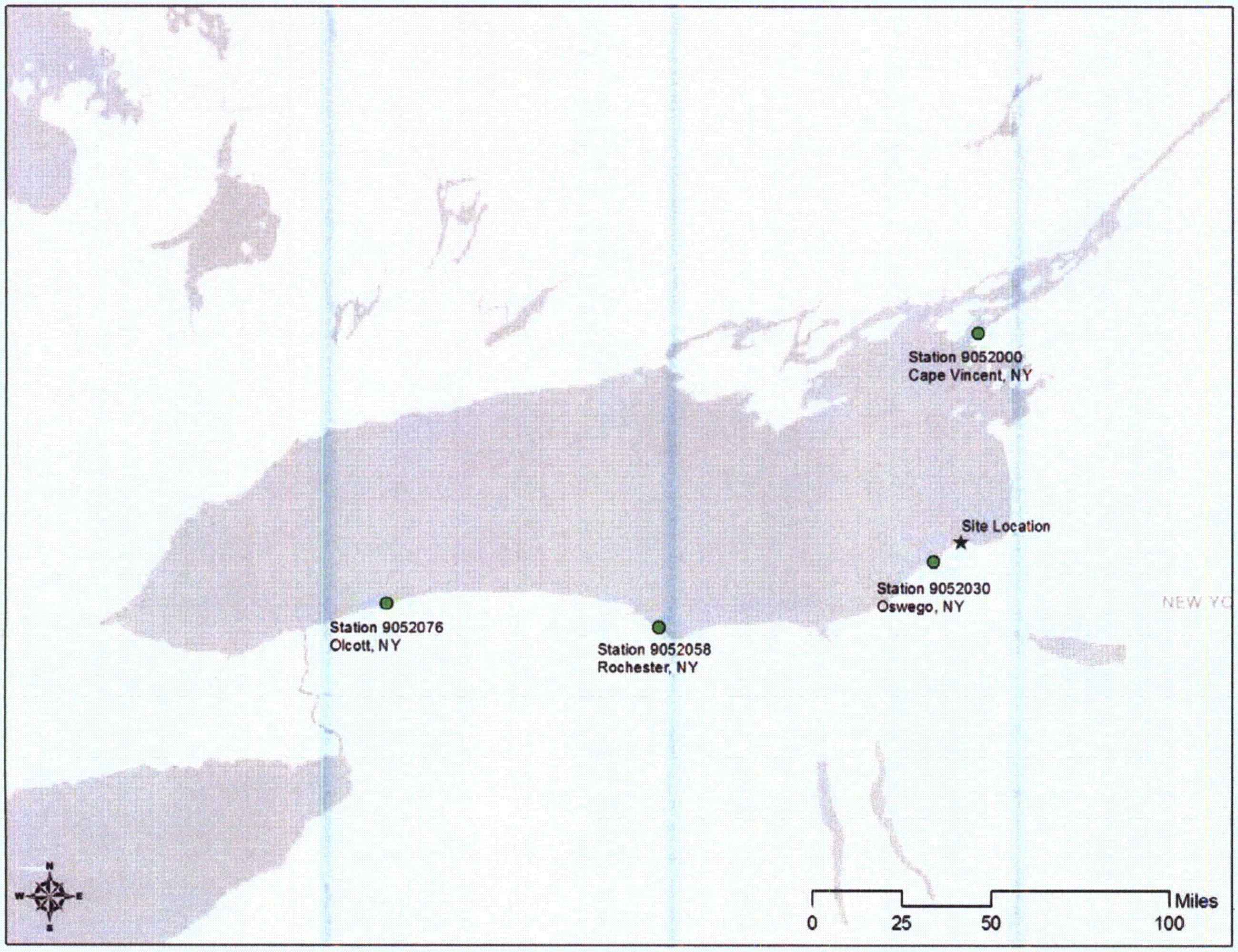

Figure 2.4-2: Locations of NOAA Water Level Stations

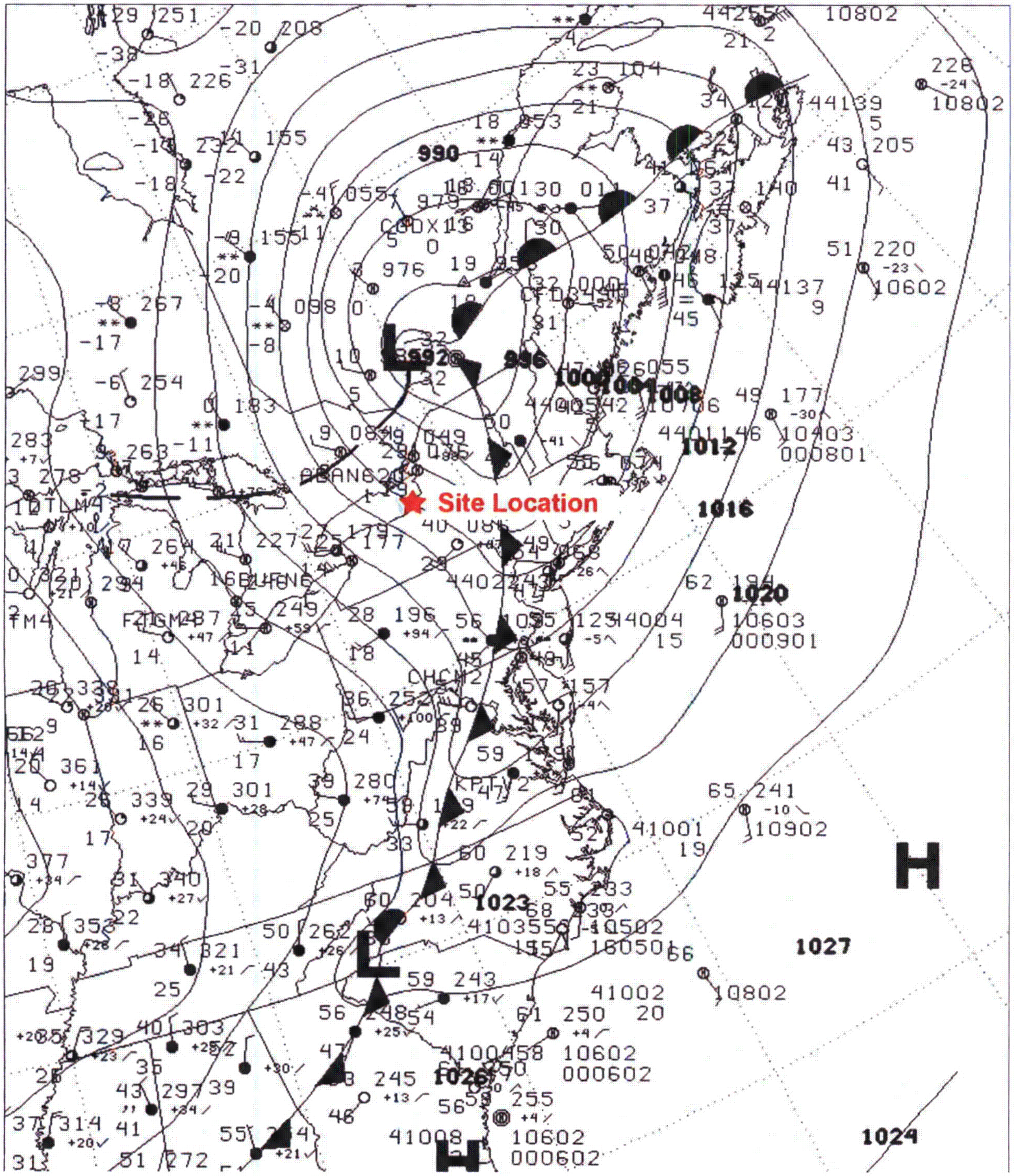

Figure 2.4-3: Pressure Map from the February 17, 2006 Design Storm

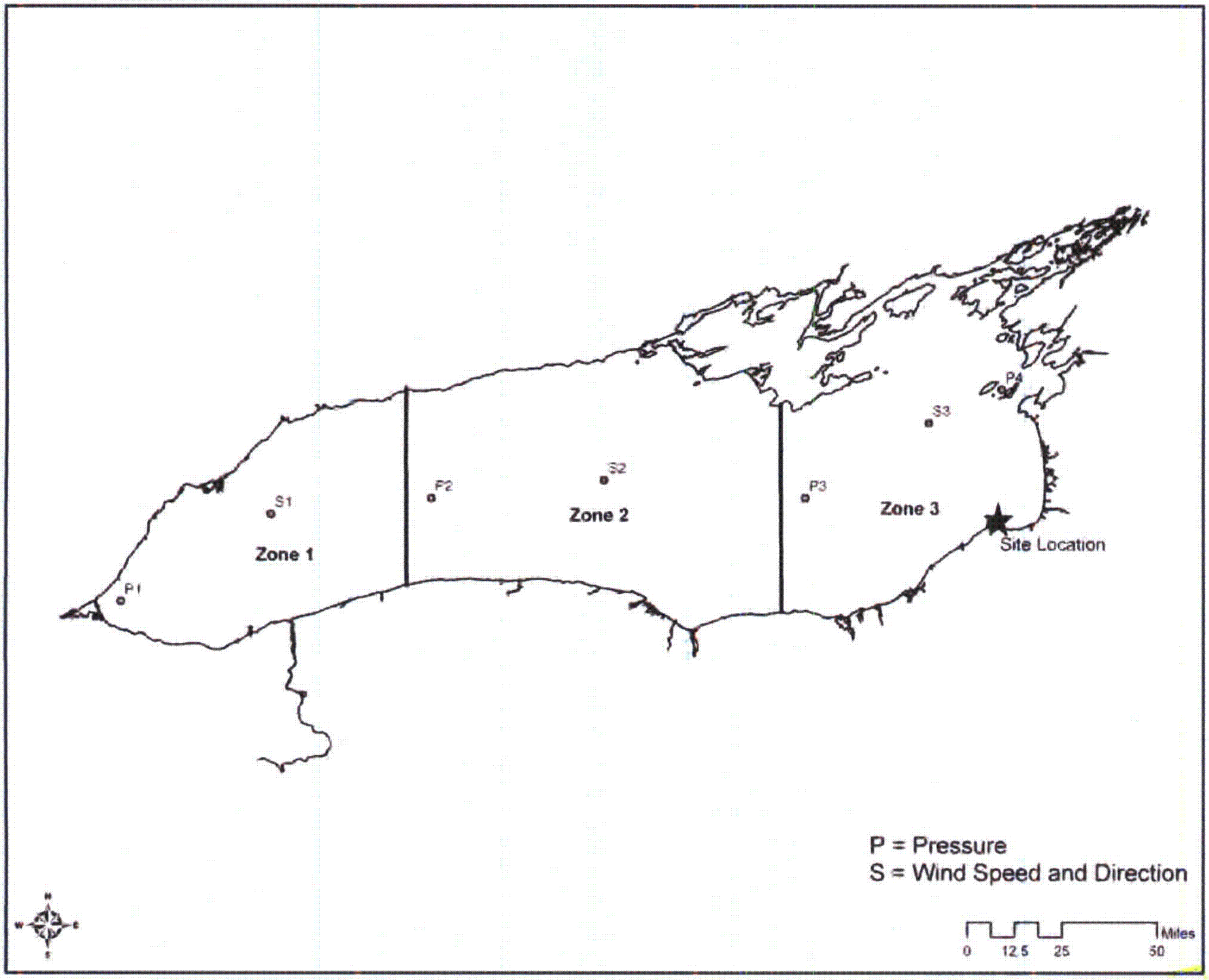

Figure 2.4-4: Zones used to Develop the PMWS Parameters

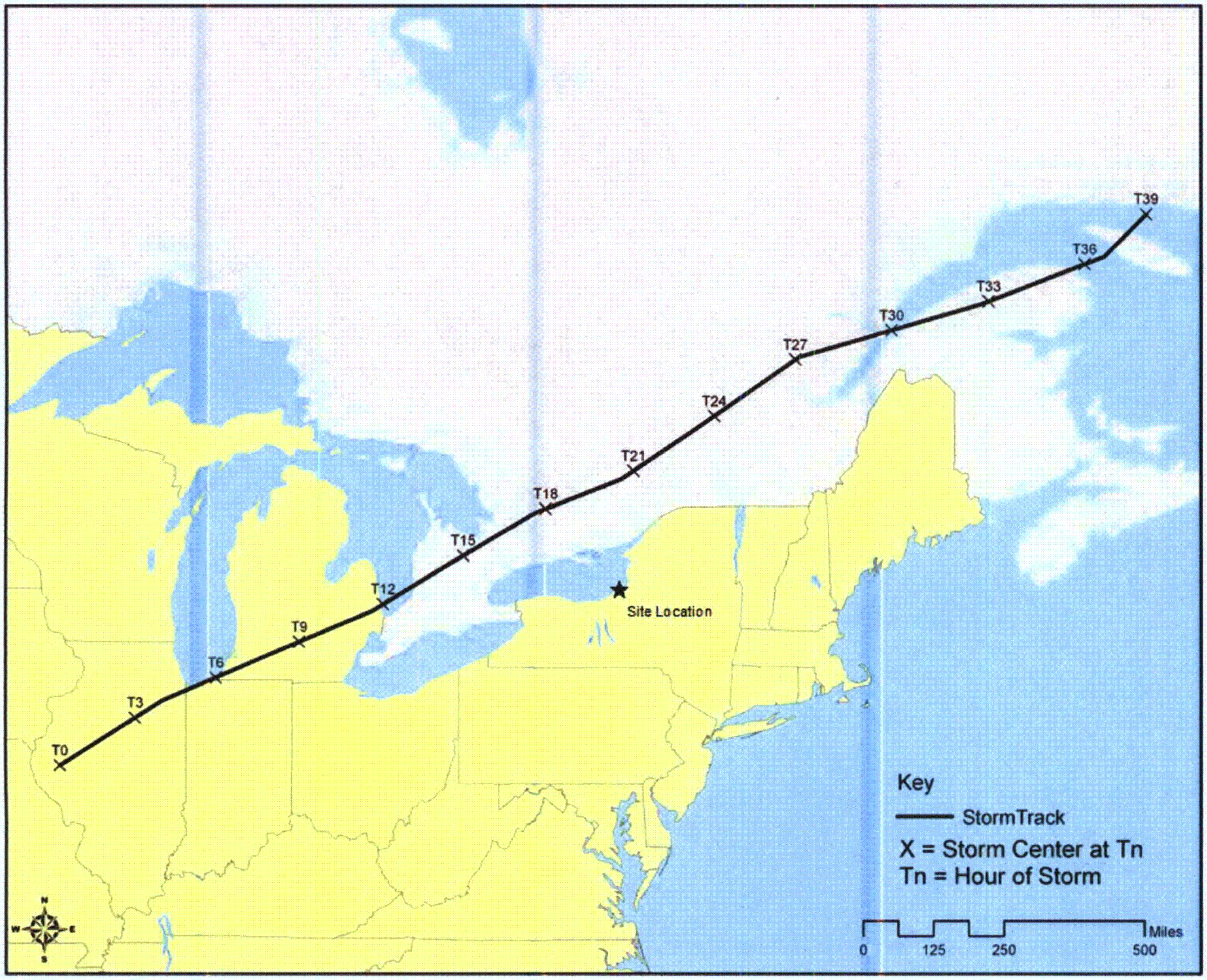

Figure 2.4-5: PMWS Simulated Track Direction

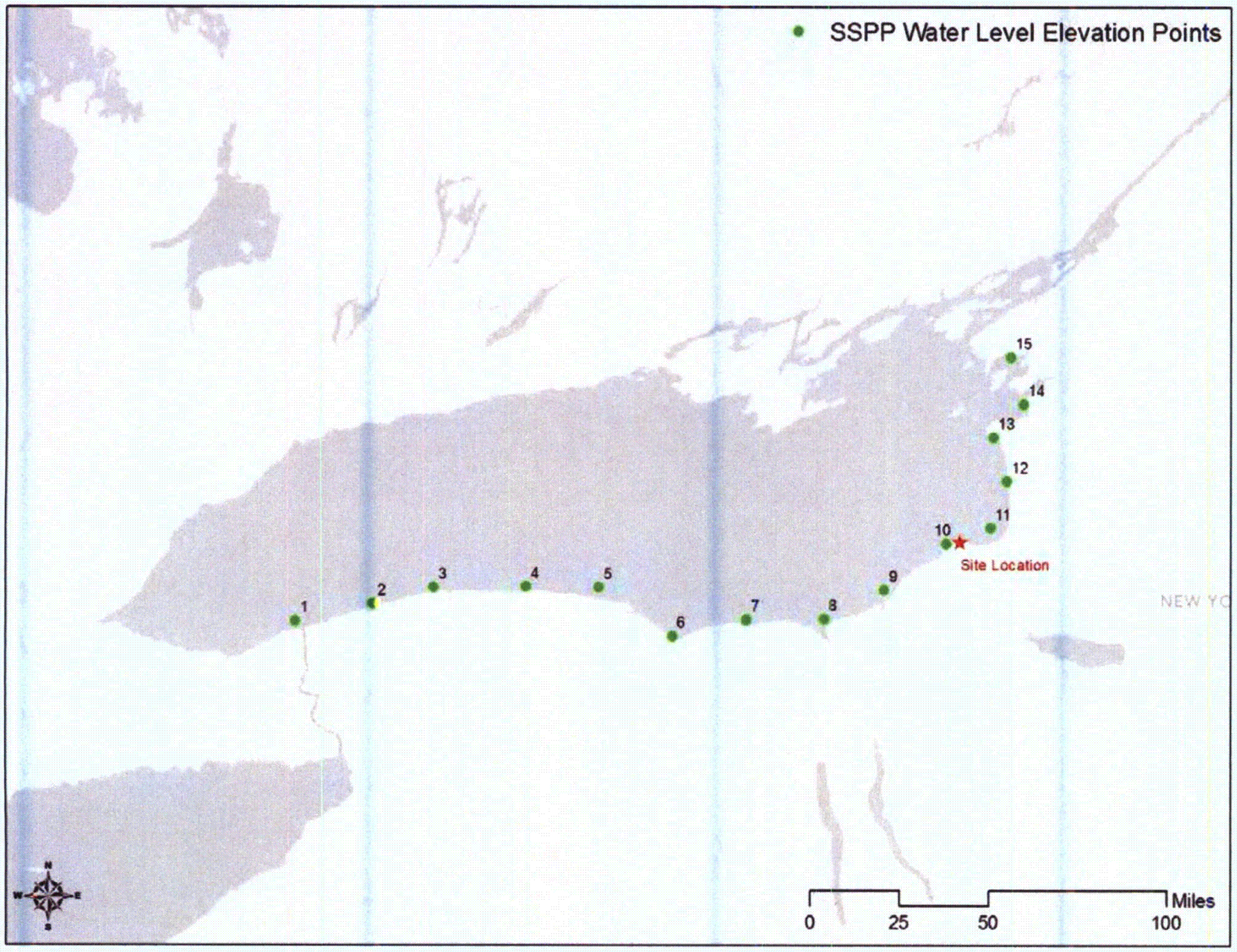

Figure 2.4-6: SSPP Model Output Locations for Lake Ontario

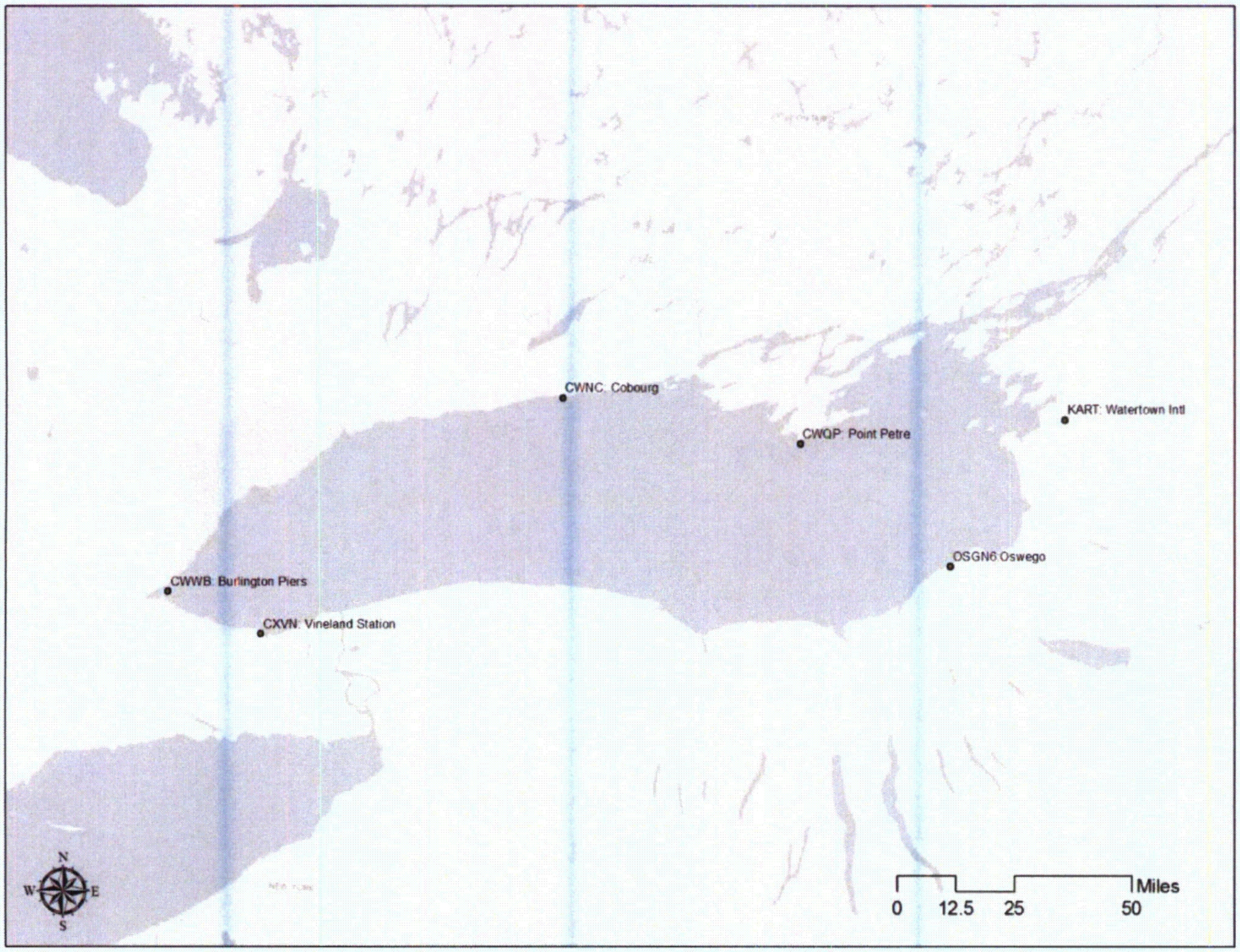

Figure 2.4-7: NOAA NCDC Weather Stations (for Wind Velocity Data)

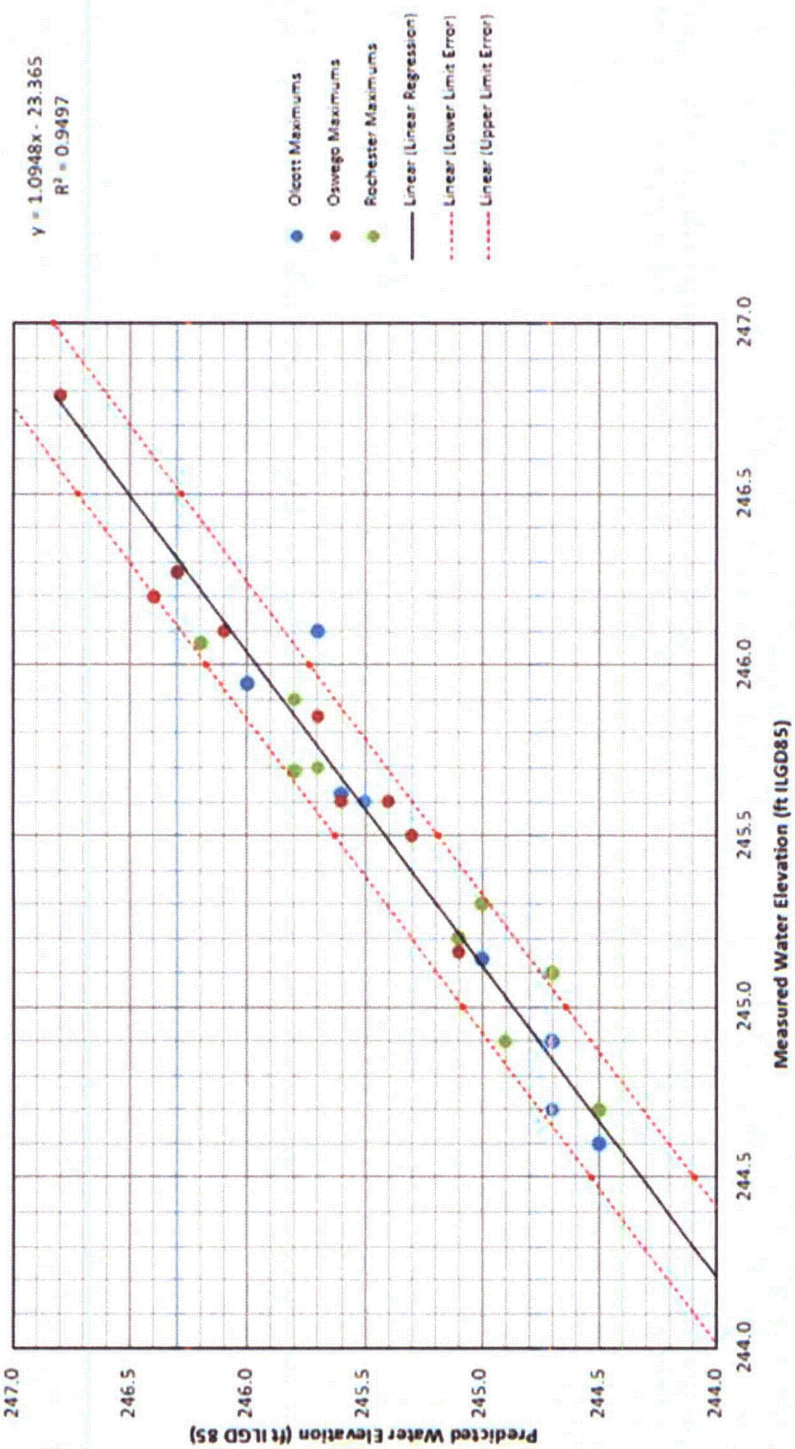

Figure 2.4-8: SSPP Model Verification for Nine Representative Storms

2.5 Seiche

2.5.1 Methodology

Enclosed basins such as cooling reservoirs, ponds or lakes are not present at NMP Units 1 and 2; therefore, an evaluation of these types of basins is not required. However, due to the coastal setting of NMP on the shore of Lake Ontario, evaluation of seiches occurring on Lake Ontario and their potential impact to NMP Units 1 and 2 was performed. This report section addresses seiches due to meteorological external forcing. Seiches can also result from lake excitations due to earthquakes and landslides (see Section 2.6 Tsunamis for discussion of these types of events).

The HHA approach described in NUREG/CR-7046 (NRC 2011) was used to determine whether a seiche in Lake Ontario can result in significant flooding at NMP Units 1 and 2. This approach initially involves the determination of the natural period of the lake, evaluation of the natural oscillation periods of the external forces, such as tropical and extra-tropical storms, comparison of the periods to determine if resonance is possible, and review of water level data to evaluate potential seiche heights. Per the HHA, more detailed analysis (including numerical models) of seiches may be required if resonance is expected and if there is little margin between the site grade and the seiche elevation.

The NMP seiche evaluation methodology included the following steps:

- determination of the natural period of Lake Ontario at Oswego based on performance of a statistical analysis of water level data and review of published results of statistical and numerical analyses of Lake Ontario;
- identification of the periods of meteorological external forcing events (e.g., extra-tropical storms) based on statistical analysis of wind data, and comparison of the external forcing and lake periods to determine if resonance is expected; and
- evaluation of potential seiche heights at NMP based on review of recorded oscillation water levels and the predicted storm surge heights.

2.5.2 Results

2.5.2.1 Determination of the Natural Period of Lake Ontario

The natural period of the lake is primarily a function of its geometry and basin depth and is independent of external forcing mechanisms. Natural periods can range from tens of seconds to several hours (Rabinovich, 2009). Research by Hamblin, Li and Simpson indicate that the natural period of oscillation for Lake Ontario is approximately five hours (Hamblin 1982, Li et al. 1975 and Simpson et al. 1964). Numerical modeling results of Rao and Schwab presented in the Army Corps of Engineers Coastal Engineering Manual (USACE 2008) indicate that the periods of the six lowest modes in Lake Ontario are 5.11, 3.11, 2.13, 1.87, 1.78 and 1.46 hours.

For a given basin, seiche periods can be extracted from observations, modeled, or calculated by statistical analyses or by Merian's formula. Merian's formula provides an approximation of the natural period based on the length and depth of an enclosed rectangular basin/waterbody (Scheffner, 2008 and

Hamblin, 1982). Since Merian's formula is an approximation for a rectangular waterbody and because sufficient water level data for extracting the seiche period exists for Lake Ontario, a statistical analysis was performed to confirm the fundamental period of the lake at Oswego (near NMP).

To confirm the fundamental oscillation period, also known as the fundamental mode, on Lake Ontario near Oswego, a spectral analysis of the NOAA six-minute water level data at the Oswego, New York (Station 9052030) was performed. The location of the Oswego gage is shown in Figure 2.5-1 along with other NOAA water level gage stations. The fundamental mode is the mode with the lowest frequency and thus the longest period (Rabinovich 2009). The spectral analysis was carried out on the longest continuous portion of the water level data (6667 days) using the software Matlab™ (Release 2011b). The analysis was performed by applying a discreet Fast Fourier Transform (FFT) with no normalization of the output. The results of the spectral analysis are presented in Figures 2.5-2 and 2.5-3. The bottom x-axis shows the frequencies present in the records; the top x-axis shows the corresponding period. The y-axis shows the relative power of each spectral peak. The three lowest periods (5.11, 3.11, 2.13 hours) are highlighted with vertical bars.

The annual variation of lake levels, indicated by the spectral peak at 365 days in Figure 2.5-2, indicates that the annual variation clearly overwhelms short-term variations such as surges and seiches. The annual variation is due to the annual hydrologic cycle and regulatory releases of water to control Lake Ontario water levels, and causes higher water levels in the spring and early summer with the highest water levels typically occur during June on Lake Ontario (Wilcox et al. 2007). As illustrated in Figure 2.5-3, the calculated spectral peak at approximately five hours is consistent with the fundamental period presented in the literature.

2.5.2.2 Evaluation of External Forcing Events and Observed Oscillations

On Lake Ontario, the recorded seiches with the largest amplitudes are caused by long period, non-convective extra-tropical storms with winds blowing parallel to the long access of the lake causing a setup at the downwind end of the lake and a corresponding water level setdown at the upwind end of the lake (U.S. ACE 2008, FEMA 2012 and Rabinovich 2009). The tracks of these storms are typically in a west to east (or southwest to northeast) direction and occur during the Late Fall and Winter months. Seiches can also be caused by other storm events, including tropical storms, changes in barometric pressure, frontal zones and short-period convective storms (squall lines with high winds, and thunderstorms). Cessation of the external force causes periodic water level fluctuations as the standing wave reflects from the ends of the lake (Melby et al. 2012).

External forcing consisting of moving disturbances, such as wind storms and wind squalls, initially cause storm surges (due to wind set-up at one end of the lake and set-down at the other end), followed by a series of oscillations (standing waves) which can be both forced (during the period of strong winds over the lake) and free (once the storm has passed). As noted above, the most significant recorded storm surges and seiches at Oswego have occurred due to long period, non-convective extra-tropical storms. If the frequency of the disturbance is different than the fundamental frequencies of the lake, the amplitude of the seiches decay fairly rapidly (over a period of a few days) due to bottom friction as the seiche oscillates between opposing shorelines. Water level data associated with storms that have resulted in large surges and seiches at Oswego illustrate the behavior described above. Water level plots were prepared from the NOAA six-minute water level measurements at the Oswego and Cape Vincent gages (NOAA 2012b) for nine storms that caused significant storm surges at the Oswego water level station between 2001 and 2012. The water level plots are presented in Figures 2.5-4 through 2.5-

12. Figure 2.5-4 presents water level data for the February 17, 2006 storm which resulted in one of the largest recorded storm surges at Oswego. As shown on this figure, an initial storm surge is observed at the Oswego and St. Vincent stations, which is followed by a series of oscillating waves occurring at the fundamental period of the lake (about every 5 hours). However, the lake system is "under-damped" and the amplitude of the oscillating waves are typically less than the initial storm surge and decrease over time. The other storms exhibit similar behavior.

To further investigate the periods of potential seiche forcing events, i.e. wind storms, a frequency analysis of wind data on Lake Ontario was performed. Surface wind data at the Rochester Airport (ROC) were compiled from 1930 to 2012. The ROC wind data is the longest and most complete wind record on the south shore of Lake Ontario. The ROC data provides 2-minute duration wind speeds which are sampled at a minimum of 1 hour intervals. This dataset was subset to cover the same time period as the six-minute water level data from the Oswego station used for the frequency analysis of water level data (1996 – 2012). A frequency analysis of the wind data was performed by applying a FFT to the wind speed time series data (similar to the frequency analysis of the water levels), to identify the fundamental frequencies in the wind record.

The results of the statistical analysis are presented in Figures 2.5-13 and 2.5-14. Peaks in the power spectrum for the wind speed power spectra can be found at 12 and 24 hours which correlate to similar peaks in the water level power spectra at those periods (Figures 2.5-2 and 2.5-3). These periods are greater than the higher order seiche periods of Lake Ontario and therefore, are not associated with seiches. Peaks associated with wind forcing at the most energetic modal seiche periods at 5.1, 3.2 and 2.3 hours (Hamblin, 1982) of Lake Ontario are not present in the frequency analysis of the wind speed data (Figures 2.5-13 and 2.5-14). A similar analysis of the full ROC record (1930-2012) showed peaks in the wind speed power spectra at the same periods. The analysis of the data shows that principal wind forcing does not display a spectral peak at either the primary, secondary, or higher order seiche periods thus precluding the likelihood that resonance will occur as a result of these wind disturbances.

As noted above, the recorded seiches with the largest amplitudes on Lake Ontario are caused by long period, non-convective extra-tropical storms that track from west to east (or southwest to northeast) direction and have strong winds that blow out of the west to northwest, parallel to the long access of the lake and cause a setup at the east end of the lake and a corresponding water level set-down at the west end of the lake. Seiches could also occur in the vicinity of NMP due to storms that do not cause an initial storm surge at NMP (although review of the water level data did not identify this condition associated with the highest recorded water levels at the Oswego station). For example, a tropical system tracking in a northerly direction across the lake could result in strong winds blowing from the east across the lake. The initial seiche response to that forcing will occur on the east side of the lake, in alignment with the first normal mode of oscillation in Lake Ontario. Analyses by Hamblin, including a spectral analysis of water level data and a numerical finite element model, indicates the following (Hamblin 1982). The node of the primary mode is located near the center of the lake and the seiche response near Oswego (at the east end of the lake) is similar to that of Niagara (at the west end of the lake). While the mechanics of seiche on Lake Ontario are not symmetrical due to variability in the lake bathymetry), the amplitudes at the western and eastern ends of the lake are not substantially different. The similarity between these amplitudes indicates that forcing opposite to the predominant direction (e.g., from the east) will not cause significantly different seiche impacts than forcing from the predominant direction (e.g. from the west).

2.5.2.3 Evaluation of Potential Seiche Heights

Seiches of significant amplitude are not a unique coastal hazard on Lake Ontario, since (as described above) their occurrence is a result of the oscillating response to a storm surge. Also, resonance between the principal forcing wind events and the lake is not expected. For these reasons, seiche amplitudes are not expected to be greater than the initial storm surge.

Table 2.5-1 presents the surge heights for the nine storms storm surges illustrated in Figures 2.5-4 through 2.5-12, based on 6-minute water level measurement data from the NOAA water level stations on Lake Ontario. The initial oscillation amplitudes measured from the water level plots is also presented. The estimated initial oscillation amplitudes were all less than the initial storm surge, with the subsequent oscillation amplitudes generally decreasing over time.

The predicted Probable Maximum Storm Surge (PMSS) resulting from the PMWS is presented in Section 2.4 and is 4.8 ft. Oscillations resulting from the PMWS surge are expected (similar to the recorded water level data) to have amplitudes less than 4.8 ft.

2.5.2.4 Seiche Effects

Seiche amplitudes are not expected to be greater than the initial storm surge and, therefore, are not expected to be the controlling flood event at NMP. The predicted Probable Maximum Storm Surge (PMSS) resulting from the PMWS is presented in Section 2.4 and is 4.8 ft. Oscillations resulting from the PMWS surge are expected (similar to the recorded water level data) to have amplitudes less than 4.8 ft. As illustrated in Figures 2.5-4 through 2.5-12, the result of the initial storm surge and follow-up oscillations is an elevated water level that lasts for a period of several days as the oscillation amplitudes diminish.

2.5.3 Conclusions

Based on the seiche evaluation for NMP Unit 1 and 2, the following conclusions are reached:

- Lake seiches of significant amplitude are not a unique coastal hazard on Lake Ontario, since their occurrence is a result of the oscillating response to a storm surge.

- On Lake Ontario, the recorded seiches with the largest amplitudes are associated with long period, non-convective extra-tropical storms with winds blowing parallel to the long access of the lake causing a set-up at the downwind end of the lake and a corresponding water level set-down at the upwind end of the lake. The tracks of these storms are typically in a west to east (or southwest to northeast) direction and occur during the Late Fall and Winter months.

- The fundamental oscillation period of Lake Ontario at Oswego is approximately 5.11 hours, as determined during this study by spectral analysis of water level data and by spectral and model analysis by others. The periods of the next five lowest modes in Lake Ontario are 3.11, 2.13, 1.87, 1.78 and 1.46 hours.

- Observations of the water level data indicate that the initial storm surges (due to wind set-up at one end of the lake and set-down at the other end) are followed by a series of oscillations (standing waves) which can be both forced (during periods of high winds) and free (once the storm has passed and the winds have subsided). The observed period of oscillations with significant amplitude is about 5 hours, consistent with the fundamental period of the lake. The amplitudes of the oscillations are observed to decrease with time due to friction.

- Resonance between the principal forcing wind events and the lake is not expected.
- Seiches could also occur in the vicinity of NMP due to storms (e.g., a northerly tracking tropical storm) that do not cause an initial storm surge at NMP (although review of the water level data did not identify this condition associated with the highest recorded water levels at the NOAA Oswego station). Spectral analysis of water level data and numerical modeling indicate that seiche amplitudes are similar on both ends of the lake, indicating that forcing opposite to the predominant direction (e.g., from the east) will not cause significantly different seiche impacts than forcing from the predominant direction (e.g., from the west) (AREVA, 2013).
- For these reasons, seiches are not expected to be the controlling flood event at NMP. The predicted Probable Maximum Storm Surge (PMSS) resulting from the PMWS is presented in Section 2.4 and is 4.8 ft. Oscillations resulting from the PMWS surge are expected to have amplitudes less than 4.8 ft.

2.5.4 References

NOTE: Refer to the Project Manager's approval (on the signature page of this report) verifying that the Constellation Nuclear Energy Group (CENG) references are valid sources of design input created in accordance with the CENG's QA program.

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Table 2.5-1: Comparison of Surge and Seiche Heights for Nine Representative Storms with High Recorded Surges on Lake Ontario

Date	Initial Storm Surge Amplitude (Oswego Station)	First Oscillation Amplitude
2/10/2001	1.10 ft	0.50 ft
2/1/2002	1.31 ft	1.30 ft
3/9/2002 - 3/10/2002	1.31 ft	0.95 ft
12/24/2004	1.38 ft	0.60 ft
2/17/2006	1.74 ft	0.65 ft
1/9/2008	1.25 ft	1.20 ft
1/30/2008	1.18 ft	0.25 ft
9/14/2008 - 9/15/2008	1.22 ft	0.90 ft
1/17/2012	1.28 ft	0.32 ft

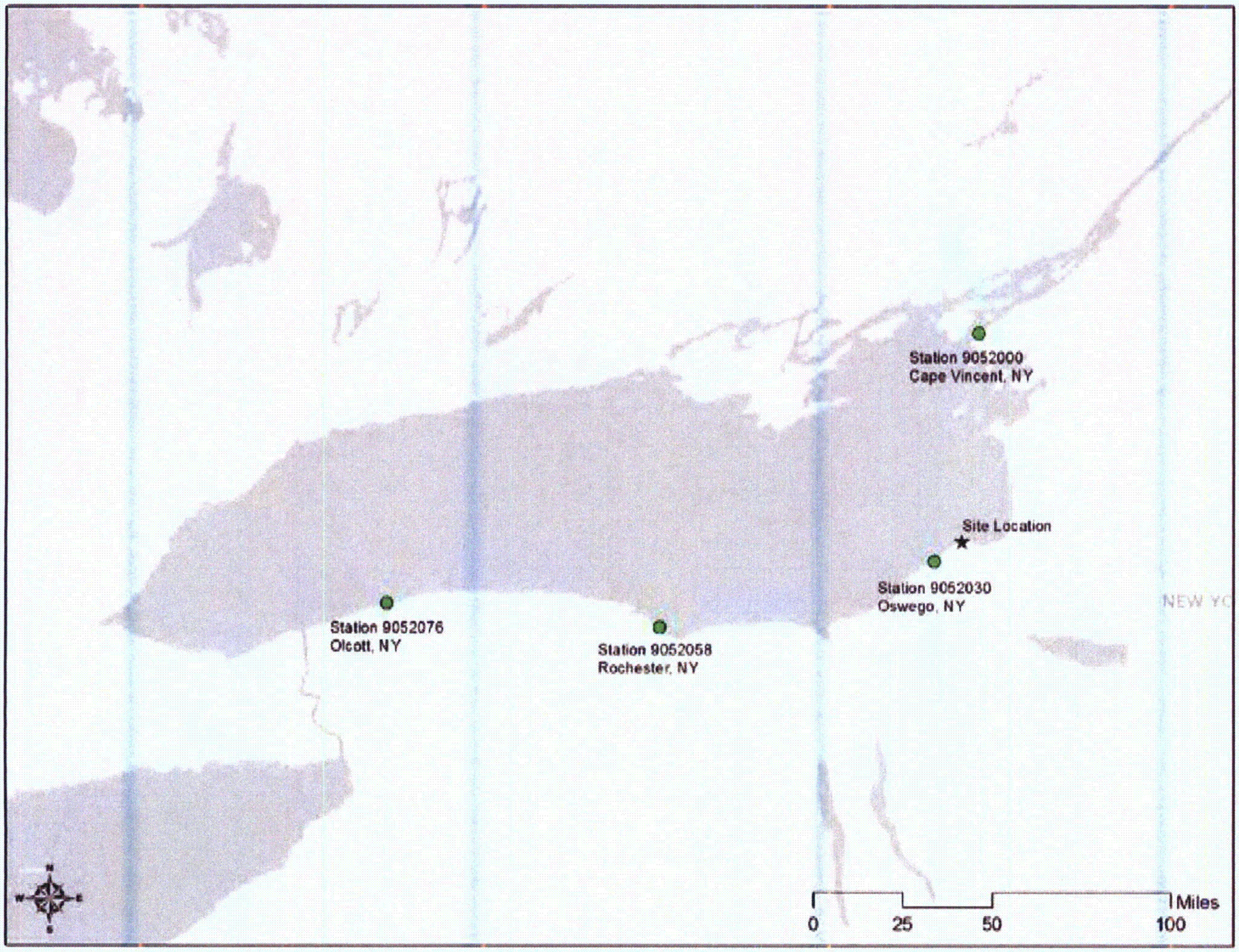

FIGURE 2.5-1: NOAA WATER LEVEL STATIONS ON LAKE ONTARIO

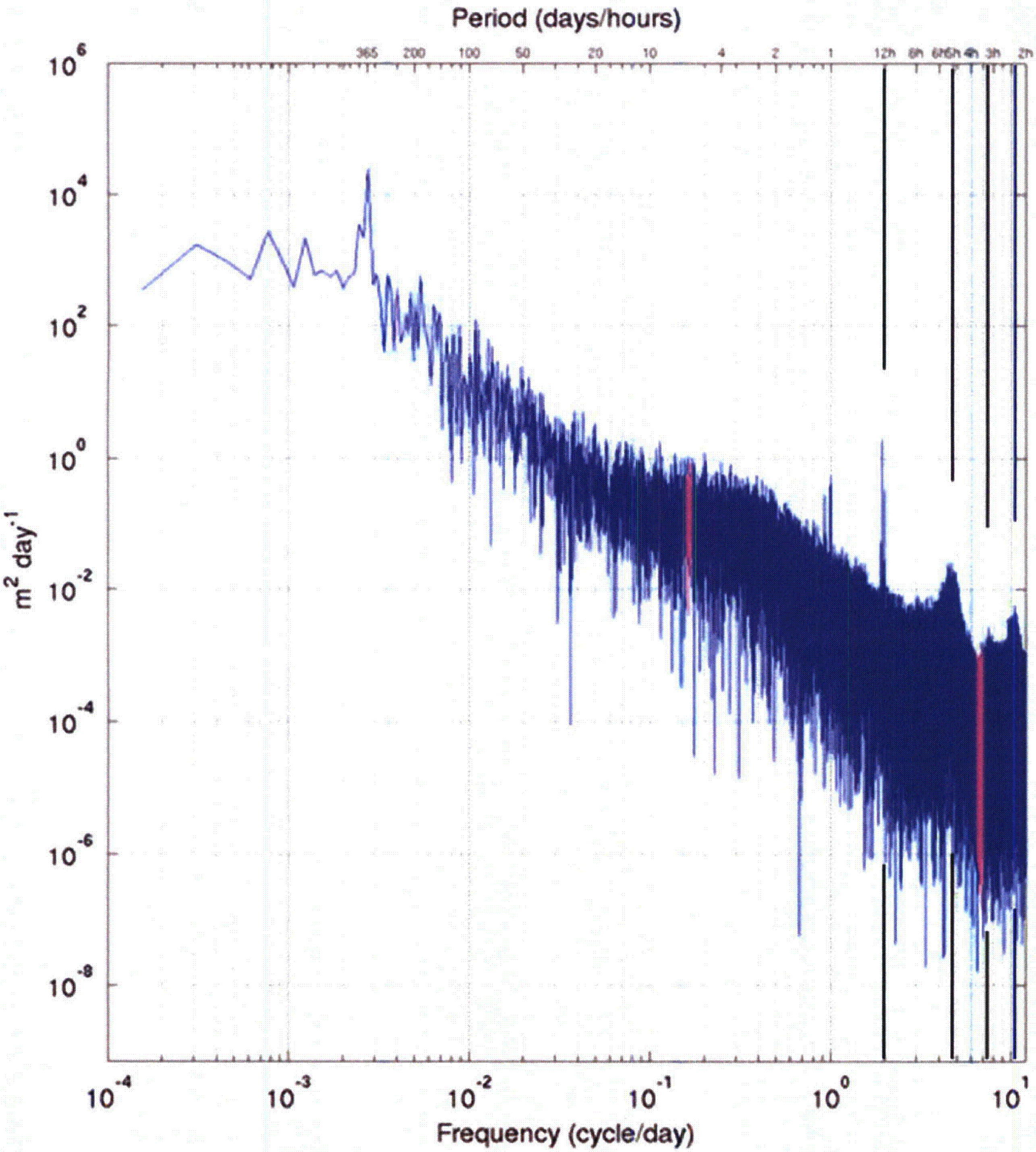

FIGURE 2.5-2: POWER SPECTRUM ANALYSIS OF OSWEGO, NY WATERLEVELS FOR PERIODS FROM 2 HOURS TO 365 DAYS

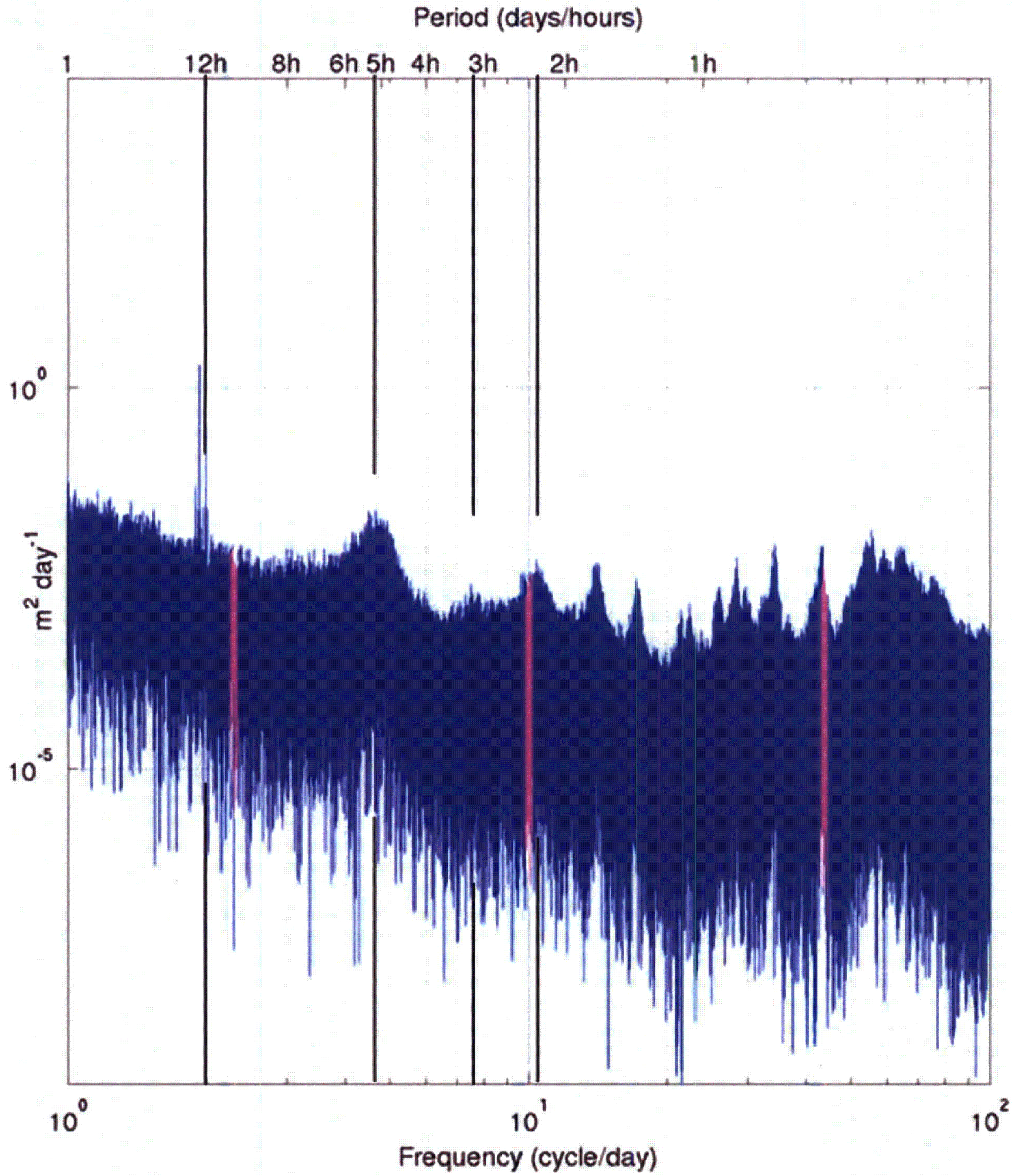

FIGURE 2.5-3: POWER SPECTRUM ANALYSIS OF OSWEGO, NY WATER LEVELS FOR PERIODS FROM 0 HOURS TO 24 HOURS

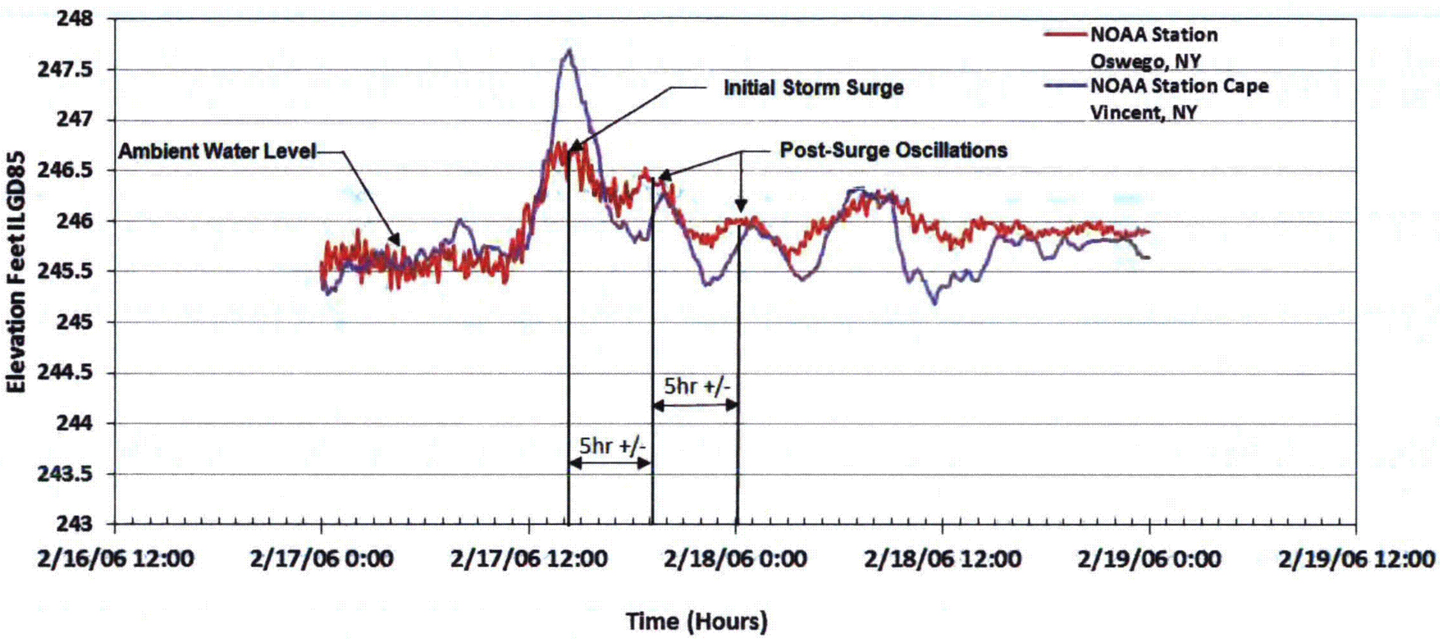

FIGURE 2.5-4: FEBRUARY 17, 2006 STORM NOAA WATER LEVEL PLOTS

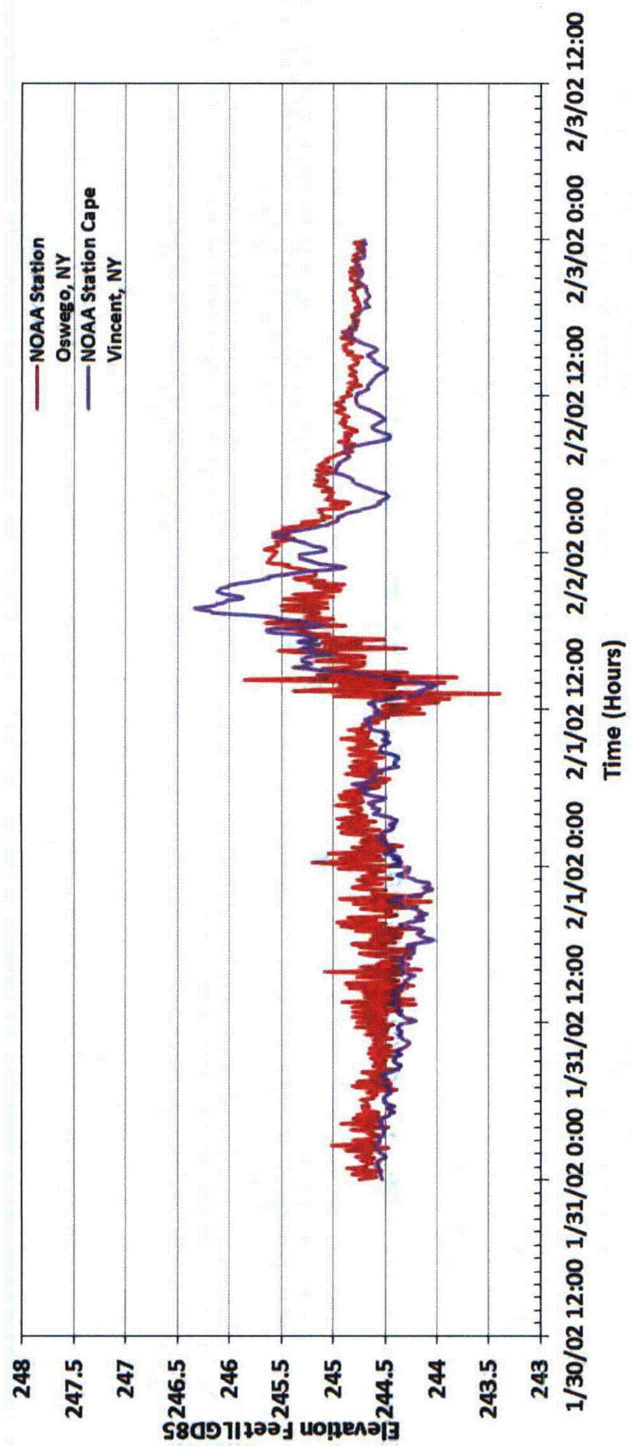

FIGURE 2.5-5: FEBRUARY 1, 2002 STORM NOAA WATER LEVEL PLOTS

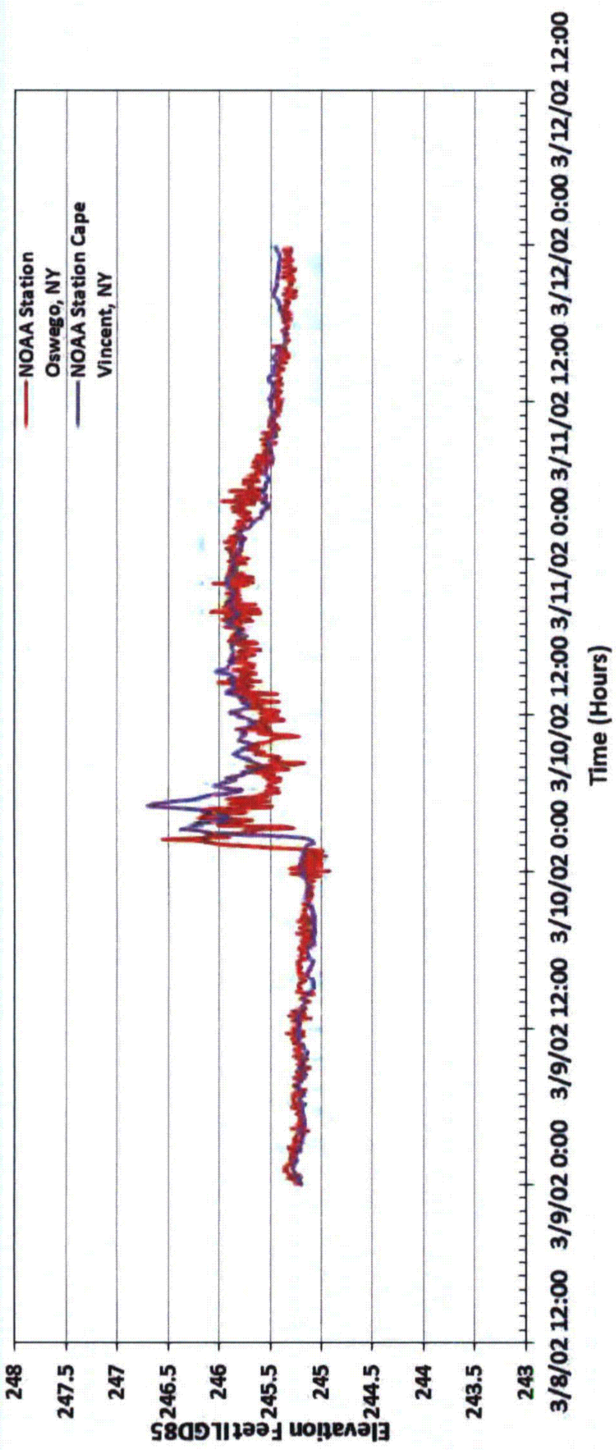

FIGURE 2.5-6: MARCH 9-10, 2002 STORM NOAA WATER LEVEL PLOTS

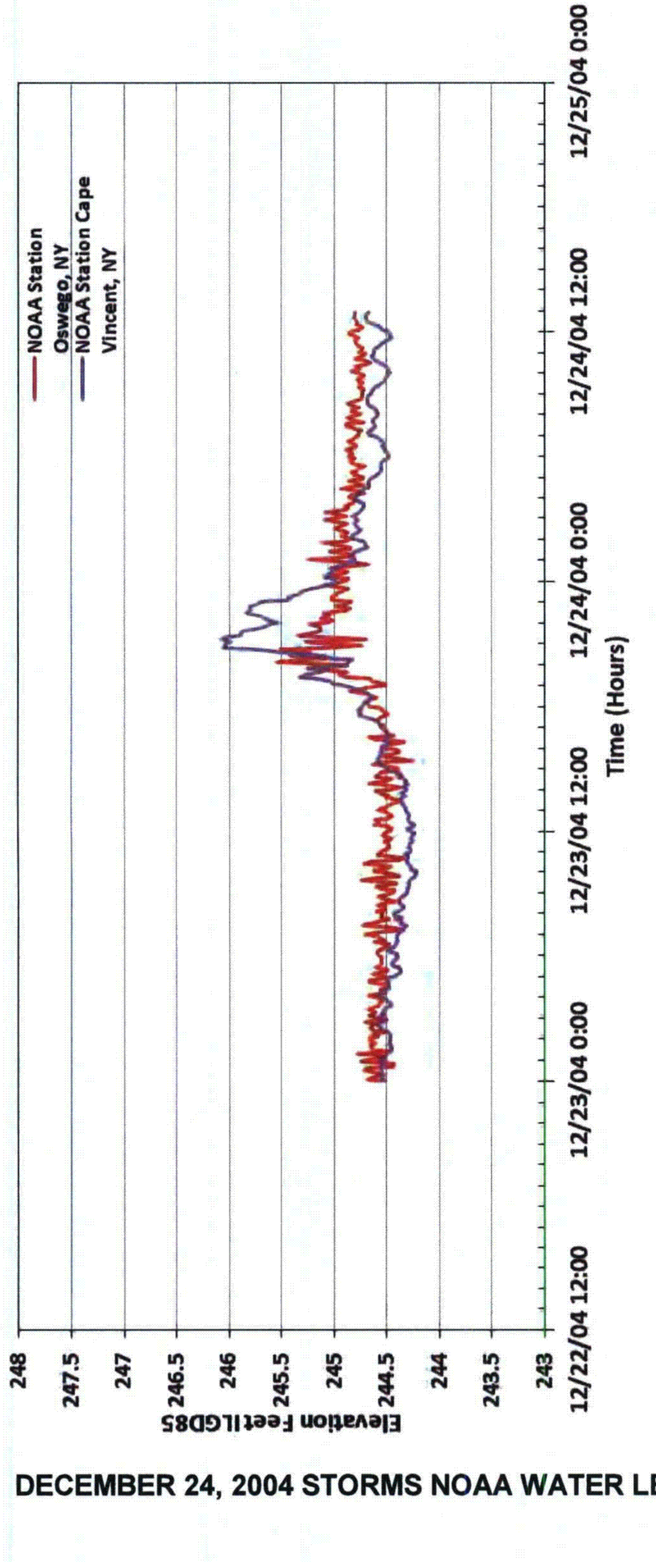

FIGURE 2.5-7: DECEMBER 24, 2004 STORMS NOAA WATER LEVEL PLOTS

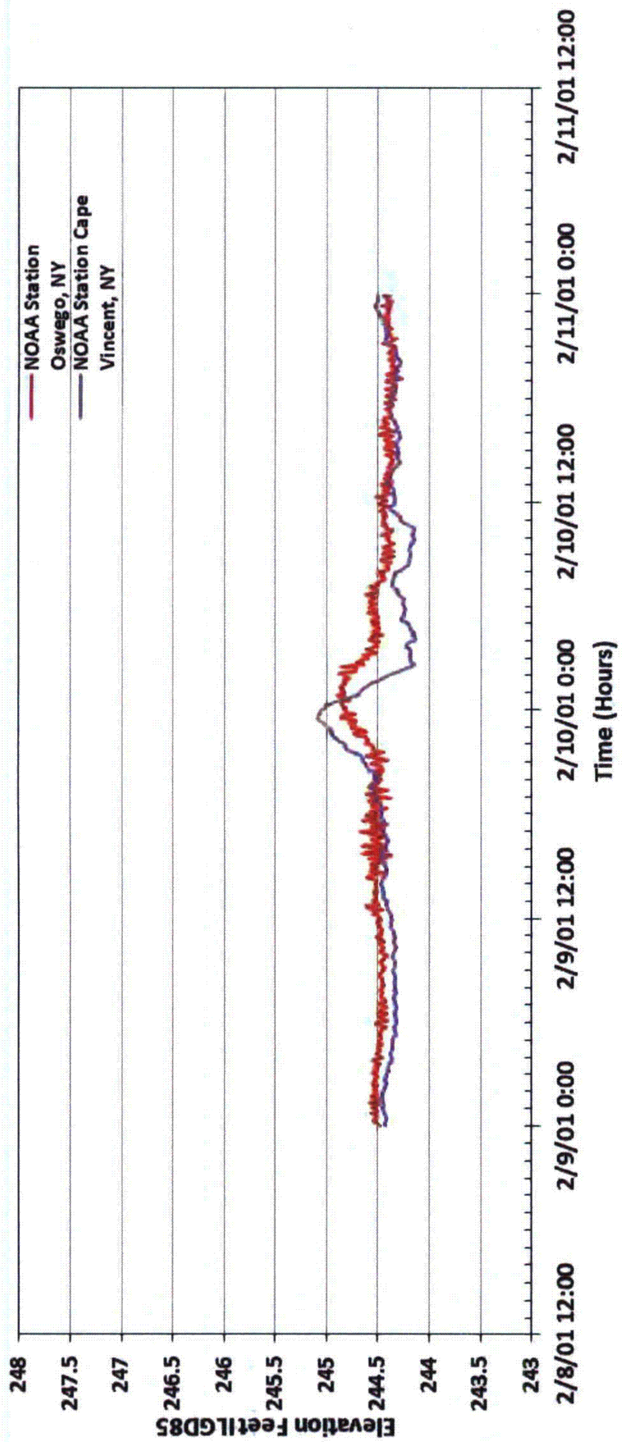

FIGURE 2.5-8: FEBRUARY 10, 2001 STORM NOAA WATER LEVEL PLOTS

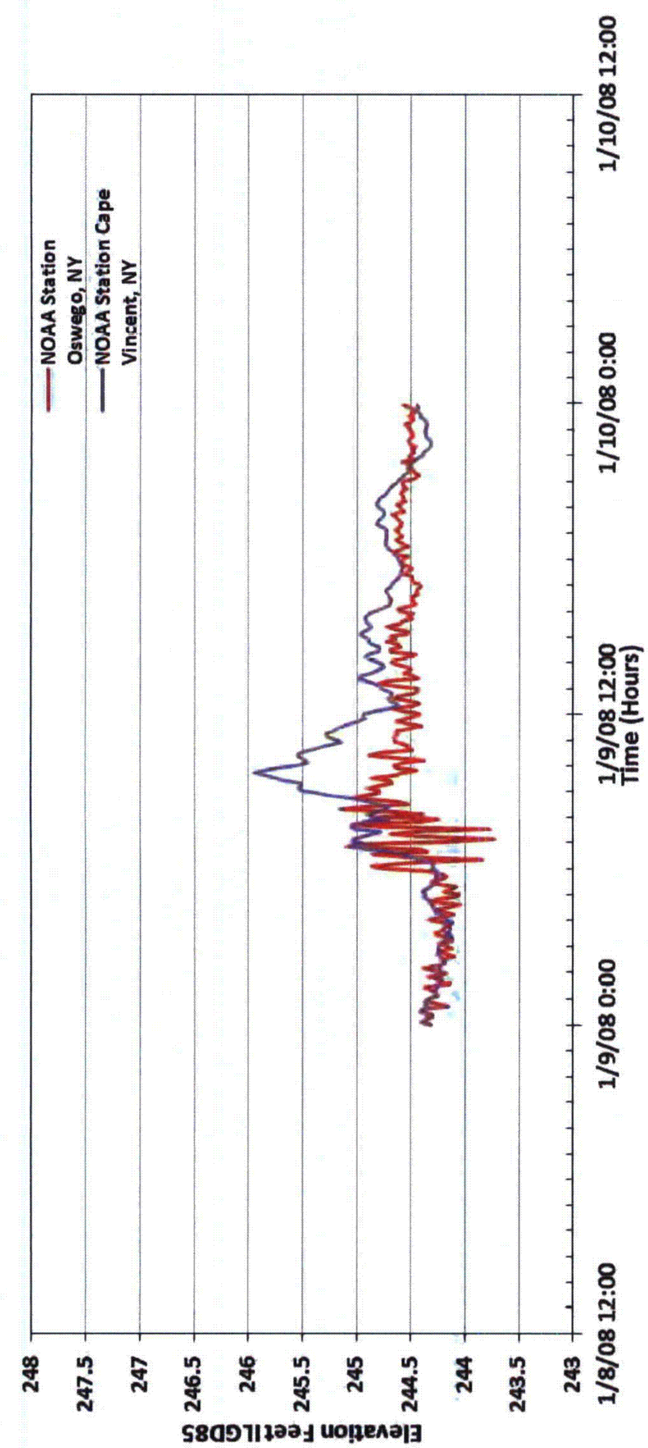

FIGURE 2.5-9: JANUARY 9, 2008 STORM NOAA WATER LEVEL PLOTS

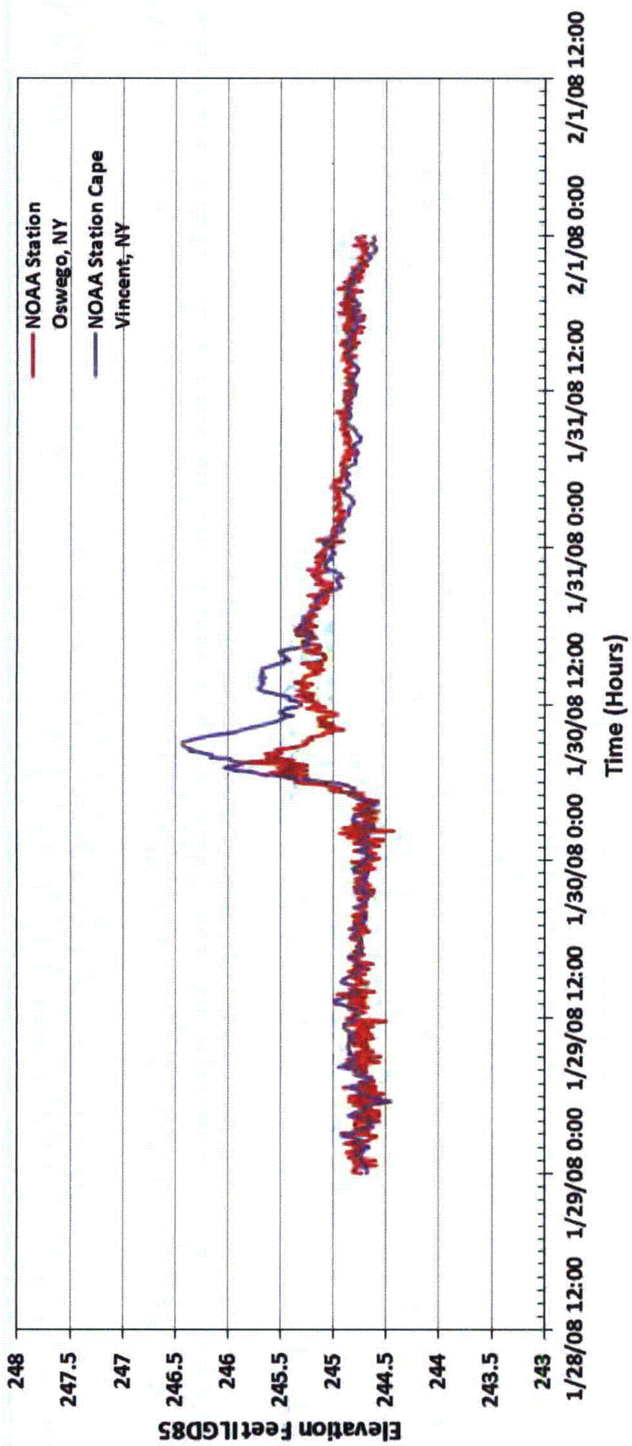

FIGURE 2.5-10: JANUARY 30, 2008 STORM NOAA WATER LEVEL PLOTS

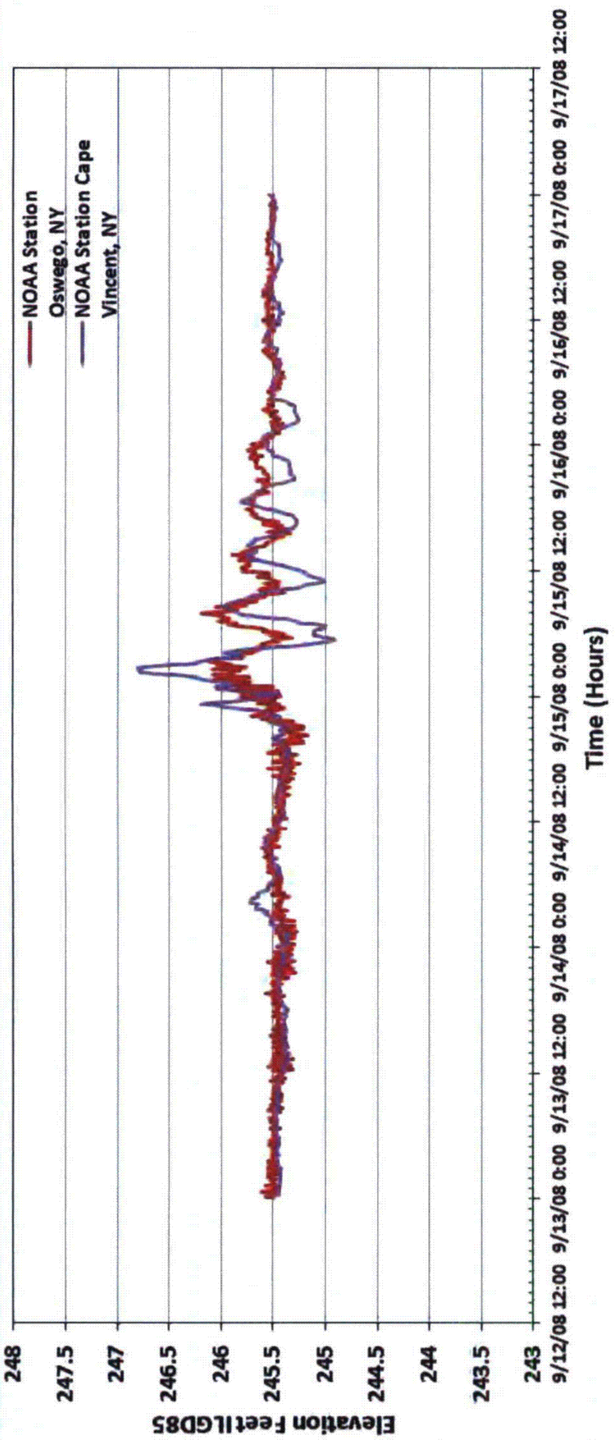

FIGURE 2.5-11: SEPTEMBER 14, 2008 STORM NOAA WATER LEVEL PLOTS

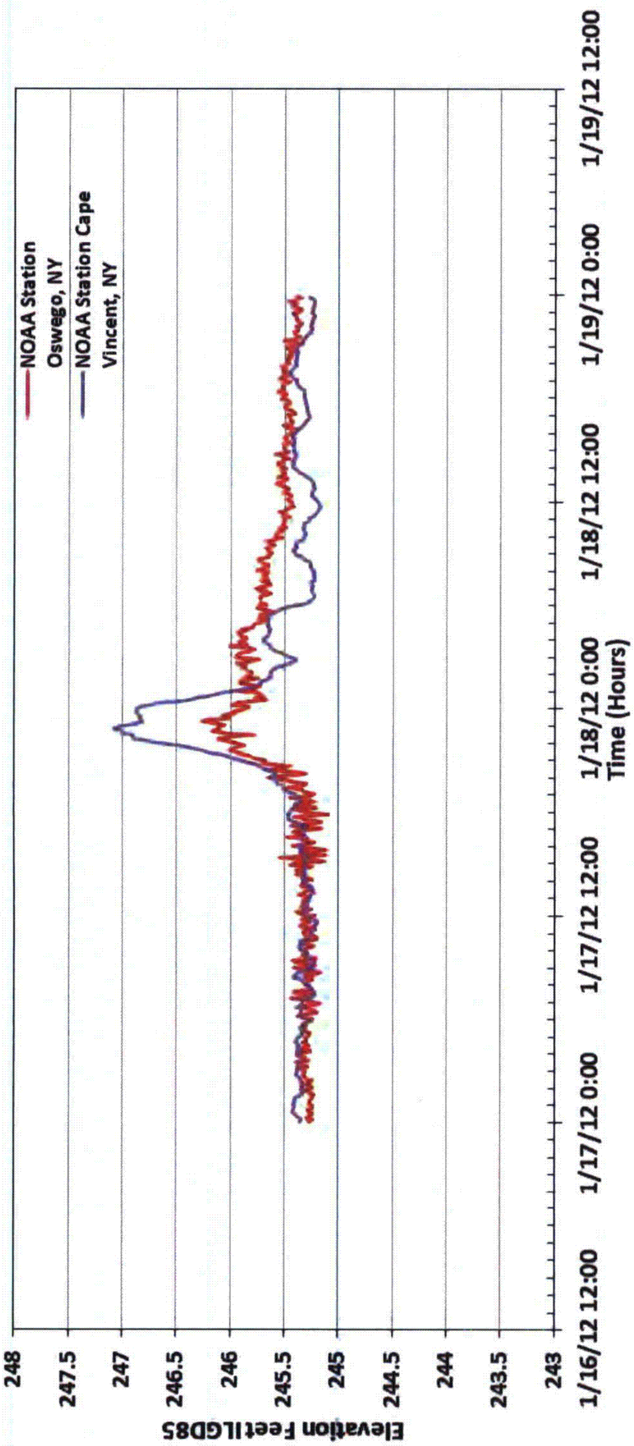

FIGURE 2.5-12: JANUARY 17, 2012 STORM NOAA WATER LEVEL PLOTS

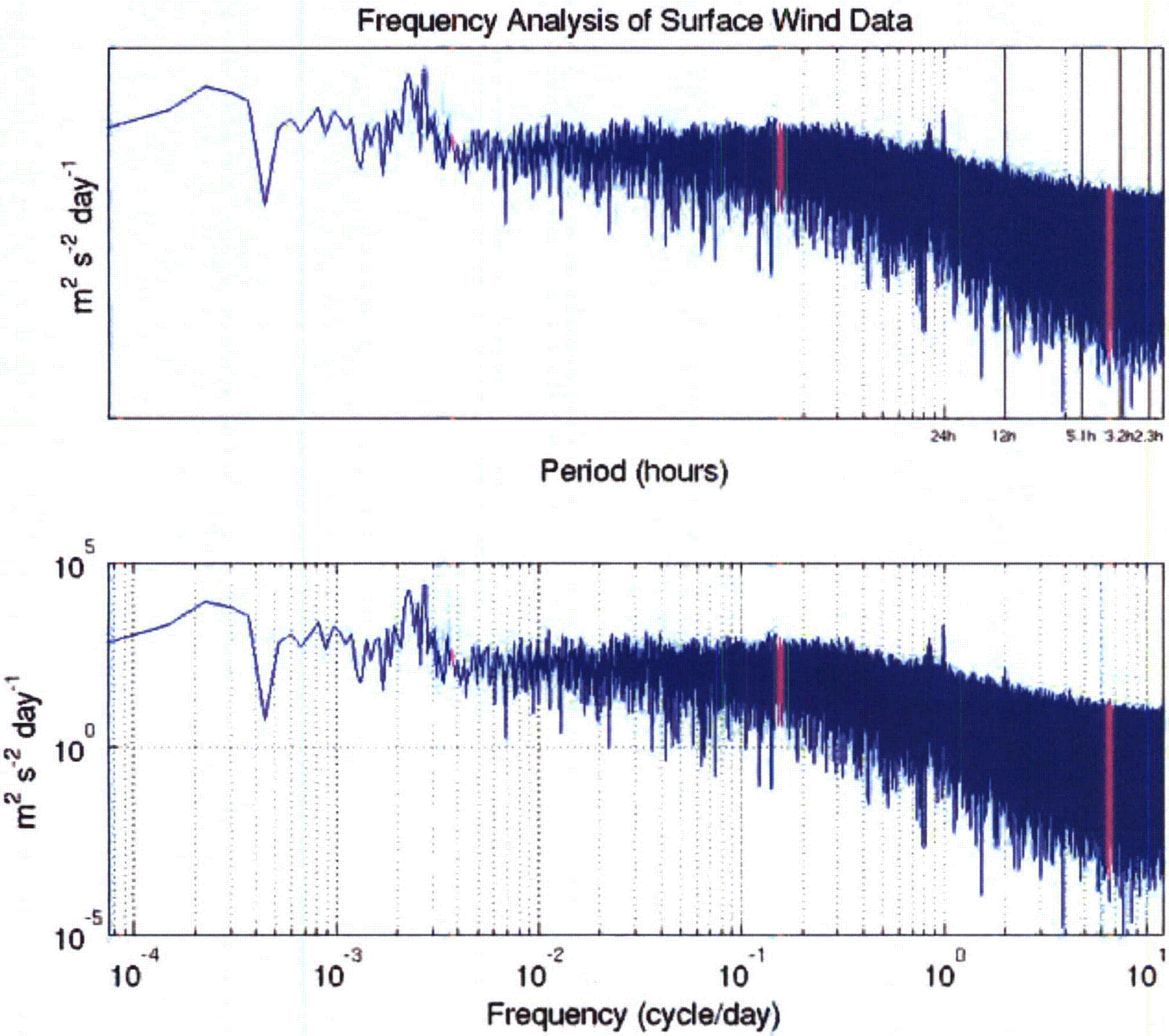

FIGURE 2.5-13: FREQUENCY ANALYSIS OF SURFACE WIND DATA

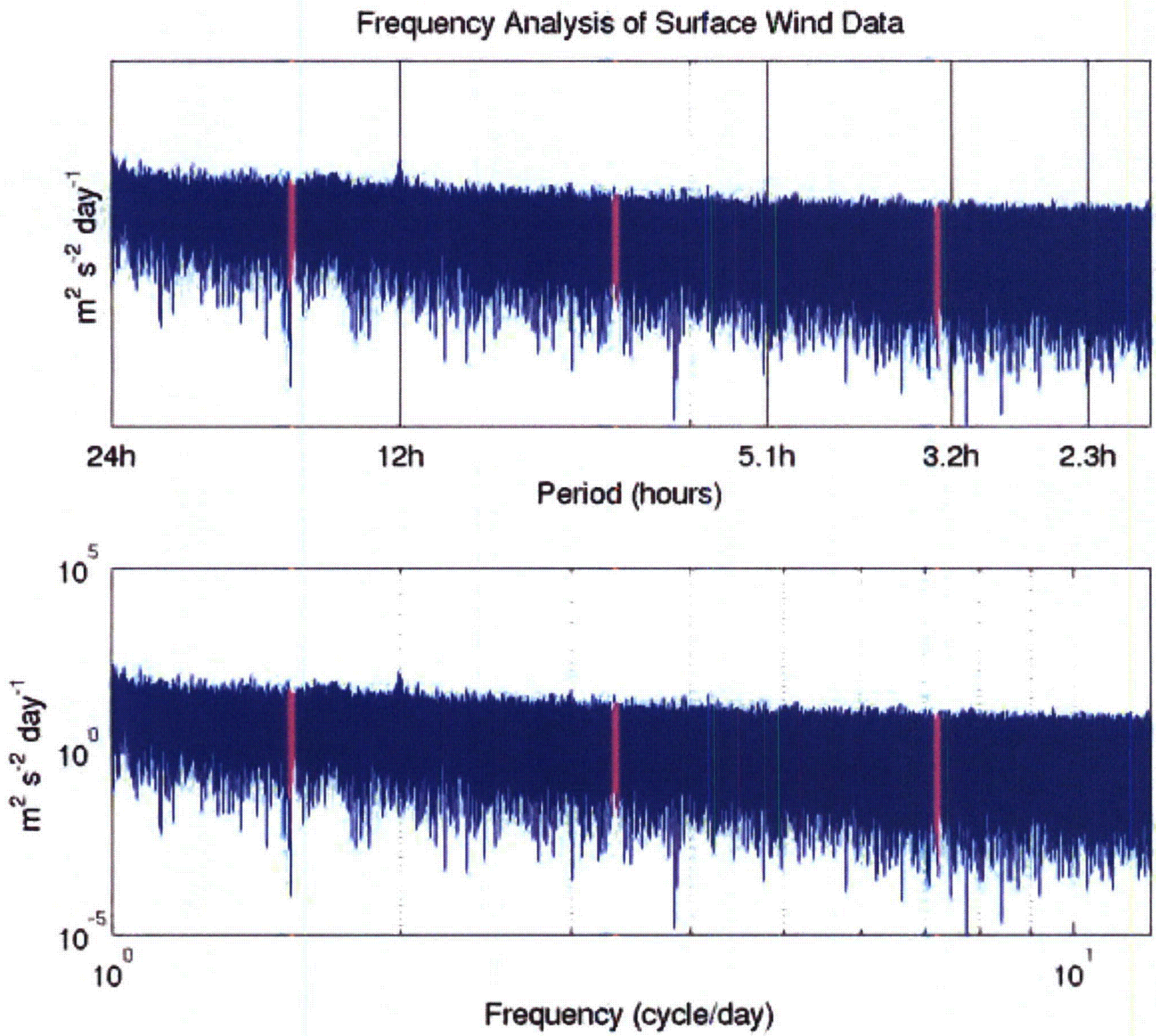

FIGURE 2.5-14: FREQUENCY ANALYSIS OF SURFACE WIND DATA

2.6 Tsunami

A tsunami is a series of water waves generated by a rapid, large-scale disturbance of a water body due to seismic, landslide, or volcanic tsunamigenic sources (NRC 2009, Section 1.1). As an inland site, NMP is not susceptible to oceanic tsunamis (NRC 2009, Section 2.1). Instead, there is the potential of tsunami-like waves in Lake Ontario.

2.6.1 Methodology

The HHA approach described in NUREG/CR-6966 (NRC 2009) was used for tsunamis and considered the first two of three steps. The third step was unnecessary based on the results of the first two steps, which answered the questions:

1. Is the site region subject to tsunamis?
2. Is the plant site affected by tsunamis?

Question 1 was answered by performing a regional survey and assessment of tsunamigenic sources. The regional survey was in four parts (AREVA 2013).

The first part of the regional survey was to review the Global Historical Tsunami Database (NOAA 2012g), maintained by the National Oceanic Atmospheric Administration's National Geophysical Data Center (NGDC), to determine the history of tsunamis. The second, third, and fourth parts of the regional survey included an assessment of the mechanisms likely to cause a tsunami.

Question 2 was answered by using the results from Question 1 to identify the primary effects of a tsunami wave near the NMP site and then performing a site screening to determine the potential effects to the NMP site (AREVA 2013).

2.6.2 Tsunami Results

2.6.2.1 Regional Survey

Tsunamis are generated by rapid, large-scale disturbance of a body of water. Therefore, only geophysical events that release a large amount of energy in a very short time into a water body generate tsunamis. The most frequent cause of tsunamis is an earthquake. Less frequently, tsunamis are generated by submarine and subaerial landslides and volcanic eruptions (NRC 2009, Section 1.3). Meteorite impacts, volcanoes, and ice falls can also generate tsunamis, but were excluded from the regional survey because meteorite impacts and volcanoes are very rare events and ice falls are generally associated with glacial ice processes.

2.6.2.1.1 NGDC Database Review

The NGDC tsunami-source-event database (NOAA 2012a) is global in extent with information dating from 2000 B.C. to the present. As an inland site, the NMP regional survey considered tsunami-like waves in the area around the Great Lakes, extending from 41° to 49° N Latitude and 76° to 92° W Longitude (Figure 2.6-1).

Seven events occurred in or near the Great Lakes. All events resulted in a seiche or disturbance in an inland river. Two of the events were caused by earthquakes, two by meteorological conditions, one by a landslide, and the two remaining unknown.

The maximum event water height increase was nearly 10 ft. in Lake Michigan; the maximum event water height increase in Lake Ontario was 5 ft. Both of these maximum heights were related to meteorological events. The maximum event water height related to an earthquake was 9 ft. in Lake Erie.

2.6.2.1.2 Earthquakes

To generate a major tsunami, a substantial amount of slip and a large rupture area is required. Consequently, only large earthquakes with magnitudes greater than 6.5 generate observable tsunamis (NRC 2009, Section 1.3.1).

Based on the geological and seismological information presented in the NMP Unit 3 FSAR (NMP3NPP 2009a, Section 2.5.1.1), the Lake Ontario region is relatively aseismic. A listing of the seismic events that have occurred within approximately a 300-mile radius of the NMP site between 1732 and 2007 includes only two earthquakes with magnitudes estimates over 5.5. Both earthquakes were magnitude 5.7 and occurred at distances of about 200 and 300 miles from the NMP site. As a result, the required level of seismic activity for development of a tsunami, i.e., an earthquake with a magnitude greater than 6.5, is essentially absent from the region.

Seismic activity outside the region can also produce seismic seiches (USGS 2012). Seismic waves from the Alaska earthquake of 1964, for example, caused water bodies to oscillate at many places in North America. Seiches were recorded at hundreds of surface-water gaging stations. The seismic seiche distribution did not have an obvious dependence on distance or azimuth from the epicenter. Instead, the distribution had a regional pattern, which reflected the influence of major geologic features. The southeastern part of the United States had the greatest density of seiches, while areas west of the Rockies, the Middle Atlantic States, and New England experienced few or no seiches. A favorable environment for seismic seiche generation includes thrust faults and locations controlled by structural uplifts and basins (USGS 2012). The Lake Ontario region, however, lacks such features.

2.6.2.1.3 Landslides

There are two broad categories of landslides: (1) subaqueous that are initiated and progress beneath the surface of the water body, and (2) subaerial that are initiated above the water and impact the water body during their progression or fall into the water body. In addition, landslide-generated tsunami-like waves have a very strong directivity in the direction of mass movement. Therefore, the outgoing wave from the landslide source propagates in the direction of the slide. In addition, the amplitude of the outgoing wave from a subaqueous landslide is affected by the terminal velocity of the movement, which in turn is a function of the repose angle, i.e., the slope angle. In addition, the amplitude of the outgoing wave from a subaqueous landslide is affected by the terminal velocity of the movement, which in turn is a function of the repose angle, i.e., the slope angle. The most common landslide mechanism for either landslide category is an earthquake (NRC 2009, Section 1.3.2).

Subaqueous Landslide – Lake Ontario Bathymetry

There are several prominent features within Lake Ontario that have the potential to produce a subaqueous landslide that could affect the NMP site. Two are shoreline areas with steep gradients. They include the Niagara Fan, located on the lake's southwest shoreline at the mouth of the Niagara River, and a ledge line, extending from SW to NE of Toronto (Figure 2.6-2). The direction of a landslide in either area, if it occurred, would be toward the opposite lake shoreline. For the Niagara Fan, a landslide would be northwest toward Toronto and the resultant tsunami-like wave, if it occurred, would not affect the NMP site. A landslide at the Toronto Ledge, if it occurred, would be southeast toward the Niagara River, which is nearly 150 miles west of the NMP site. In addition, the steepest slope of the Toronto Ledge is only about 5 degrees (Table 2.6-1). Thus, given a landslide, its speed would be limited and judged unlikely to generate an observable tsunami-like wave. As a result, the effect to the NMP site would be minimal, if any.

There are three distinct features within the lake basins that have the potential to produce a subaqueous landslide that could affect the NMP site. The first is the Scotch Bonnet Ridge, which separates the Mississauga and Genesee Basins (Figure 2.6-2). The ridge is oriented NE-SW. Thus, the direction of a landslide on the east side of the ridge, if it occurred, would be SE, toward but south of the NMP site. But the ridge has a relief of less than 20 meters and maximum slope of less than 5 degrees (Table 2.6-1). Thus, given a landslide, its mass and speed would be limited and judged unlikely to generate an observable tsunami-like wave.

The second is the Point Petre Ridge, northeast of the Genesee Basin (Figure 2.6-2). The ridge is oriented NNE-SSW. Thus, the direction of a landslide on the east side of the ridge, if it occurred, would be ESE, toward the NMP site. But, like the Scotch Bonnet Ridge, the ridge has a relief of less than 20 meters and maximum slope of less than 5 degrees (Table 2.6-1). Accordingly, given a landslide, its mass and speed would be limited and judged unlikely to generate an observable tsunami-like wave.

The third is a series of distinct NE-SW ridges that occupy the floor of the Rochester Basin, Lake Ontario's deepest basin (Figure 2.6-2). These ridges have a relief of 15-25 meters and a natural spacing of 250-1000 meters, with a linear aspect and uniform width. Most of the ridges have relatively flat tops with steep side slopes, some of which are steeper to the northwest, others to the southeast, and some are symmetrical in cross-profile. The maximum slope of these ridges, however, is less than 10 degrees (Table 2.6-1). The direction of a landslide, if it occurred, would be SE toward but south of the NMP site. But like both the Point Petre Ridge and Scotch Bonnet Ridge, given a landslide, its mass and speed would be limited and judged unlikely to generate an observable tsunami-like wave.

Subaerial Landslide - Lake Ontario Topography

The geographical areas where subaerial landslides occur are generally limited to areas of steep shoreline topography (NRC 2009, Section 1.3.2).

Similar to the lake bathymetry, the Lake Ontario shoreline has linear topographic features with uniform gradients around most of the perimeter (Figure 2.6-3). The land is either flat or gently rolling. The land on the eastern and northern shorelines also has similar characteristics.

There is, however, one dissimilar feature that has the potential to produce a subaerial landslide due to its steep gradient. The Scarborough Bluffs on the western Lake Ontario shoreline in Toronto is an escarpment that rises nearly 280 ft. above the lake and spans a shoreline length of 10 mi. (Eyles 1985). Due to the bluffs general NE-SW orientation, the direction of a landslide and resultant wave, if it occurred, would be SE toward southwestern lake shoreline north of the Niagara River, located more than 150 miles west of the NMP site. Thus, given a landslide, there would be little, if any, effect to the NMP site due to the direction and distance of the wave from the site.

2.6.2.1.4 Regional Survey Findings

Based on the regional survey results,

- NMP is not susceptible to oceanic tsunamis; instead, there is the potential of tsunami-like waves in Lake Ontario.
- Seven tsunami-like waves, namely seiches, have occurred in or near the Great Lakes. Two of these events were caused by earthquakes, one by a landslide, and the remainder by meteorological conditions.
- The Lake Ontario region is relatively aseismic. The largest recorded earthquakes are magnitude 5.7. The required level of seismic activity to generate an observable tsunami of magnitude greater than magnitude 6.5, therefore, is essentially absent from the region.
- Subaqueous landslides, if they occurred, are unlikely to generate an observable tsunami-like wave due to the limited bathymetric relief of ridges (less than 40 m) and their respective slopes (less than 10 degrees).
- Subaerial landslides are unlikely to occur around the perimeter of the Lake Ontario due to limited topographic relief. The one area with sufficient topographic relief, Scarborough Bluffs near Toronto, is oriented such that the direction of a landslide and resultant tsunami-like wave, if it occurred, would be toward the Niagara River on the southeastern lake shoreline more than 150 miles west of the NMP.

2.6.2.2 Site Screening

Based on the Regional Survey, tsunami-like waves (seiches) have occurred in or near the Great Lakes, including Lake Ontario. Most of the reported waves were caused by meteorological conditions. Two, however, are related to earthquakes and one to a landslide.

The primary effects of the tsunami-like waves on the NMP site are flooding due to runup from the wave and loss of cooling water due to dry intakes during drawdown caused by the receding wave.

2.6.2.2.1 Flooding Due to Runup

The elevation to entrances that house SSC important to safety for both NMP Units 1 and 2 is at grade level 261 ft. or higher (NMP1 1992, Section 3.1; NMP2 2010, Section 3.4.1.1.2). However, the NMP Unit 2 site is protected from Lake Ontario flooding by a revetment-ditch system to an elevation of 263 ft. (NMP2 2010, Section 3.4.1.1.2).

Lake Ontario has been regulated since 1960 (see Section 1.6.2). Nevertheless, if unregulated, projected lake levels could reach a maximum of 250.2 ft. (NMP3NPP 2009, Section 2.4.1.5).

From the NGDC database results in the Regional Survey, the maximum reported tsunami-like wave in any of the Great Lakes, caused by other than a meteorological event, is 9 ft. and is the result of an 1823 earthquake. Adding this value to the maximum projected lake level of 250.2 ft. yields 259.2 ft., which is a physical margin of 1.8 ft. below the 261 ft. grade level of the NMP Unit 1 entrance to SSC important to safety and 3.8 ft. below the 263 ft. grade level of the NMP Unit 2 flood protection revetment-ditch system.

2.6.2.2.2 Dry Intakes Due to Drawdown

NMP Unit 1 and Unit 2 have differing intake structures. Cooling water for NMP Unit 1 is drawn from a single intake structure located 1,100 ft. offshore near the bottom of Lake Ontario. The top of the intake is at elevation 228.0 ft. (NMP1 1965).

Cooling water for NMP Unit 2 is also drawn from offshore near the bottom of Lake Ontario. Unit 2, however, has two intake structures, located 1,300 and 1,400 ft. offshore. The top of the each intake is at lake elevation 232.5 ft. (NMP2 2004).

Prior to lake-level regulation, Lake Ontario reached a historic minimum level of 242.7 ft. (NMP3NPP 2009, Section 2.4.11.3). The available physical margin, therefore, between the top of the Unit 1 intake and the historic minimum unregulated lake level is 14.7 ft. (242.7-228.0). For Unit 2, the available physical margin between the top of the intakes and the historic minimum unregulated lake level is 11.2 ft (242.7-232.5).

Thus, assuming that the amplitude of the drawdown is no larger than that of runup (9 ft), there is sufficient physical margin for both Unit 1 and Unit 2 to protect SSC important to safety.

2.6.3 Conclusions

Based on the regional survey and site screening results, the following conclusions were established:

- As an inland site, the NMP site is not subject to oceanic tsunamis; however, tsunami-like waves (seiches) have occurred. Most of the reported waves were caused by meteorological conditions, but two were related to earthquakes and one to a landslide.
- Tsunami-like waves generated from
    - an earthquake are limited because the required level of seismic activity for development of a tsunami, i.e., an earthquake with a magnitude greater than 6.5, is essentially absent from the region;
    - a subaqueous landslide is unlikely to generate an observable tsunami-like wave due to the limited bathymetric relief of ridges and their respective slopes; and
    - a subaerial landslide is unlikely to occur due to limited topographic relief. The one area with sufficient topographic relief, Scarborough Bluffs near Toronto, is oriented such that

the direction of a landslide and resultant tsunami-like wave, if it occurred, would be toward the southeastern lake shoreline, more than 150 miles west of the NMP site.

- Notwithstanding the occurrence of tsunami-like waves, the potential effects on the NMP site (wave runup and draw down) are negligible because there is sufficient available physical margin to protect SSC important to safety. The physical margin is based on the maximum recorded tsunami-like wave resulting from an earthquake in the Great Lakes region occurring coincident with the maximum (runup) and minimum (drawdown) lake levels.

2.6.4 References

NOTE: Refer to the Project Manager's approval (on the signature page of this report) verifying that the Constellation Nuclear Energy Group (CENG) references are valid sources of design input created in accordance with the CENG's QA program.

AREVA 2013. AREVA Document No. 51-9189826-000, Tsunami Hazard Assessment at Nine Mile Point Nuclear Station Units 1 and 2 Site, 2013.

Eyles 1985. N. Eyles, et al., Applied Sedimentology in an Urban Environment – the Case of Scarborough Bluffs, Ontario; Canada's Most Intractable Erosion Problem, Geoscience Canada, Vol. 12, No. 3, pp. 91-104, 1985.

NMP1 1965. Nine Mile Point Nuclear Station Unit 1, Dwg. No. C-15451-C, Rev. 1, Lake Intake Structure.

NMP1 1992. Nine Mile Point Nuclear Station Unit 1 Final Safety Analysis Report, Section 3.1.

NMP2 2004. Nine Mile Point Nuclear Station, Unit 2, Dwg. No. EC-015G, Sheet 1, Rev. 9, Plans, Sections & Details Intake Structure.

NMP2 2010. Nine Mile Point Nuclear Station Unit 2 Final Safety Analysis Report, Section 3.4.1.1.2.

NMP3NPP 2009a. Nine Mile Point 3 Nuclear Power Plant, Final Safety Analysis Report, Rev. 1, Section 2.5, Geology, Seismology, and Geotechnical Engineering (ADAMS Accession No. ML090970449).

NMP3NPP 2009b. Nine Mile Point 3 Nuclear Power Plant, Final Safety Analysis Report, Rev. 1, Section 2.4, Hydrologic Engineering (ADAMS Accession No. ML090970448).

NOAA 2012a. National Oceanic Atmospheric Administration, National Geophysical Data Center, Tsunami Database Website: http://www.ngdc.noaa.gov/hazard/tsu.shtm; Accessed July 17, 2012.

NOAA 2012c. National Oceanic Atmospheric Administration, National Geophysical Data Center, Bathymetry Website: http://www.ngdc.noaa.gov/mgg/greatlakes/greatlakes.html; Accessed July 20, 2012.

NOAA 2012d. National Oceanic Atmospheric Administration, National Geophysical Data Center, Lake Ontario Bathymetry Website: http://www.ngdc.noaa.gov/mgg/greatlakes/lakeontario_cdrom/html/gmorph.htm; Accessed July 20, 2012.

NRC 2009. NUREG/CR-6966, Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America, U.S. NRC, March 2009 (ADAMS Accession No. ML091590193).

UniStar 2009. Response to NRC Request for Additional Information RAI No. 99, Question 02.04.06-10, 02.04.06-12 through 02.04.06-17, Probable Maximum Tsunami Flooding, Calvert Cliffs Nuclear Power Plant, Unit 3, August 24, 2009.

USGS 2012. U.S. Geological Survey, Seismic Seiches, Website: http://earthquake.usgs.gov/learn/topics/seiche.php, accessed August 23, 2012.

WTBM 2012. World Terrain Base Map, Website: http://www.arcgis.com/home/item.html?id=c61ad8ab017d49e1a82f580ee1298931; accessed August 6, 2012.

Table 2.6-1: Lake Ontario Prominent Bathymetric Feature Slopes

Prominent Feature (Name or Description)	Gradient Length (m)	Gradient height (m)	Slope (degrees)
Toronto Ledge	500	40	4.6
Scotch Bonnent Ridge	750	22	1.7
Point Petre Ridge	2000	40	1.1
W. of Point Petre	400	24	3.4
Rochester Basin	144	24	9.5

Source: NOAA 2012 d

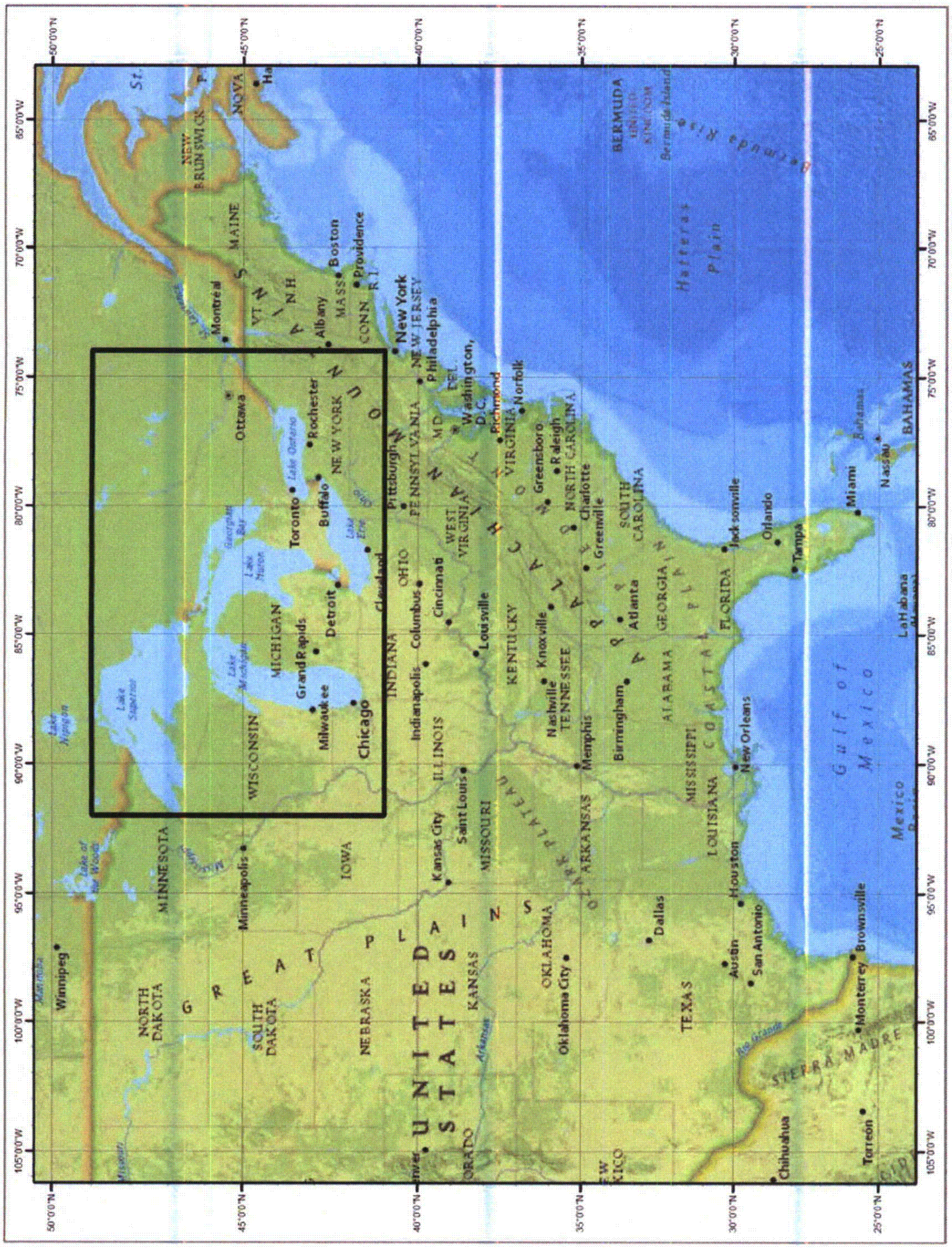

Figure 2.6-1: NGDC Tsunami-Source-Event Database Region

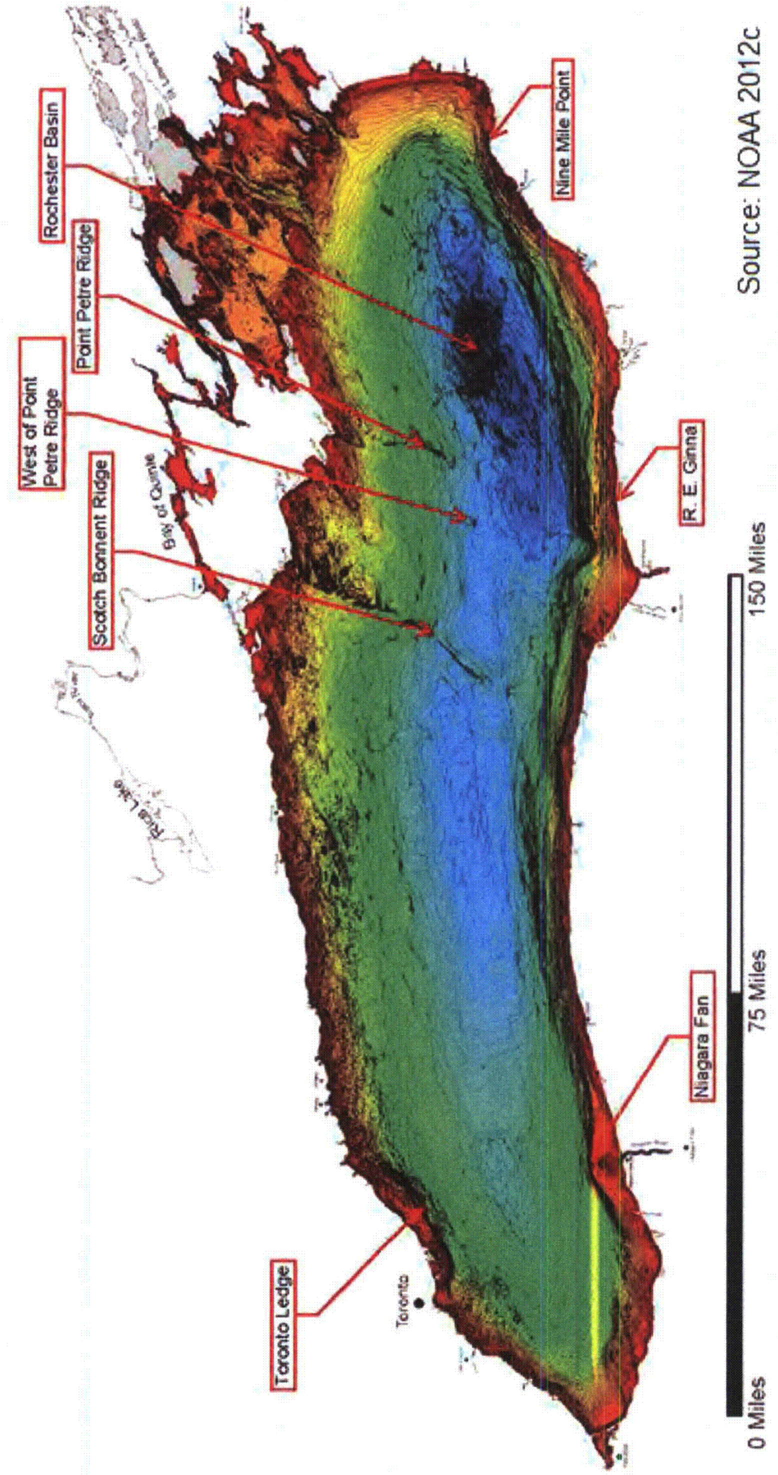

Figure 2.6-2: Lake Ontario Bathymetry Map

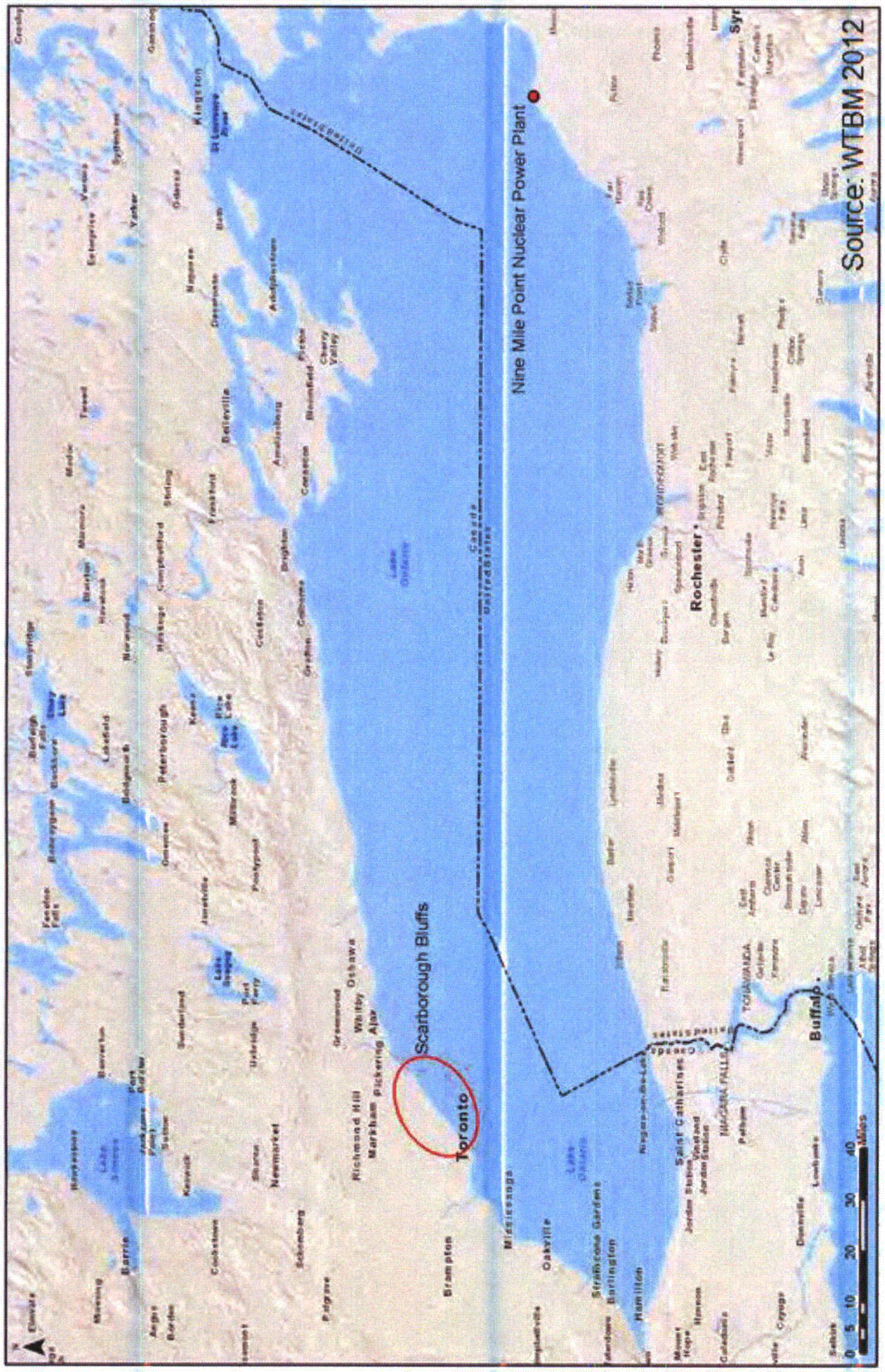

Figure 2.6-3: Lake Ontario Topography Map

2.7 Ice Induced Flooding

2.7.1 Methodology

The HHA approach described in NUREGCR-7046 (NRC 2011) was used for Ice Induced Flooding (AREVA 2013) along with the analyses performed as part of the NMP Unit 3 Nuclear Power Plant (NMP3NPP) Combined License (COL) application (NMP3NPP 2009).

The proposed NMP3NPP project is located adjacent to the NMP Unit 1 and Unit 2 site. As such, it represents the same hydroloclimatic setting as NMP Unit 1 and Unit 2 and is characterized by similar flooding mechanisms. Therefore, the potential of ice and ice induced flooding analyses performed at NMP3NPP apply to NMP Unit 1 and Unit 2 as well.

2.7.2 Ice Induced Flooding Results

Historical data characterizing ice conditions at the NMP site have been collected and the effects evaluated as part of the NMP3NPP COL application (NMP3NPP 2009, Section 2.4.7). These data included ice cover and thickness evaluations in Lake Ontario developed by the National Oceanic and Atmospheric Administration (NOAA) and ice jam records from the Unites States Army Corps of Engineering (USACE).

Although most tributaries to Lake Ontario are prone to ice formation, there has been no ice jam formation or flooding on Lake Ontario due to breaching of ice jams on upstream tributaries or the downstream Saint Lawrence River. The USACE Ice Jam Database maintains records of current and historical ice jams within the United States. The nearest historical ice jams data on record occurred on the Oswego River in January of 1952 and January of 2004 (NMP3NPP 2009, Section 2.4.7.8).

There are no records of ice jam formation on the Saint Lawrence River causing flooding on Lake Ontario (NMP3NPP 2009, Section 2.4.7.8). The International Saint Lawrence River Board of Control (ISLRBC) regulates Lake Ontario outflow to the Saint Lawrence River and thereby controls lake levels in Lake Ontario. Following the close of the navigation season, ISLRBC reduces the Lake Ontario outflow to promote the formation of a smooth, stable ice cover on the St. Lawrence River. The stable ice cover formation is beneficial in that it reduces the risk of ice jams on the river (NMP3NPP 2009, Section 2.4.7.8).

In addition, there are no major perennial streams on the NMP site or major streams close to the site that would contribute to the potential of ice induced flooding at the site.

2.7.3 Conclusions

Based on the historical records, ice induced flooding at the NMP site is unlikely and would not affect the SSCs important to safety because:

- The nearest historical ice jams data on record occurred on the Oswego River, which is more than 6 mi from the NMP site.

- The ISLRBC reduces the Lake Ontario outflow to promote the formation of a smooth, stable ice cover on the St. Lawrence River, which is beneficial in that it reduces the risk of ice jams on the river.
- There are no major streams close to or on the site that would contribute to the potential of ice induced flooding at the NMP site.

2.7.4 References

NOTE: Refer to the Project Manager's approval (on the signature page of this report) verifying that the Constellation Nuclear Energy Group (CENG) references are valid sources of design input created in accordance with the CENG's QA program.

AREVA 2013. AREVA Document No. 51-9189708-000, Ice-Induced Flooding at the Nine Mile Point Nuclear Station Units 1 and 2 Site, 2013.

NMP3NPP 2009. Nine Mile Point 3 Nuclear Power Plant, Final Safety Analysis Report, Rev. 1, Section 2.4, Hydrologic Engineering (ADAMS Accession No. ML090970448).

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

2.8 Channel Migration or Diversion

The flood hazard associated with channel mitigation or diversion at the NMP site is judged to be negligible based on the NMP Unit 3 Nuclear Power Plant (NMP3NPP) Combined License (COL) application.

As described in Section 2.4.9 of the NMP3NPP FSAR, the seismic, topographical, geologic, and thermal evidence in the region shows there is very limited potential for upstream diversion or rerouting of Lake Ontario (due to channel migration, river cutoffs, ice jams, or subsidence) to adversely impact safety-related facilities or water supplies. In addition, there are no perennial streams or rivers in the NMP watershed. The closest stream, Lakeview Creek, is outside the NMP site. See Section 2.2, Flooding in Streams and Rivers, for additional information.

2.8.1 References

NOTE: Refer to the Project Manager's approval (on the signature page of this report) verifying that the Constellation Nuclear Energy Group (CENG) references are valid sources of design input created in accordance with the CENG's QA program.

NMP3NPP 2009. Nine Mile Point 3 Nuclear Power Plant, Final Safety Analysis Report, Rev. 1, Section 2.4, Hydrologic Engineering (ADAMS Accession No. ML090970448).

2.9 Combined Effect Floods

2.9.1 Methodology

The combined effect flooding evaluation uses the methodology and combinations outlined in Section 3.9 and Appendix H of NUREG/CR-7046 (USNRC 2011). The criteria for combining flood mechanisms are also discussed in ANSI/ANS 2.8-1992 (ANS 1992), as referenced noted in NUREG/CR-7046. Because NMP Units 1 and 2 are immediately adjacent to one another and share the same meteorological and hydrological setting on the shore of Lake Ontario, they are subject to the same combination of flood effects.

As described in Section 2.2, Flooding in Streams and Rivers, NMP Units 1 and 2 are not subject to riverine flooding, as the nearest perennial stream, Lakeview Creek, is located in a separate watershed. Lakeview Creek does not overflow its watershed boundaries during the PMF. In addition, NMP Units 1 and 2 are not located along the shore of an open or semi-enclosed body of water. Therefore, combined flood effects related to sites at streamside locations per NUREG/CR-7046, Appendix H, Sections H.3 and H.4.2 do not apply to NMP. Similarly, Sections 2.3, Dam Breaches and Failures, and 2.6, Tsunami, demonstrate that NMP Units 1 and 2 are not affected by dam failures or tsunami; therefore, NUREG/CR-7046, Appendix H, Sections H.2 and H.5 also do not apply.

Note that even though the Local Intense Precipitation may produce the worst case flood elevation for NMP Unit 1 and Unit 2, it is considered an independent flood event and is not included in the Combined Effects Floods evaluation based on recommended methods for combining the flood generating mechanisms presented in Appendix H of NUREG/CR-7046. Specifically, NUREG/CR-7046, Section H.1 requires evaluation of riverine or hydrologic phenomena that does not apply to evaluation of runoff on non-riverine sites. The alternatives to be evaluated as per Section H.1 are provided below, along with their relationship to the LIP flood:

- mean monthly base flow: baseflow (i.e., average stream flow) is non-existent within NMP.
- median antecedent soil moisture: the LIP does not consider soil moisture (i.e., the watershed is considered to be completely impervious as per NUREG/CR-7046).
- antecedent rainfall: By definition, the LIP is centered over the site, which represents a small watershed (i.e., site extent). Such watersheds have short lag times on the order of 10 minutes or less as per NUREG/CR-7046, Appendix B which means antecedent floods pass quickly and do not appreciably affect results.
- wind wave activity: The LIP results in primarily unconfined overland flows of limited depth. Any flooding due to the LIP would not result in significant wind-generated waves due to flood depth limitations.
- snowpack: The LIP is applied to developed areas (i.e., plant site) which are typically maintained/managed to limit snowpack accumulation. NUREG/CR-7046 Appendix B requires the evaluation of all-season 1-square-mile PMP as per HMR-51 and HMR-52 (as opposed to cool-season PMPs discussed in HMR-53).

As a result, the following combination of flood effects applies to NMP, which is located along the shore of Lake Ontario, per NUREG/CR-7046, Appendix H, Section H.4:

- Probable Maximum Storm Surge (PMSS) and Seiche with wind-wave activity.
- The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

The PMSS and the antecedent water levels in Lake Ontario are developed in Section 2.4, Storm Surge. The seiche is bounded by the PMSS, as described in Section 2.5, Seiche. The analysis of wind-generated waves and their potential impact is presented here. Wind-generated waves will contribute to elevated water levels at NMP Units 1 and 2 during severe storms. NMP Units 1 and 2 are protected against wave and storm surge impacts by shore protection structures consisting of a rubblemound dike at NMP Unit 1 and a combination dolosse and rubblemound revetment ditch structure at NMP Unit 2. The analysis of wind-generated waves included the following steps:

1. Review historical wave data from four US Army Corps of Engineers (USACE) Wave Information Studies (WIS) database and assess the results.
2. Calculate nearshore wave heights and periods using the SWAN (version 40.51 AB) numerical wave model.
3. Calculate the heights and periods of the waves breaking on the shore protection structures.
4. Calculate wave runup along the NMP shoreline using the following independent methods:
    a. van Gent method, which is currently recommended by FEMA for the evaluation of wave runup at barriers and structures in the Great Lakes;
    b. Coastal Engineering and Design Analysis System (CEDAS) computer program, which was developed by the USACE (Leenknecht et al. 1992); and
    c. Delft Hydraulics method, presented in the Coastal Engineering Manual (USACE 2011e).
5. Evaluate the wave runup results to determine the runup heights to be used for the evaluation of wave overtopping impacts and then calculate wave runup overtopping rates based on the selected wave runup heights using the CEDAS computer program.
6. If overtopping of the shore protection structures is found to occur, then calculate peak flow rates within, and peak water levels elevation along, the swale/drainage ditch behind NMP Unit 1 Dike and NMP Unit 2 Revetment Ditch will be determined using the HEC-RAS computer program. Flow rates to be based on inflow of water from the wave runup overtopping.

2.9.2 Results

The historic site information and new wind, wave and bathymetric information use various vertical datums due to discontinuation of older vertical datums. Table 2.9-1 presents a conversion matrix for the various vertical datums.

2.9.2.1 Historical Data Overview

The WIS provides a source of long term wave records on Lake Ontario. WIS is a database of hindcast wind and nearshore wave conditions provided and maintained by the USACE. The goal of WIS is to provide a long record of wind and wave data for use in coastal engineering with a temporal and spatial resolution not widely available from wind and wave measurements. WIS data are routinely validated against other modeling efforts and observations based on USACE standards (USACE 2010). Data in Lake Ontario are produced by the WAVAD model and are available from 1961 to 2000.

There are four WIS stations located immediately offshore of the NMP Units 1 and 2 sites (Figure 2.9-1). WIS provides summaries of the top 10 events based on peak significant wave height for each station (USACE a-d 2011). Table 2.9-2 summarizes the depth, range of significant wave heights, and associated wave periods for the top 10 events at each station. Wave heights range from 17.1 to 22.3 feet. The peak significant wave height of 22.3 feet among the four stations occurred at Station 91136 and had an associated peak period of 10.3 seconds. The range of peak periods for the top 10 wave events across all stations is 9.3 to 11.1 seconds.

The WIS stations provide a good indication of wave conditions in the vicinity of the site. Because they are in deeper water than the SWAN output points discussed below, the wave height provided by WIS would become depth limited as they approach shore; therefore, the wave heights are not directly comparable to the wave heights predicted by the SWAN model. However, since period is invariant, it can be compared to the shallow water wave periods predicted by SWAN. The range of peak periods provided by the WIS stations varies from 9.3 to 11.1 seconds. As discussed below in Section 2.9.2.2, the peak periods predicted by SWAN for the PMWS range from 9.0 to 11.9 seconds, showing very good agreement with the WIS data.

2.9.2.2 Nearshore Wave Heights and Periods

The nearshore wave heights and periods were calculated using the SWAN (Version 40.51 AB) numerical wave model. SWAN (Version 40.51 AB) is a third-generation wave model developed by Delft University of Technology, and computes random, short-crested wind-generated waves in coastal regions and inland waters.

SWAN model inputs were developed for Lake Ontario, including Lake Ontario bathymetry and the temporally and spatially varying wind field (wind velocities and direction developed in Section 2.4). A nested model bathymetry grid was used, consisting of a coarse grid (0.01 degree by 0.01 degree) covering the entire Lake Ontario and a finer grid (0.002 degree by 0.002 degree) for the area offshore of NMP Units 1 and 2. The grid bottom depths for Lake Ontario were developed using the National Oceanic and Atmospheric Administration's (NOAA) National Geophysical Data Center's (NGDC) Great Lakes Bathymetry dataset (Virden, W.T. 1999) and were adjusted to reference them to the design PMSS still water elevation. The wind forcing (the PMWS wind field) was applied to the entire model grid for each model time step.

Figures 2.9-2 and 2.9-3 present the limits of the model grids and bathymetry.

SWAN model output was specified at four nearshore water locations at the NMP Units 1 and 2 sites to characterize water waves in the vicinity of the NMP Units 1 and 2 shore protection structures. The positions of the four points are shown in Figure 2.9-4.

The four points are representative of conditions along the NMP Units 1 and 2 shoreline protection structures. Points 1 and 3 are located at the west and east ends of the site, respectively. Due to the high resolution of the SWAN grid, Point 1 is in shallower water than Points 2 and 3, so an additional point, Point 1A, was added at the west end of the site at the same latitude as Points 2 and 3. Point 3 is located where the Unit 1 and Unit 2 structures meet.

The SWAN model outputs are presented on Table 2.9-3. Both significant wave height, H_s, which is the average height of the maximum 1/3 of all waves, and peak period, T_p (which is the peak spectral wave period associated with the maximum significant wave height), are included in the output. The direction column line in Table 2.9-3 indicates the direction towards which waves are propagating using a Cartesian convention (i.e., directions are measured counterclockwise from east). The maximum wave heights, H_{max}, and maximum wave periods, T_{max}, in Table 2.9-3 were calculated from the SWAN output data using conversion coefficients of 1.67 and 1.2, respectively (ANS 1992). The results presented in Table 2-9.3 indicate that the significant wave height varies from 8.2 to 12.5 feet, and the peak period varies from 9.0 to 11.9 seconds.

2.9.2.3 Depth Limited Wave Conditions and Wave Run-up

Shoreline topographic and bathymetric data were developed from the USACE Great Lakes Topography/ Bathymetry LiDAR dataset (USACE 2007) and is shown in Figure 2.9-5. As waves move into shallow water they shoal, i.e. increase in height, as they contact the bottom. As the depth decreases, the height of the wave increases until the wave reaches a point where the height of the wave is too large for its length and the wave breaks. For shore protection structures where shallow water conditions along the structure toe limit wave height, the maximum or design waves are considered depth limited. The depth limited wave height is calculated by multiplying the depth by the shallow water by the breaking limit of 0.78 (FEMA 2007, FEMA 2009).

As shown in Table 2.9-4, water depths along the toe of the NMP shore protection structures, which were determined by subtracting the structure toe depth shown in Figures 2.9-9 through 2.9-14 from design still water elevation, range from 7.5 to 10.1 feet. A shallower toe depth was used for Transect 5 since the depths offshore of the structure limit the incident wave heights. Because the SWAN model maximum wave heights listed in Table 2.9-3 exceed the wave heights allowed by the breaking limit of 0.78 for the water depths at the NMP structure toe, the maximum wave heights are depth limited.

Wave runup is the vertical height above the stillwater level to which water from an incident wave will runup the face of a structure (USACE 1984). It is dependent on the nearshore wave height, period, and direction, and in the case of structures, their geometry, roughness, and permeability.

The NMP Units 1 and 2 sites, located on the southeast shore of Lake Ontario, are protected by a combination of shoreline structures. Figure 2.9-6 shows an oblique aerial view of both structures (Booij, 1999). The NMP Unit 1 site is protected by a rubblemound dike with a crest elevation that ranges from elevation 261.6 to of 264.2 feet (NGVD29) as shown in Figure 2.9-7. The dike has a drainage ditch on the landward side, which drains to the west to Lake Ontario. The Unit 1 dike is tied into the revetment ditch system that protects NMP Unit 2 site, which has a design elevation of 263 feet (USLS35) (SWEC 1983). The NMP 2 revetment ditch structure drains to the east to Lake Ontario.

Since both structure geometry and nearshore bathymetry vary along the NMP Units 1 and 2 sites, six transects were established to capture the range of nearshore and structure characteristics (Figure 2.9-8). Transects were located at either end of the site (Transect 1, Transect 6), at the lowest point along the dike/revetment system (Transect 3), and in several representative locations along the shore protection structures. The transect elevations were developed using the 2007 USACE Great Lakes Topography/Bathymetry LiDAR dataset (USACE 2007). The transect elevation profiles are presented on Figures 2.9-9 to 2.9-14.

The depth at the structure toe was calculated using the PMSS stillwater elevation. Slopes for the transects across the NMP Unit 2 revetment (Transects 4 through 6) were set to 0.5 based on the slope protection design drawings (S&WEC 1983, Constellation 2003). Slopes for the transects across the NMP Unit 1 dike (Transects 1 through 3) were calculated by fitting a least squares line through the elevation data between the crest and the toe. Table 2.9-4 summarizes the depth at structure toe and structure slope for each transect.

Wave input to the run-up calculations include the depth-limited wave height, which is the maximum wave height at the structure, and the peak wave period from the SWAN model result.

Wave runup for the NMP Units 1 and 2 shore protection structures was calculated using three empirical methods:

a. the van Gent method (which is currently recommended by FEMA for the evaluation of wave runup at barriers and structures in the Great Lakes);

b. Coastal Engineering & Design Analysis System (CEDAS) computer program which was developed by the USACE; and

c. the method developed by Delft Hydraulics from the Coastal Engineering Manual (USACE 2011e).

Input to the wave run-up calculations includes the depth-limited wave heights, deep water wave lengths, peak spectral period, depth limited wave heights, nearshore slope, wave period and structure slope.

The van Gent approach is appropriate for smooth and rough impermeable structures with varied uniform or composite slopes. The wave runup predicted by van Gent empirical equations is correlated to wave height and the surf similarity parameter, also known as the Irribarren Number. It is a dimensionless parameter that is used to describe the characteristics of wave phenomena and provides a useful metric for comparison of waves. It is a function of significant wave height, peak period, and structure slope. The van Gent equations are formulated to predict runup using input on shallow water wave conditions at the structure toe. Application of the van Gent equations predicts $R_{2\%}$, which is the runup exceeded by 2% of the runup events.

CEDAS is a comprehensive collection of coastal engineering design and analysis software, developed by or for the U.S. Army Engineer Waterways Experiment Station. The runup module of the CEDAS software program is based on the empirical runup equation developed by Waterways Experiment Station (Leenknecht et al. 1992). As with the van Gent equations it is correlated to the wave height and the surf similarity parameter. The program requires the selection of structure surface roughness coefficients. For the NMP Unit 1 Dike, the roughness coefficient for a rubblemound, two armor stone layer structure with an impermeable core was selected based on observed site conditions. For the NMP Unit 2 Revetment Ditch structure the roughness coefficient for a Dolosse armor layer structure was selected.

Wave runup on a rock armored impermeable slope was studied by Delft Hydraulics and is presented as Equation VI-5-12 in the Part VI Chapter 5 of the CEM (USACE 2011e). As with the other runup equations it is correlated to the wave height and the surf similarity parameter. Equation VI-5-12 allows the prediction of wave runup for various runup exceedance levels including the 2% and the 33% ("significant"). It is valid for relatively deep water in front of a structure where the wave height distribution is close to the Rayleigh distribution. The equation provides an upper limit for wave runup at NMP since the wave runup is based on deep water conditions at the structure toe and is based on an impermeable slope. The runup results for Equation VI-5-12 were adjusted for slope roughness since the Unit 1 structure slopes is highly irregular and is not representative of a typical interlocked armor slope, and the Unit 2 structure has Dolosse armor units. A surface roughness reduction factor of 0.75 (USACE 2011e) and 0.43 (FEMA 2012) were applied to the Unit 1 and Unit 2 results, respectively.

The results of the wave runup calculations for the six transects and three methods are presented in Table 2.9-5. The results show that the van Gent Equation predicts lower wave runup at the site. The results from CEDAS and Equation VI-5-12 are similar, with the predicted runup for Equation VI-5-12 being higher for Unit 1 dike and lower for Unit 2 Revetment/Drainage Ditch.

Since the wave runup heights from Equation VI-5-12 are dependent on deep water conditions at the structure toe, the use of the Equation for depth limited wave conditions will over predict runup since the models assume that the input wave heights are deep water waves (USACE 2011e). Therefore, the runup results from CEDAS were selected for the evaluation of potential wave overtopping of the NMP Units 1 and 2 shore protection structures.

2.9.2.4 Wave Overtopping

When the CEDAS predicted wave runup heights determined in Section 2.9.2.4 and summarized in Table 2.9-6 are added to the PMSS still water level of 252.83 feet (USLS35), the calculated runup elevations for NMP Unit 1 range from 260.23 to 263.0 feet (USLS35). This indicates that wave overtopping and potential landslide flooding can occur since the crest elevations exceed the crest elevation of the NMP Unit 1 Dike as shown on Figure 2.9-7 (Note elevations on Figure 2.9-7 are based on the NGVD29, however difference between NGVD29 and USLS35 is 0.03').

The calculated runup elevation for NMP Unit 2 ranged from 260.0 to 261 feet (USLS35), which is lower than the design crest elevation of 263.0 feet (USLS35) for the Unit 2 revetment ditch structure.

The wave overtopping calculations were performed using the CEDAS wave runup/overtopping option. CEDAS uses an empirical equation which is dependent on the wave runup, deep water wave height, structure height, freeboard. The wave overtopping calculations included the effects of wind-blown wave spray. The maximum wind speed of 100 mph developed in the PMWS calculation (AREVA 2012, PMWS) was used in the overtopping calculation. As a result, wave overtopping may occur even though the predicted wave runup elevation is less than the structure crest elevation.

The wave overtopping calculations for Unit 1 utilizes the same transects as the wave runup. To provide a conservative estimate of the potential wave overtopping the largest structure toe depth for the NMP 1 dike (10.1 feet) was used for all the wave overtopping calculations, which results in a significant wave height of 7.9 feet for all calculations. Except for the structure toe depth, the parameters used for the wave overtopping calculations were those used for the wave runup calculations. To account for the variation in the crest elevation of the Unit 1 Dike, the wave overtopping calculations were performed by segments along the structure crest, which were selected based on the crest elevation. Figure 2.9-7 shows the six segments developed for the wave overtopping calculation.

The results of the wave overtopping calculations, which are presented Table 2.9-6, show that during an extreme storm event wave overtopping of the Unit 1 dike will cause flooding of the drainage ditch at a rate of approximately 205 cubic feet per second (cfs). Because the predicted wave runup does not exceed the crest elevation of the Unit 2 revetment/ditch structure wave overtopping of the structure should not occur for the probable maximum storm event.

2.9.2.5 Hydraulic Modeling of Wave Overtopping at NMP Unit 1

To evaluate the potential flooding impacts of the wave overtopping of the Unit 1 dike, a computerized hydraulic model (HEC-RAS Version 4.1) was developed to evaluate water surface elevations in the drainage swale/ditch landward of the Unit 1 dike. The HEC-RAS model consists of a total of seven cross sections (Figure 2.9-15). Six cross sections (Stations 0 through 600) were developed along the south side of the Unit 1 dike and one additional cross section (Station -500) was developed approximately 500 feet offshore in Lake Ontario to represent the "downstream" boundary condition for the HEC-RAS model. The length of the HEC-RAS model is approximately 1,100 feet. Elevations used in the HEC-RAS model refer to the USLS 1935 datum.

<u>Model Geometry</u>: Cross section geometry was based on the topographic survey plan (C.T. Male 1999). Cross sectional geometry at Station -500 was estimated from the bathymetric contours shown in Figure 2.9-4. The topography south of the Unit 1 dike forms a swale or ditch, which appears to generally flow from east to west. The bottom of the swale ranges in elevation from approximately 250 to 252.6 feet (USLS35), well below the site grade elevation of 261 feet (USLS35).

<u>Manning's n-values</u>: A conservative Manning's "n" of 0.1 was assigned for Station 600 through Station 0 along the ditch, based on the typical ranges provided by HEC-RAS Reference Manual (USACE 2010) (Appendix H) for vegetated channels. A Manning's n-value of 0.04 was used for Station -500 (located in the Lake to represent a downstream boundary condition).

<u>Inflow</u>: Flow rates at each section were calculated based on the wave overtopping rates calculated in Section 2.9.2.4 and summarized in Table 2.9-7. The overtopping rates for each cross section station are based on the length along the structure times the per linear foot overtopping rate upstream of the cross section. Due to the grades behind the dike the flow increases from upstream (east) to downstream (west).

Cross Section Station No.	Flow (cfs)
600	0.1
560	28.0
480	53.6
280	162.0
120	189.2
0	204.5

A steady state flow calculation was performed using the option of "Mixed" for flow regime in HEC-RAS. The HEC-RAS calculated water surface elevation profile is shown in Figure 2.9-16. The maximum water surface elevation is approximately 258.1 feet (USLS35) for the upstream-most portion of the area, approximately 3 feet below entryways to safety-related SSCs and at least 2 feet below typical site grade.

2.9.3 Conclusions

NUREG/CR-7046, Appendix H, Section H stipulates that the following combination of flood effects applies to sites along the shores of enclosed bodies of water.

- Probable Maximum Storm Surge (PMSS) and Seiche with wind-wave activity.
- The lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

The wave runup heights calculated using the CEDAS software program for NMP Unit 2 ranged from 7.45 feet to 8.16 feet (Table 2.9-5) which are slightly higher than the Unit 2 CLB of 7 feet. The prior runup calculations used a composite slope method to evaluate the runup due to higher deepwater wave heights. The composite slope method uses an iterative method to approximate the effect of the stepped slope. The current calculations are based on the runup due to the maximum depth limited wave heights at the revetment toe and did not use the composite slope method. The differences in the wave runup methodologies are the likely cause of the increase in the wave runup heights for the Unit 2 Revetment /Ditch structure. Information for NMP Unit 1 runup calculations is not available so a comparison of results is not possible.

Based on the results presented In Section 2.9, the following conclusions can be reached:

- During an extreme storm, the associated waves combined with the PMSS design water elevation are conservatively predicted to cause wave runup that will overtop the Unit 1 shore protection structure. Overtopping of the Unit 2 shore protection structure is not expected due to wave runup dissipation from the Dolosse armor units which protect the seaward slope of the structure.

- Wave overtopping of the Unit 1 dike will result in localized flooding within the swale /ditch landward of the dike; however, the maximum calculated water surface elevation is approximately 258.1 feet (USLS35) for the eastern end of the swale/ditch, which approximately 3 feet below entryways to safety-related SSCs, which are at elevation 261feet (USLS35) and at least 2 feet below typical site grade.

Thus, the combination-effects flood hazard at the NMP site does not affect safety-related SSCs for either Unit 1 or Unit 2.

2.9.4 References

NOTE: Refer to the Project Manager's approval (on the signature page of this report) verifying that the Constellation Nuclear Energy Group (CENG) references are valid sources of design input created in accordance with the CENG's QA program.

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Table 2.9-1: Nine Mile Point Vertical Datum Conversions

		To				
		USLS35 (ft)	IGLD55 (ft)	IGLD85 (ft)	NGVD29 (ft)	NAVD88 (ft)
From:	USLS35	0	-1.23	-0.73	-0.03	-0.68
	IGLD55	1.23	0	0.5	1.2	0.55
	IGLD85	0.73	-0.5	0	0.7	0.05
	NGVD29	0.03	-1.2	-0.7	0	-0.66
	NAVD88	0.68	-0.55	-0.05	0.66	0

Table 2.9-2: Summary of Extreme Wave Conditions Estimated at the USACE WIS Stations

WIS Station	Depth (Ft.)	Range of Significant Wave Height for Top 10 Events (Ft.)	Range of Peak Wave Periods for Top 10 Events (s)
Station 91134	56	17.1 to 19.7	9.3 to 11.1
Station 91136	187	18 to 22.3	9.3 to 10.3
Station 91137	138	17.4 to 21.6	9.3 to 10.3
Station 91138	102	17.4 to 21.3	9.3 to 11.1

Notes 1. See Figure 2.9-1 for location of WIS Stations.

Table 2.9-3: Summary of Shallow Water Wave Characteristics

SWAN Output Point	Depth (Ft.)	H_s (Ft.)	T_p (s)	H_{max} (Ft.)	T_{max} (s)	PMWS T_p Range (s)	PMWS Peak Wave Direction (° counter clockwise from E)
Shallow Water Point 1	14.8	8.2	10.4	11.5*	12.5	9.4 to 11.9	305 to 320
Shallow Water Point 1A	30.8	12.5	10.4	21.0	12.5	9.0 to 11.4	320 to 327
Shallow Water Point 2	26.2	11.8	10.4	19.4	12.5	9.0 to 11.9	320
Shallow Water Point 3	20.0	9.8	10.4	15.7*	12.5	9.4 to 11.9	305 to 312

Notes
1. See Figure 2.9-4 for location of Shallow Water Points.
2. T_p is the peak spectral wave period associated with the maximum significant wave height.
3. H_s is the average height of the maximum 1/3 of all waves
4. * Indicates that H_{max} is limited by depth

Table 2.9-4: Summary of Transect Characteristics

Transect	Start Point (degrees)	End Point (degrees)	Structure Slope (tan α)	Depth at Toe (Ft.)
Transect 1	43.522053 N 76.412295 E	43.522394 N 76.412608 W	0.45	7.5
Transect 2	43.522588 N 76.411097 W	43.522949 N 76.411369 W	0.41	9.9
Transect 3	43.522717 N 76.410562 W	43.523103 N 76.410755 W	0.5	10.1
Transect 4	43.522862 N 76.410153 W	43.523273 N 76.410348 W	0.5	9.0
Transect 5	43.523063 N 76.40758 W	43.523461 N 76.407727 W	0.5	10.0
Transect 6	43.523294 N 76.406368 W	43.523691 N 76.406522 W	0.5	8.7

Note :
1. See Figure 8 for location of Transects.
2. The structure slope and depth at toe are determined from Figures 11 through 16

Table 2.9-5: Wave Runup Calculation Results for NMP Units 1 and 2

Transect (1)	T Peak (T_p)	Structure Toe Depth (Ft.) (2)	Hs @ Toe (Ft.)	Tangent Structure Slope	Surf Similarity Parameter (SSP) for Peak Wave Period (T_p) (3)	SSP for Mean Wave Period (Tm) (4)	2% Runup for CEM Eq. VI-5-12 (5)	2% Runup for CEM Eq. VI-5-12 (Adj for Roughness) (Ft.)	Runup for CEDAS Wave Runup Program (Ft.)	2% Runup for van Gent Equation (Smooth Slope) (Ft.) (6)
1	10.4	7.5	5.85	0.45	4.38	3.59	12.33	9.24	7.72	1.20
2	10.4	9.9	7.72	0.41	3.47	2.85	14.62	10.97	9.28	2.00
3	10.4	10.1	7.88	0.5	4.19	3.44	16.27	12.2	10.18	1.69
4	10.4	9.0	7.02	0.5	4.44	3.64	14.89	6.40	7.45	1.42
5	10.4	10.0	7.80	0.5	4.21	3.46	16.15	6.94	8.16	1.67
6	10.4	8.7	6.79	0.5	4.52	3.70	14.5	6.24	7.25	1.35

Notes: 1. See Figure 8 for location of Transects.

2. Hs equals Depth at toe times 0.78

3. SSP is the surf similarity parameter, equal to: $\tan\alpha/\sqrt{Hs/Lo}$

4. Wave period used in calculation is Tm which equals SSP times 0.82.

5. CEM Eq. VI-5-12 is $\dfrac{R_{ui\%}}{H_s} = \begin{cases} A\,\xi_{om} & \text{for} \quad 1.0 < \xi_{om} \le 1.5 \\ B(\xi_{om})^C & \text{for} \quad \xi_{om} > 1.5 \end{cases}$

6. van Gent Equation is $\dfrac{R_{2\%}}{\gamma H_s} = \begin{cases} c_0\xi & \xi \le p \\ c_1 - c_2/\xi & \xi > p \end{cases}$

Table 2.9-6: NMP Unit 1 Wave Overtopping Summary

Reach	Reach Length (Ft.)	Average Structure Crest Elev., (Ft.) (USLS35)	Cotan Structure Slope	Design Base Water Level (Ft.) (USLS35)	Depth at Structure Toe (Ft.)	Total Structure Height	T Peak (T_p) (s)	H_s (Ft.)	H_o (Ft.) (CEDAS Calc.)	H_o/gT^2	CEDAS Runup (Ft.)	Runup Elev. (USLS35) (Ft.)	CEDAS Over-topping (CuFt/Sec-Ft.)	Over-topping Reach (CuFt/Sec)
1	250	263.5	2.2	252.8	10.1	20.8	10.4	7.9	6.3	0.0018	9.8	262.6	0.17	42.5
2	40	261.5	2.4	252.8	10.1	18.8	10.4	7.9	6.3	0.0018	9.3	262.1	0.54	21.6
3	110	262.5	2	252.8	10.1	19.8	10.4	7.9	6.3	0.0018	10.2	263.0	0.38	41.8
4	90	262	2	252.8	10.1	19.3	10.4	7.9	6.3	0.0018	10.2	263.0	0.5	45.0
5	40	261.6	2	252.8	10.1	18.9	10.4	7.9	6.3	0.0018	10.2	263.0	0.64	25.6
6	80	262.6	2	252.8	10.1	19.9	10.4	7.9	6.3	0.0018	10.2	263.0	0.35	28.0
													Total	204.5

Note: 1. See Figure 17 for segment locations

2. Total Structure Height equals Average Structure Crest Elev. minus Design Base Water Level plus Depth at Structure Toe.

3. H_o is deep water wave height associated with H_s which is calculated by CEDAS.

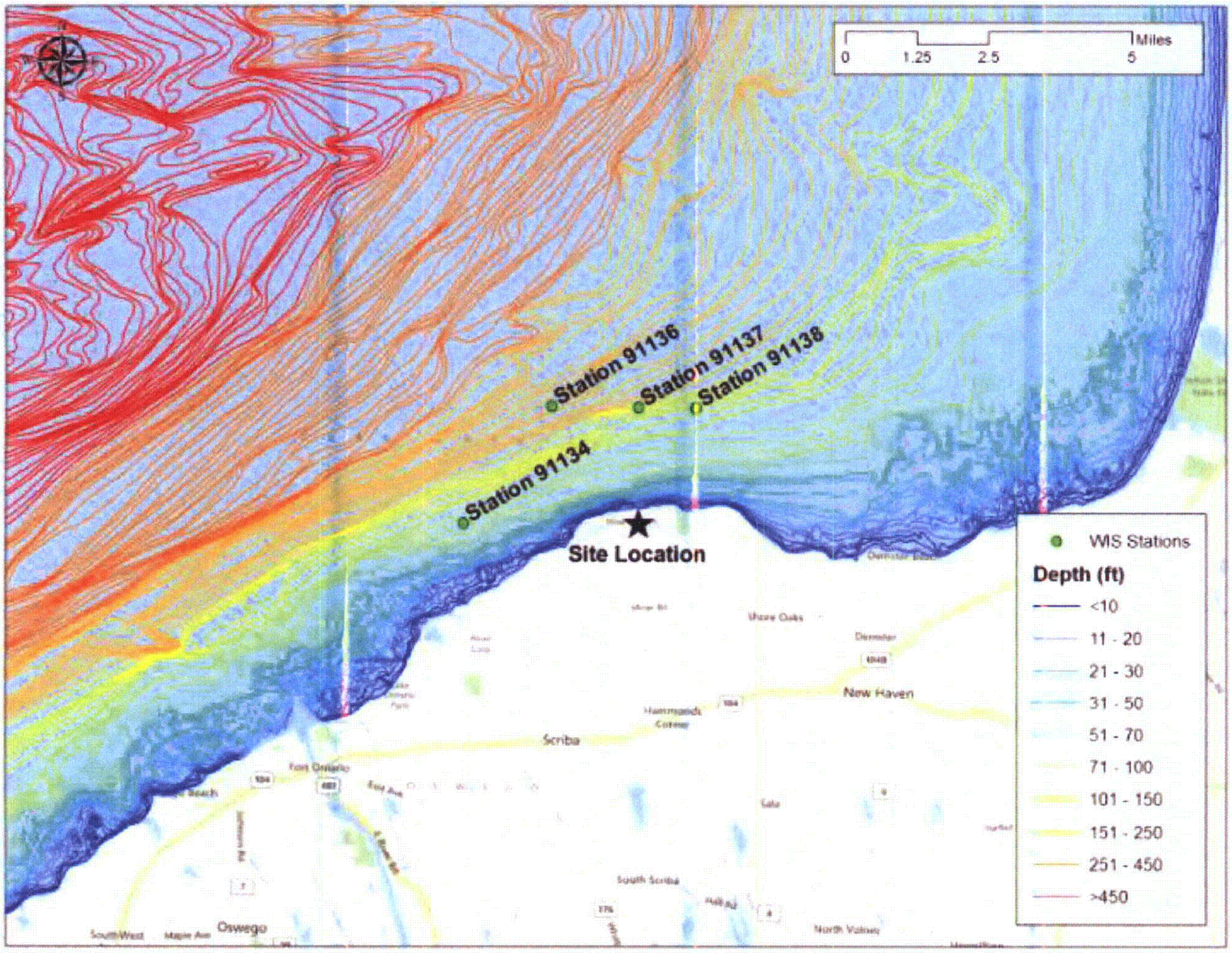

Figure 2.9-1: Site locus showing location of site on Lake Ontario and WIS wave stations offshore of NMP Units 1 and 2. Depth references to NOAA Lake Ontario Low Water Datum, 243.4 Ft. NAVD88.

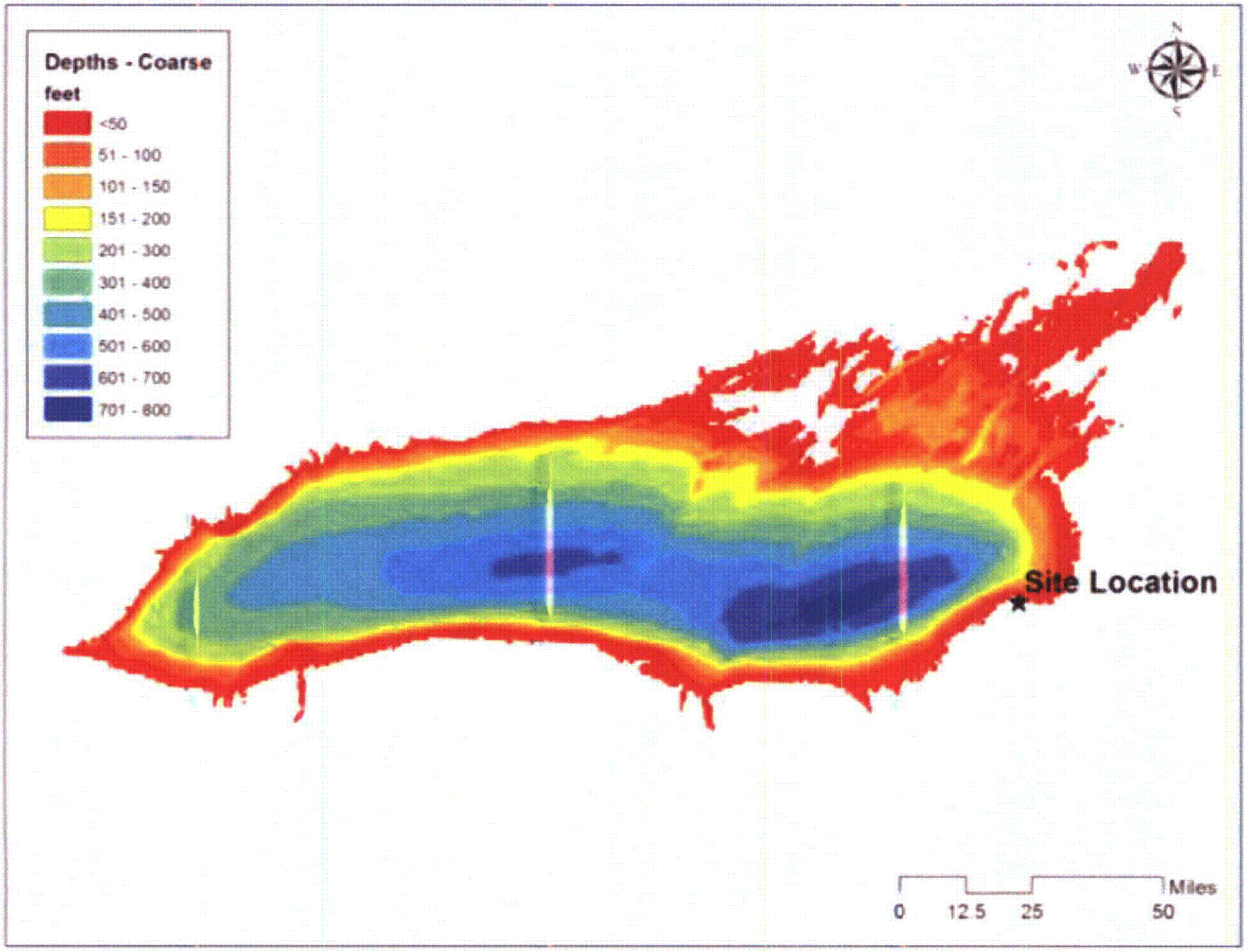

Figure 2.9-2: Depths used for 0.01 degree resolution SWAN grid (Virden, W.T. 1999). Location of NMP Units 1 and 2 marked in black (depths referenced to elevation 252.8 feet NGVD 29).

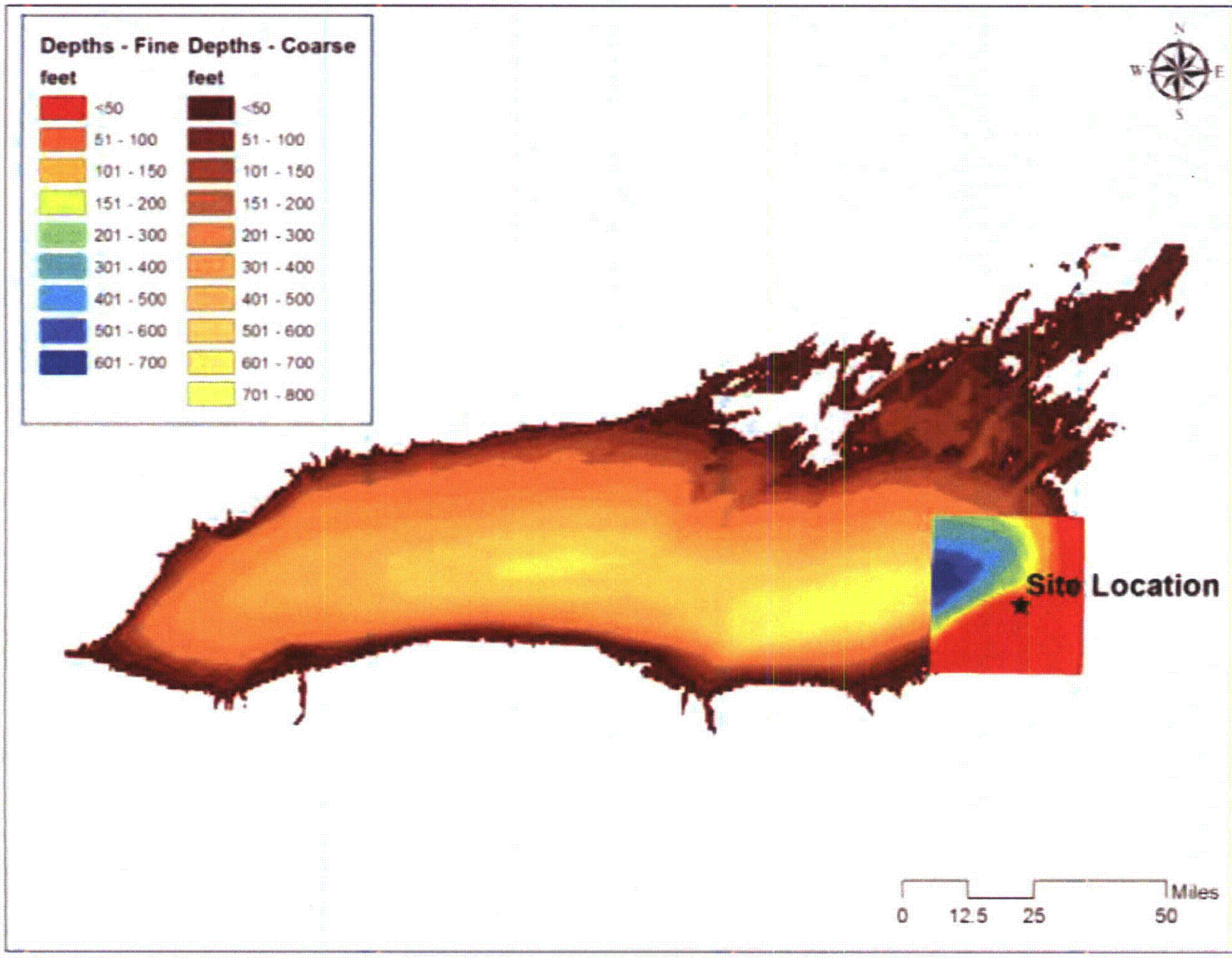

Figure 2.9-3: Depths used for 0.002 degree resolution nested SWAN grid (Virden, W.T. 1999). Location of NMP Units 1 and 2 marked in black (depths referenced to elevation 252.8 feet NGVD 29).

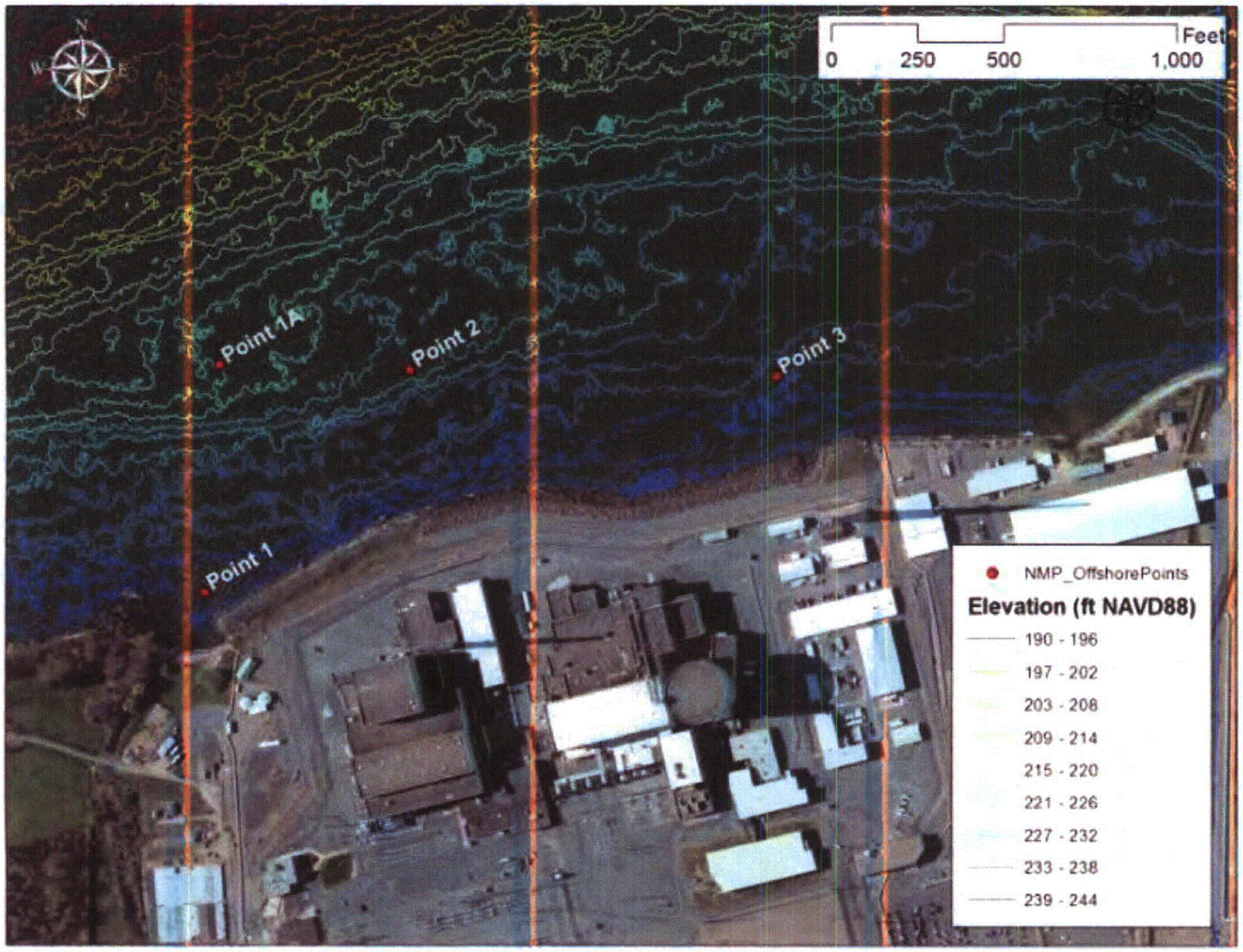

Figure 2.9-4: Nine Mile Point Shallow aerial photograph with NOAA bathymetry and location of shallow water wave points.

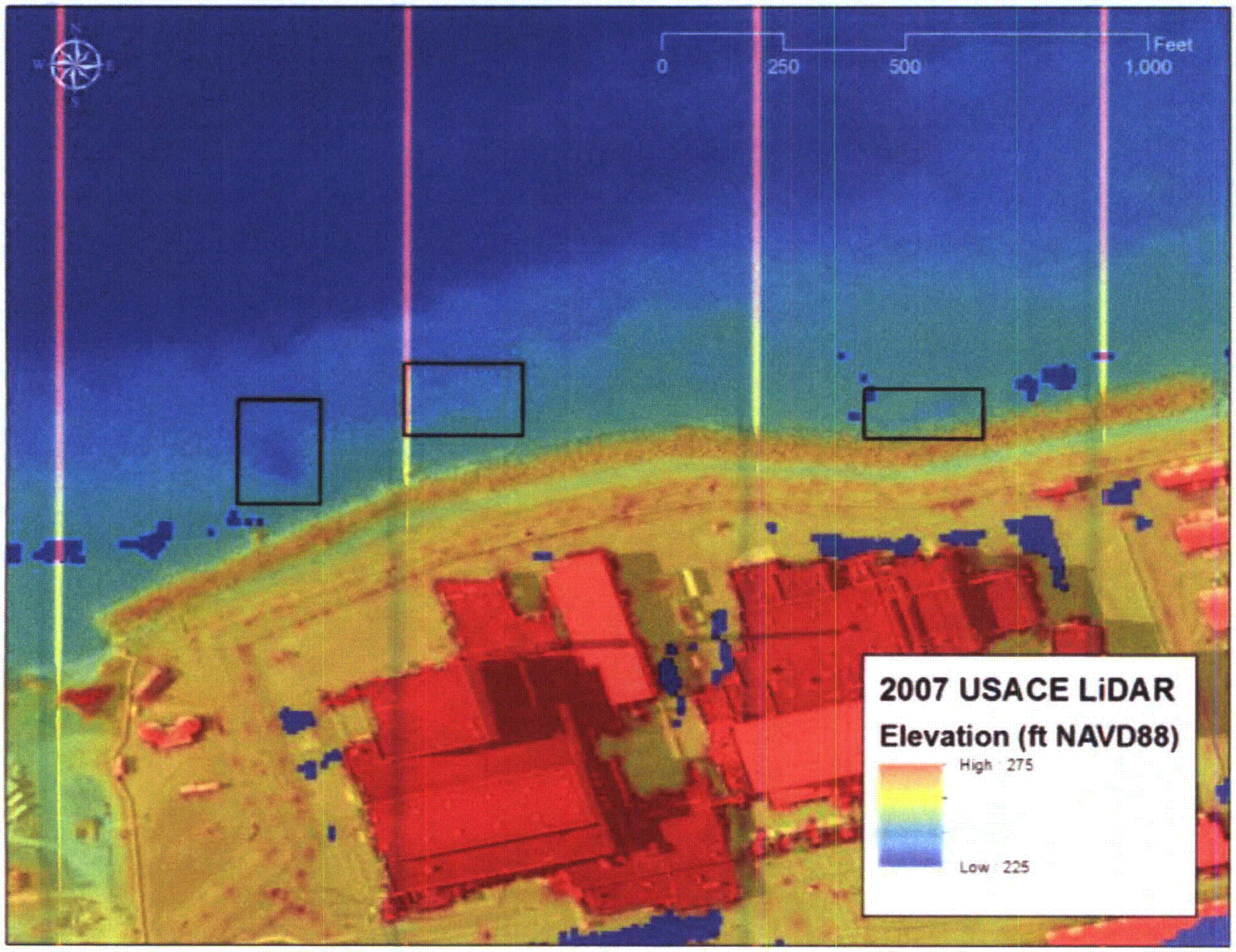

Figure 2.9-5: 2007 USACE Great Lakes Topo/Bathy LiDAR data in the vicinity of NMP units 1 and 2 (USACE 2007). Black outlines show areas of deep water. Areas of missing data shown in dark blue.

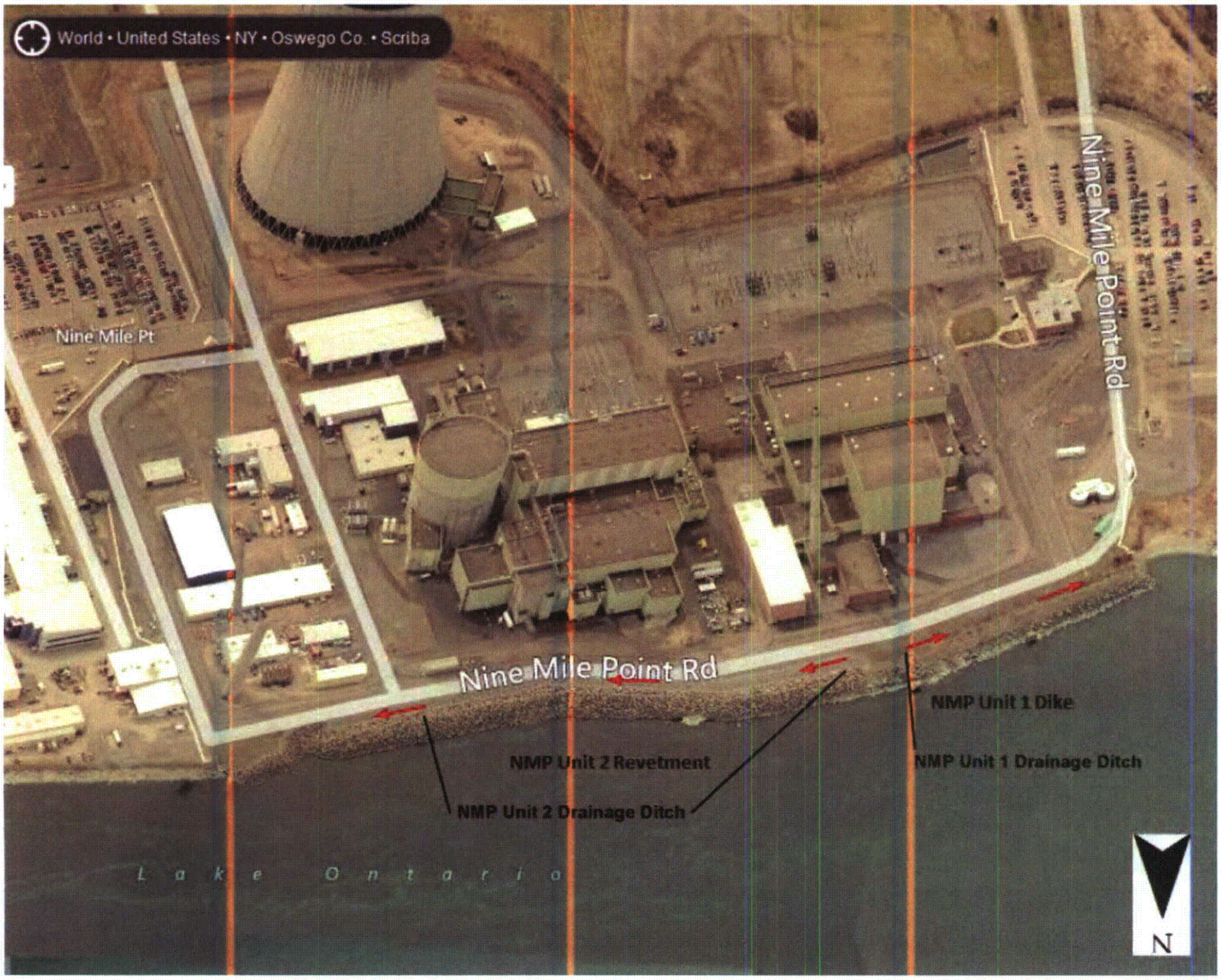

Figure 2.9-6: Oblique aerial imagery depicting the shoreline protection structures for NMP Units 1 and 2.

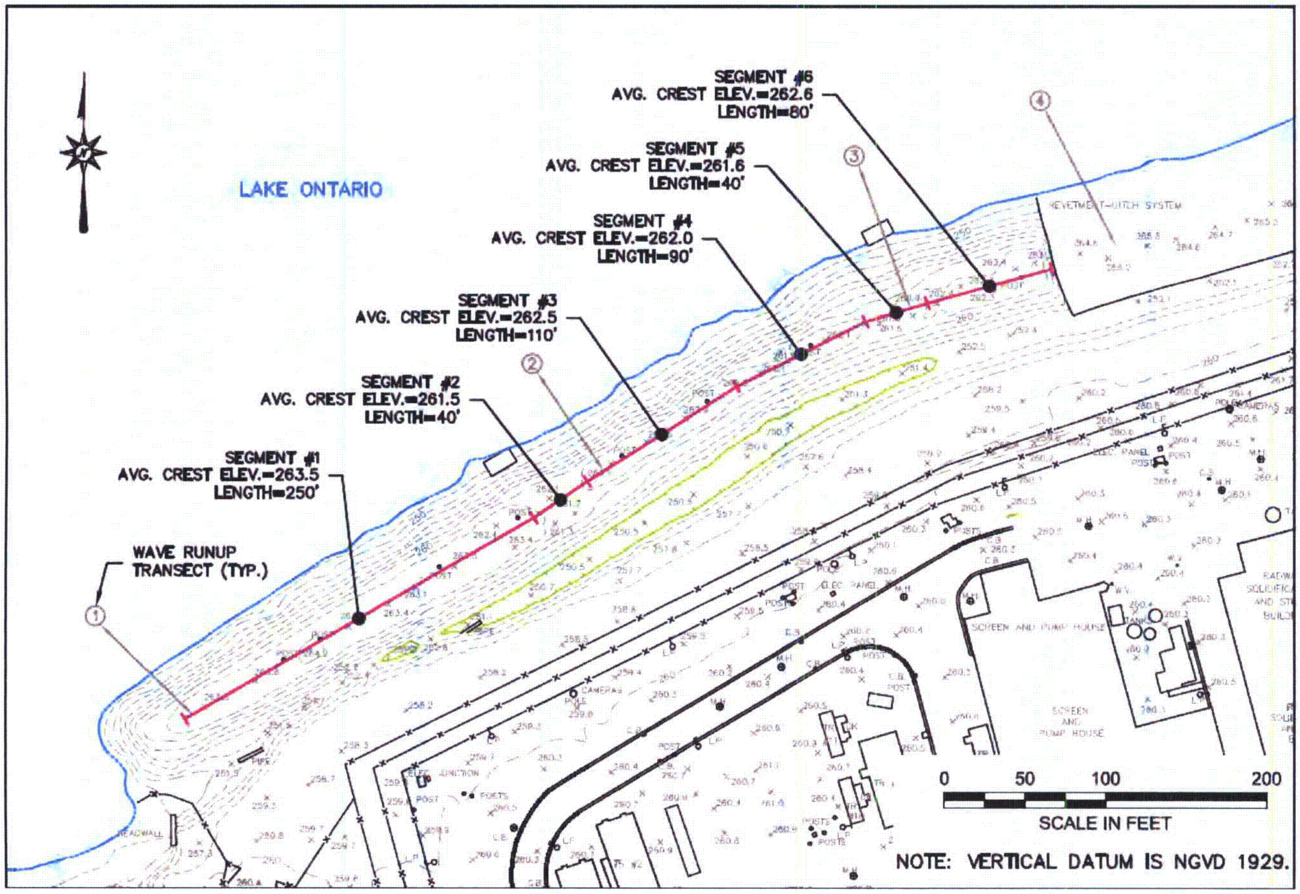

Figure 2.9-7: NMP Unit 1 shoreline showing dike, wave runup transect locations and wave overtopping calculation reach data.

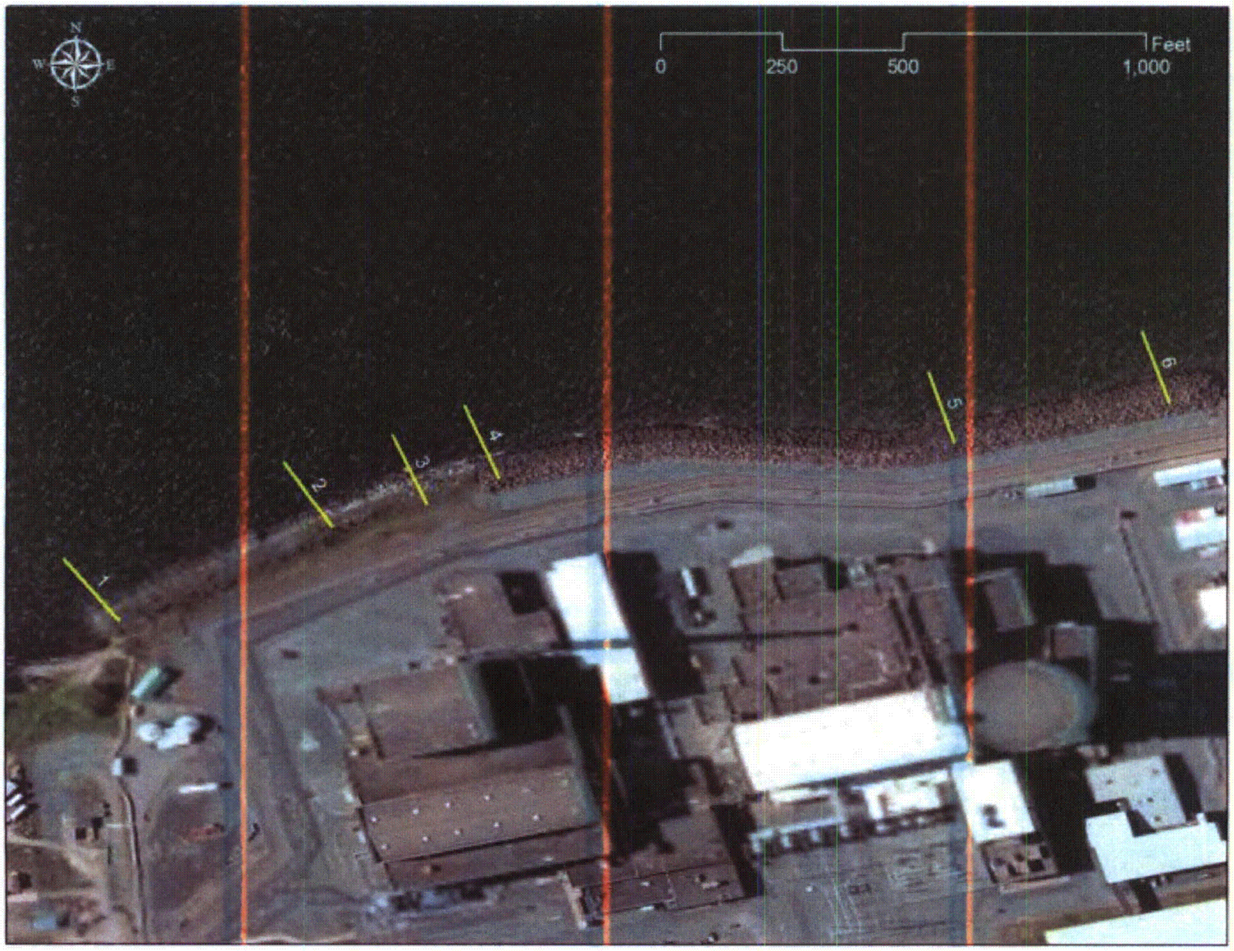

Figure 2.9-8: Transects (shown in yellow) along NMP units 1 and 2 dike/revetment system used for wave runup calculations. Imagery provided by Bing Maps (ESRI 2012).

Figure 2.9-9: Elevation profiles along each Transect 1; Still Water Elevation (SWEL) from PMSS equals 252.15 feet NAVD88. Least-squares fit to structure slope indicated by solid black line.

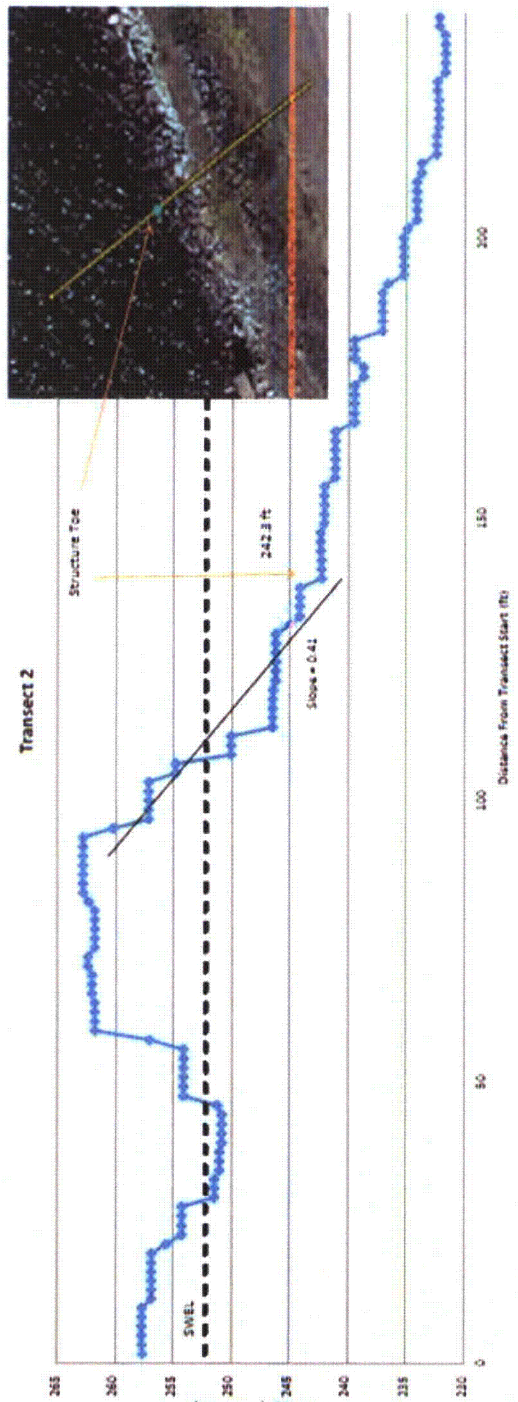

Figure 2.9-10: Elevation profiles along each Transect 2; Still Water Elevation (SWEL) from PMSS equals 252.15 feet NAVD88. Least-squares fit to structure slope indicated by solid black line.

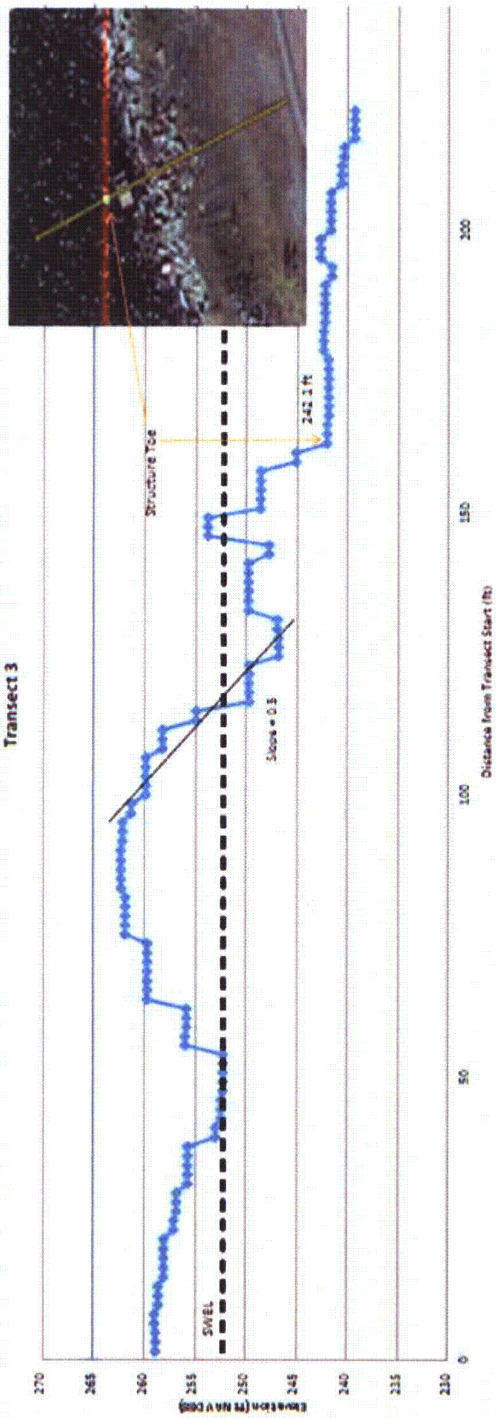

Figure 2.9-11: Elevation profiles along each Transect 3; Still Water Elevation (SWEL) from PMSS equals 252.15 feet NAVD88. Least-squares fit to structure slope indicated by solid black line.

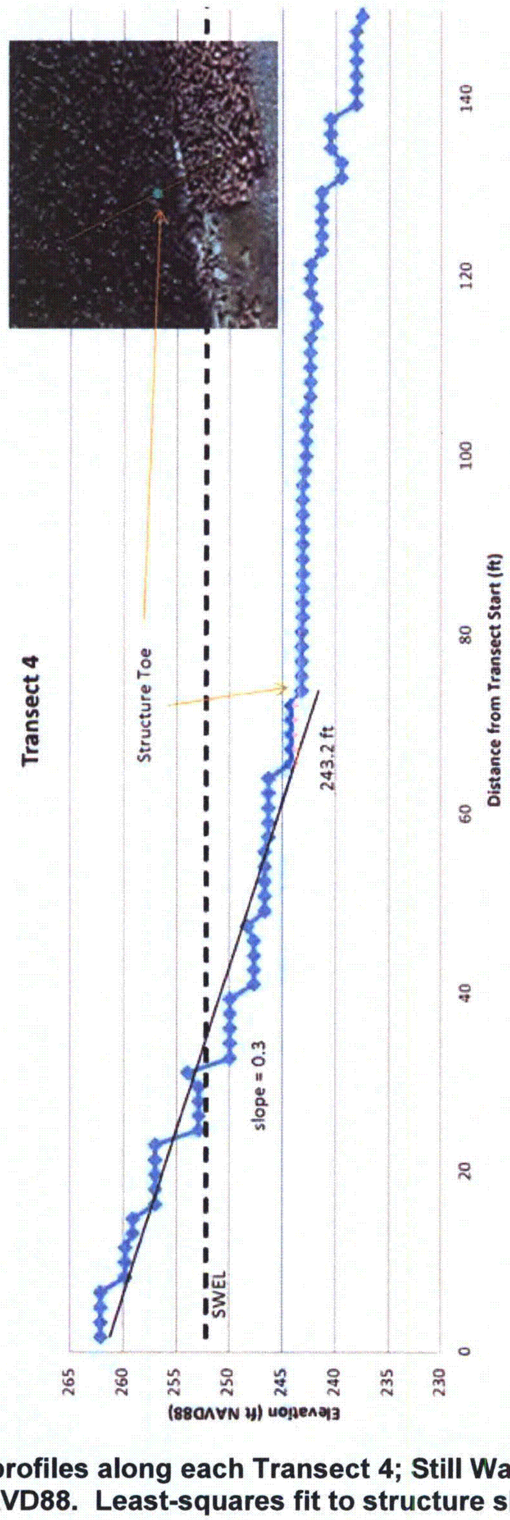

Figure 2.9-12: Elevation profiles along each Transect 4; Still Water Elevation (SWEL) from PMSS equals 252.15 feet NAVD88. Least-squares fit to structure slope indicated by solid black line.

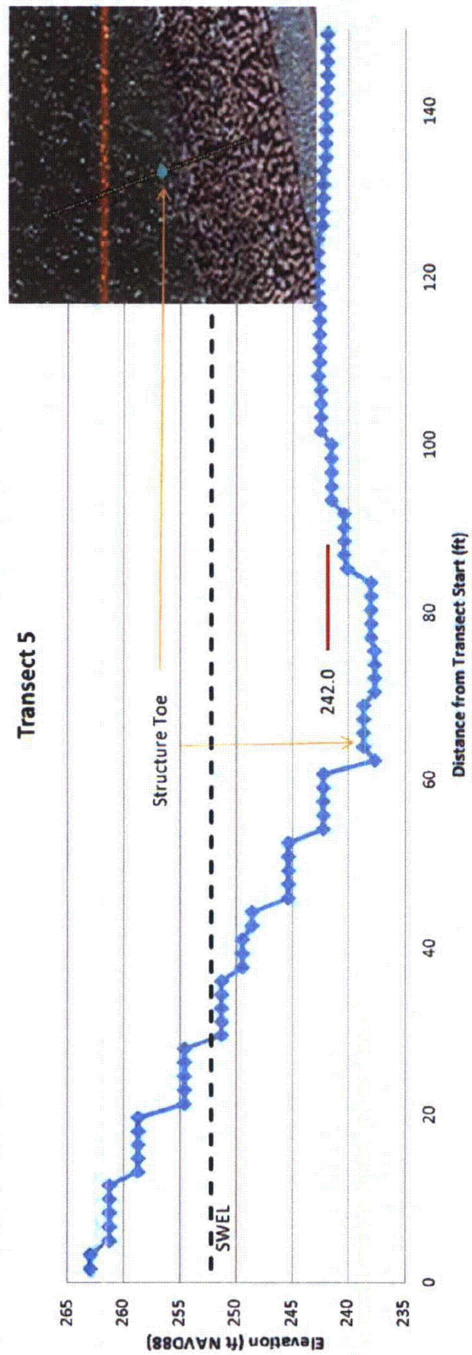

Figure 2.9-13: Elevation profiles along each Transect 5; SWELfrom PMSS equals 252.15 feet NAVD88. Least-squares fit to structure slope indicated by solid black line. Note: Structure toe depth based on offshore depth of 242 which limits incident wave heights.

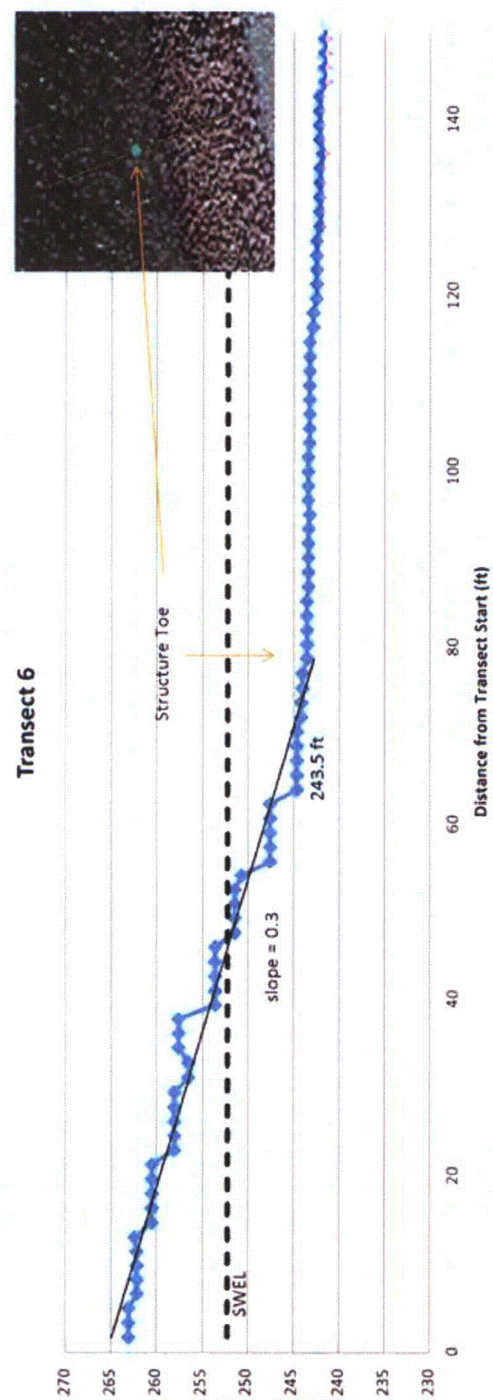

Figure 2.9-14: Elevation profiles along each Transect 6; Still Water Elevation (SWEL) from PMSS equals 252.15 feet NAVD88. Least-squares fit to structure slope indicated by solid black line.

Figure 2.9-15: HEC-RAS Cross Section Locations along Unit 1 Dike Drainage Ditch

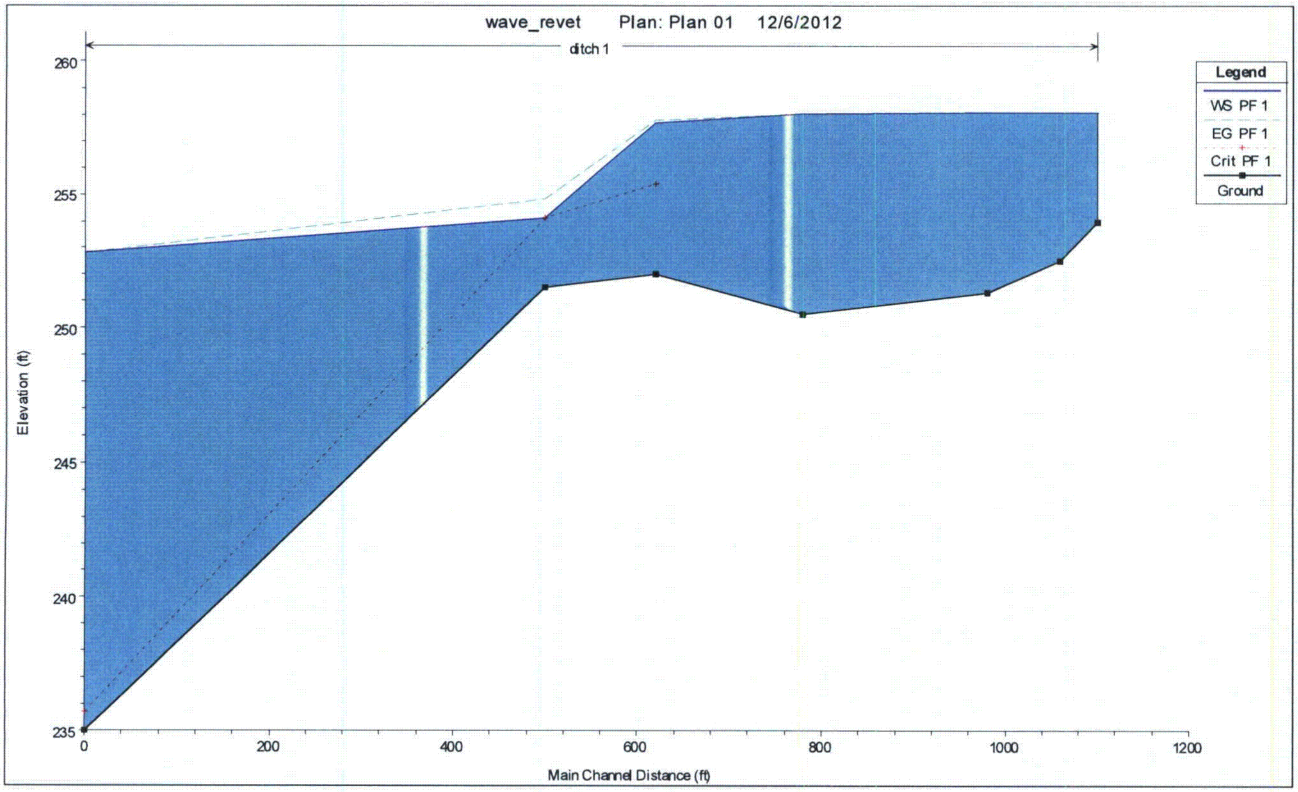

Figure 2.9-16: HEC-RAS Calculated Profile (Vertical Datum USLS35)

3.0 COMPARISON OF CURRENT AND REEVALUATED FLOOD CAUSING MECHANISMS

3.1 Summary of Flood Reevaluation Results

This section compares the current and reevaluated flood-causing mechanisms. It provides an assessment of the current licensing basis flood elevation to the reevaluated flood elevation for each applicable flood-causing mechanism.

3.1.1 Local Intense Precipitation

In the immediate vicinity of NMP Units 1 and 2, the calculated maximum flood elevation of 262.2 ft (USLS35) for Unit 1 and elevation 262.4 ft (USLS35) for Unit 2 exceed the design flood elevation 261 ft (USLS35) for all Category I structures as per Unit 2 USAR (USAR 2010), for up to approximately 20 hours during the 72-hour PMP.

Results indicate higher water elevations, up to elevation 263.7 ft (USLS35) between the non-safety-related structures east of Unit 2, between buildings such as NMP Warehouse, Site Services Building and Change House.

See Section 2.1 for detailed information.

3.1.2 Flooding on Rivers and Streams

Lakeview Creek is the only perennial stream in the site vicinity. The calculated PMF flow in Lakeview Creek is calculated to be 17,290 cfs. Flooding on Lakeview Creek has no impact on the NMP site due to the watershed layout and separation from the NMP site.

See Section 2.2 for detailed information.

3.1.3 Dam Breaches and Failures

The effects resulting from the hypothetical failure of the six dams/locks simultaneously in the Oswego River would produce an insignificant increase in the water level on Lake Ontario of approximately 0.2 inches.

The effects of lower than normal Lake Ontario water levels due to the failure of the dams at the outlet of Lake Ontario in the St. Lawrence River have been considered in plant design. A margin of 12.6 ft. and 8.1 ft. exists above the tops of the NMP Unit 1 and Unit 2 intakes, respectively, to account for the lowest water level projected.

Potential dam breaches and failures in the site region would not affect the SSCs important to safety at either NMP Unit 1 or Unit 2.

See Section 2.3 for detailed information.

3.1.4 Storm Surge

The predicted PMSS elevation is 252.8 ft (USLS35), based on a PMSS height of 4.8 ft.

NMP Unit 1 and Unit 2 are protected with a shore protection dike (Unit 1) and a revetment dike and drainage ditch (Unit 2). The elevations of the top of the revetment dikes, and the plant grade behind the dikes are above the predicted PMSS still water elevation; therefore, impact to NMP Unit 1 and Unit 2 structures due solely to the surge water level are not predicted.

See Section 2.4 for detailed information.

3.1.5 Seiche

Lake seiches of significant amplitude are not a unique coastal hazard on Lake Ontario, since their occurrence is a result of the oscillating response to a storm surge.

The predicted Probable Maximum Storm Surge (PMSS) resulting from the PMWS is presented in Section 2.4 and is 4.8 ft. Oscillations resulting from the PMWS surge are expected to have amplitudes less than 4.8 ft, resulting in a Probable Maximum Seiche below elevation 252.8 ft (USLS35).

See Section 2.5 for detailed information.

3.1.6 Tsunami

As an inland site, the NMP site is not subject to oceanic tsunamis; however, tsunami-like waves (seiches) have occurred.

The following mechanisms capable of causing a tsunami-like wave are unlikely to impact the site as indicated below:

- an earthquake is limited because the required level of seismic activity for development of a tsunami, i.e., an earthquake with a magnitude greater than 6.5, is essentially absent from the region;
- a subaqueous landslide is unlikely to generate an observable tsunami-like wave due to the limited bathymetric relief of ridges and their respective slopes; and
- a subaerial landslide is unlikely to occur due to limited topographic relief. The one area with sufficient topographic relief, Scarborough Bluffs near Toronto, is oriented such that the direction of a landslide and resultant tsunami-like wave, if it occurred, would be toward the southeastern lake shoreline, more than 150 miles west of the NMP site.

Notwithstanding the occurrence of tsunami-like waves, the potential effects on the NMP site (wave runup and draw down) are negligible because there is sufficient available physical margin to protect SSCs important to safety. The physical margin is based on the maximum recorded tsunami-like wave resulting from an earthquake in the Great Lakes region occurring coincident with the maximum (runup) and minimum (drawdown) lake levels.

See Section 2.6 for detailed information.

3.1.7 Ice-Induced Flooding

Based on the historical records, ice induced flooding at the NMP site is unlikely and would not affect the SSCs important to safety.

The nearest historical ice jams data on record occurred on the Oswego River, which is more than 6 mi from the NMP site.

The ISLRBC reduces the Lake Ontario outflow to promote the formation of a smooth, stable ice cover on the St. Lawrence River, which is beneficial in that it reduces the risk of ice jams on the river.

There are no major streams close to or on the site that would contribute to the potential of ice induced flooding at the NMP site.

See Section 2.7 for detailed information.

3.1.8 Channel Migration or Diversion

There is very limited potential for upstream diversion or rerouting of Lake Ontario (due to channel migration, river cutoffs, ice jams, or subsidence) to adversely impact safety-related facilities or water supplies. In addition, there are no perennial streams or rivers in the NMP watershed. The closest stream, Lakeview Creek, is outside the NMP watershed.

See Section 2.8 for detailed information.

3.1.9 Combined Effect Flooding

The combined effects evaluation consisted of the following flood mechanisms:

- Probable Maximum Storm Surge (PMSS) and Seiche with wind-wave activity; and
- the lesser of the 100-year or the maximum controlled water level in the enclosed body of water.

During an extreme storm, the associated waves combined with the PMSS design water elevation are conservatively predicted to cause wave runup that will overtop the Unit 1 shore protection structure. Overtopping of the Unit 2 shore protection structure is not expected due to wave runup dissipation. Wave overtopping of the Unit 1 dike will result in localized flooding within the swale /ditch landward of the dike; however, the maximum calculated water surface elevation is approximately 258.1 ft (USLS35), which approximately 3 ft below entryways to safety-related SSCs and at least 2 ft below typical site grade.

Thus, the combination-effects flood hazard at the NMP site does not affect safety-related SSCs for either Unit 1 or Unit 2.

See Section 2.9 for detailed information.

3.2 Summary of Walkdown Findings

The walkdown report (CENG 2012) indicates that flooding of SSCs can occur during the CLB LIP flood event. CLB LIP flood event is based on a 20-minute precipitation event, which has a limited duration of flood water intrusion into SSCs at both NMP Unit 1 and Unit 2. See Section 1.3 for detail information.

3.2.1 NMP Unit 1

All structure penetrations below elevation 261 ft (USLS35) are sealed against water. The CLB-LIP flood event had a maximum elevation of 261.75 ft (USLS35) with a duration of 20 minutes. Flooding of buildings is assumed to occur when flood waters exceed an elevation of 261 ft (USLS35). Consequently, the flood event could lead to a loss of offsite power and diesel generator failure, if the water in the diesel rooms rose to 261.75 ft (USLS35). Based on the limited duration of 20 minutes and volume of flood water intrusion during the CLB LIP event, the impact of LIP flooding to the SSCs was determined to not present a significant hazard.

Based on the limited duration of 20 minutes and volume of flood water during the CLB LIP event, the impact of LIP flooding to the SSCs was determined not to present a significant hazard due to the low probability of the loss of offsite power and time available to recover a diesel generator before battery depletion by the NRC during the IPEEE response (see Section 4.3.1).

3.2.2 NMP Unit 2

All structure penetrations below elevation 261 ft are sealed against water. The CLB-LIP flood event had a maximum elevation of 262.5 ft, with a 20-minute duration. Based on the limited duration and volume of flood water intrusion during the CLB LIP event, impact of LIP flooding to the SSCs was determined to be negligible with the exception of the Diesel Generator Building. Due to significant flooding from the CLB LIP event, flexible caulking material protects the Diesel Generator Building up to 263 ft.

3.3 Impacts of Flood Elevations

A comparison of the CLB flood elevations for each flooding mechanism to the new evaluation flood elevations is provided in Table 3.3-1.

The controlling reevaluated flood mechanism for NMP Unit 1 and Unit 2 is the Local Intense Precipitation event, as detailed in Section 2.1 of this report. Flood elevations at specific buildings are shown in Table 3.3-2. Based on the LIP evaluation, flooding of SSCs may occur as flood waters exceed elevation 261 ft for both Unit 1 and Unit 2, which is modeled to occur for as long as 20 hours, depending on the specific location on-site.

Based on the flood reevaluation, the maximum flood elevation from the Local Intense Precipitation (Section 2.1) event exceeds the CLB for NMP Unit 1 in both water surface elevation and duration. Based on the flood reevaluation, the LIP CLB elevation for NMP Unit 2 is not exceeded, however, the duration of the reevaluated event is significantly longer which may result in additional impacts to safety-related SSCs. See Sections 4 and 5 for further discussion.

Table 3.3-1: Flood Elevation Comparison

Unit	Flood Mechanism	CLB Flood Water Elevation (ft) (USLS35)	New Flood Water Elevation (ft) (USLS35)
Unit 1	Local Intense Precipitation	261.75 (IPEEE)	262.2 (max)
	Flooding on Rivers and Streams	NA	NA, screened
	Dam Breaches and Failures	NA	NA, screened
	Storm Surge	NA	252.8
	Seiche	NA	Bound by PMSS
	Tsunami	NA	NA, screened
	Ice-Induced Flooding	NA	NA, screened
	Channel Migration or Diversion	NA	NA, screened
	Combined Effect	NA	258.1
Unit 2	Local Intense Precipitation	262.5	262.4 (max)
	Flooding on Rivers and Streams	NA, screened	NA, screened
	Dam Breaches and Failures	NA, screened	NA, screened
	Storm Surge	254.0	252.1
	Seiche	NA, screened	Bound by PMSS
	Tsunami	NA, screened	NA, screened
	Ice-Induced Flooding	NA, screened	NA, screened
	Channel Migration or Diversion	NA, screened	NA, screened
	Combined Effect	261.0	258.1

Table 3.3-2: LIP Flood Elevations Summary

Unit	Building Identification	Maximum Water Elevation (ft) (USLS35)	Maximum Flow Depth (ft) (USLS35)
Unit 1	Reactor Building & Turbine Building	261.1 to 262.1	0.6 to 1.5
	Radwaste Building, Screenwell Building & Offgas Stack Building	260.8 to 262.0	0.2 to 1.3
	Administration Building	262.2 ±	1.7 to 2.4
Unit 2	Reactor Building	262.0 to 262.4	1.6 to 2.8
	Turbine Building & Switchgear Building	262.2 ±	2.5 ±
	Radwaste Building, Screenwell Building & Offgas Stack Building	260.6 to 261.9	0.6 to 1.8
	Condensate Storage Building	261.4 to 262.1	1 to 1.5
	Control Building & Diesel Generator Building	262.3 ±	1.6 to 2.6
East of Unit 2	Access Building & Maintenance Building	262.6 ±	0.6 to 2
	Operations Building	262.3 to 263	0.4 to 2.8
	Change House	262 to 262.5	1.3 to 2.7
	Site Services Building	262.4 to 263.7	2.7 to 4.3
	NMP Warehouse	262 to 263.3	1 to 2.5
	East Security Building	263.2 ±	0.5 to 2.7

4.0 INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED

4.1 Regulatory Background

The NRC 50.54(f) letter of March12, 2012 provides that flood hazard reevaluations are performed using present-day regulatory guidance and methodologies applicable to new nuclear plant applications. Therefore to the extent that existing plant flooding evaluations did not use these assumptions and methods, any issues identified during reevaluations should be treated as neither CLB deficiencies nor vulnerabilities. Plant-specific vulnerabilities are defined by NRC (NRC 2011) as those features important to safety that when subject to an increased demand due to the newly calculated hazard evaluation have not been shown to be capable of performing their intended safety function(s). Such vulnerabilities are beyond the CLB for the facility and do not call into question operability. However, vulnerabilities identified during the flood hazard reevaluations should be entered into the problem identification/corrective action process and dispositioned accordingly. If the reevaluated flood hazard at a site is not bounded by the current design basis, licensees are requested to perform an Integrated Assessment, per NRC direction.

In general, discrepancies identified during the flood hazard reevaluations (i.e., reevaluation results that indicate a concern with the design or licensing basis of the plant) are dispositioned similar to discrepancies identified during the conduct of walkdowns. The following additional information should be considered:

1) Flood hazard reevaluations are being performed in two phases. In Phase 1 of the 50.54(f) letter process, flood hazards are reevaluated using present-day regulatory guidance and methodologies applicable to new plant applications. If the reevaluated hazard is not bounded by the design basis flood at the site, licensees must perform an integrated assessment for external flooding. During Phase 2 of the 50.54(f) letter process, NRC staff will use the Phase 1 results to determine whether additional regulatory actions are necessary (e.g., update the CLB and SSCs important to safety).

2) All flooding reevaluation results that indicate existing flood protection features are not adequate to protect the plant from the reevaluation hazard should be entered into the problem identification/corrective action process. These conditions will also be evaluated as part of the Integrated Assessment whose results will be reported to the NRC within two years after submittal of the reevaluation report.

4.1.1 Reportability/Interim Actions

Plant-specific vulnerabilities based on new hazard assessments are conditions beyond the CLB and do not call into question operability, and need not be reported to the NRC pursuant to 10 CFR 50.72 or 10 CFR 50.9. NRC notification of the flooding reevaluation results and any actions taken in response to them will be reported in the Reevaluation Report and the Integrated Assessment, if necessary, required by the 50.54(f) letter.

The 10 CFR 50.54(f) Request For Information dated March 12, 2012 requires that interim actions be identified for all plant-specific vulnerabilities discovered during the flood hazard reevaluations. These actions are intended to provide a level of assurance the plant will be safe during a flood event until that time when the total plant response to the reevaluated hazard is determined by the Integrated Assessment and any necessary long term actions are identified.

In most cases, a PMP event will not cause an immediate flooding concern. Thus, the predicted times between the initiating event and the time of adverse impact could be a key consideration in determining the appropriateness of compensatory actions.

4.2 Interim Evaluation and Actions Taken or Planned For NMP

Section 2.0 of this report contains the reevaluation of flood hazards at NMP1 and NMP2 using present-day regulatory guidance and methodologies. Section 3.0 of this report summarizes the comparison of the current licensing basis flood elevation to the reevaluated flood elevation for each applicable flood-causing mechanism. Only one reevaluated flood mechanism, Local Intense Precipitation (LIP) for both NMP1 and NMP2, exceeded the design basis flood, and is discussed below. For NMP2, the assumed flood duration above elevation 261 ft (USLS35) has significantly increased. This will impact the amount of water ingress into safety related structures. For NMP1, the flood duration and flood elevation height have increased. This will impact the NMP1's IPEEE evaluation on vulnerabilities associated with the external flooding event.

Since the LIP for NMP1 and NMP2 is not bounded by the design basis flood at the site, then the site must perform an integrated assessment for external flooding. An interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis must be described prior to completion of the integrated assessment.

4.3 Comparison of NMP1 and NMP2 Design Basis Flood to LIP NMP1 Reevaluation

4.3.1 Current Design Basis-NMP1 and NMP2

NMP1 was designed and built prior to the requirements presented in the NRC Standard Review Plan (SRP) criteria for external floods (NUREG-75/087). Therefore, the evaluation and documentation to satisfy the SRP external flooding criteria was not required. However, the NMP1 Individual Plant Examinations for External Events (IPEEE) process was used to find vulnerabilities with respect to the SRP external flooding criteria. Various possible flood scenarios were considered and information from calculations for NMP2 were used to show that the only flooding scenario of concern for the plant was one involving a probable maximum precipitation (PMP) event. The NMP1-LIP flood event had a maximum elevation of 261.75 ft (USLS35), with a duration of 20 minutes. This value includes the assumptions that the stormwater drainage system is inoperable and that the culverts located southwest of the NMP1 switchyard are not blocked, allowing water to flow from offsite onto the plant site.

The personnel entrance and equipment access to buildings important to safety are provided at or above elevation 261 ft (USLS35). Once the flood level exceeds elevation 261 ft (USLS35), water may seep into the buildings through the doors. Per the NMP1 IPEEE there is a potential for flooding in the Diesel Generator Building if water level reaches 261.75 ft (USLS35). Based on the limited duration of 20 minutes and volume of flood water during the CLB LIP event, the impact of LIP flooding to the SSCs was determined not to present a significant hazard due to the low probability of the loss of offsite power and time available to recover a diesel generator before battery depletion by the NRC during the IPEEE response (NRC 1999).

The rate of stormwater inflow into the NMP2 buildings from the PMP were based on 20-minute duration above elevation 261 ft (USLS35). NMP2 USAR Table 2.4-15 provides a summary of results analysis of building flooding due to the PMP based on HMR 51 and 52 based on the 20 minute duration (NMP 1984).

4.3.2 LIP/PMP Reevaluation

The maximum calculated flood elevation and flow depth at NMP occur during the 72-hour PMP as described in Section 2.1. The use of the 72-hour duration PMP for the NMP reevaluation is based on the example Case 3 done in Appendix B of NUREG/CR-7046. This example was applicable to the site due to the presence of onsite NMP drainage features delineated on the USGS 1980 topographic quadrangle map. Despite the man-made changes to these features, one is currently physically connected to the upstream drainage area south of Lake Road by a culvert. The LIP-induced flood is the result of the PMP centered over the site area and the local watershed. Flooding of these small drainages was expected to occur coincident with the LIP; therefore, they were also included in this analysis, i.e. the FLO-2D model applied the rainfall in the watershed outside of the site.

Based on the LIP calculation for the NMP site the following conclusions are reached:

- The maximum LIP flood elevation at NMP is evaluated based on a 72-hour PMP. Results of the 6-hour PMP simulation are given in Section 2.1. In general, the 72-hour PMP with a 20-hour flood duration above 261 ft (USLS35) yields flood elevations up to approximately 0.6 ft higher than the results from the 6 Hour PMP simulation with a 14-hour flood duration above 261 ft (USLS35).
- In the immediate vicinity of NMP1 and NMP2, the maximum water surface elevations predicted by the FLO-2D model are up to elevation 262.4 ft (USLS35) and are similar to the previously calculated CLB LIP-PMP elevation 262.5 ft (USLS35) presented in NMP2 USAR (USAR 2010), but slightly higher than the CLB LIP-PMP elevation of 261.75 ft (USLS35) for Unit 1.
- The calculated maximum flood elevation 262.2 ft (USLS35) for Unit 1 exceed the design flood elevation 261 ft (USLS35) for all Category I structures as per NMP2 USAR (USAR 2010), for up to approximately 20 hours during the 72-hour PMP. The 20-hour duration exceeds the NRC accepted duration of 20 minutes for the Unit 1 Diesel Generator Building. The flood elevation for NMP2 is slightly lower, but the time duration above elevation 261 ft (USLS35) for both units is above the NRC accepted duration of 20 minutes

4.4 Interim Actions

Interim actions will be taken by NMP1 and NMP2 to provide a level of assurance the plants will be safe during a flood event until that time when the total plant response to the reevaluated hazard is determined by the Integrated Assessment.

The PMP event does not cause an immediate flooding concern at NMPNS. The time between the prediction of a PMP event and the potential flooding event will be greater than 24 hours giving the plant time to initiate potential flood mitigation measures. Operating Procedures N1-OP-64 (NMP1) and N2-OP-102 (NMP2), Meteorological Monitoring (NMP 2012a, NMP 2012b) provides the response plan for potential multiple meteorological events. The NMP2's procedure invokes Procedure EPIP-EPP-26, Natural Hazard Preparation and Recovery (CENG 2012). These procedures would need to be reviewed and updated as determined by the Integrated Assessment.

4.5 Interim Evaluation and Actions Planned to Reduce Flood Elevations and Duration from LIP

The plan to reduce flood elevation for NMP1 and reduce the flood duration above elevation 261 ft consists of modification of drainage systems and flood mitigation measures. Blocking of the culvert that connects the NMP site to the offsite watershed area south of Lake Road will allow for reduction of the assumed PMP event from 72 hours to 6 hours based on NUREG/7046. Under this modification, the site is physically separated from the watershed; therefore, a lower PMP event duration is used per NUREG/7046 guidance. In addition, modification to onsite drainage systems will reduce the flood elevations during a PMP as well as further reduction of the flood duration. Finally, the shorter, 6 hour, duration PMP will be mitigated by new temporary or permanent flood mitigation measures.

4.5.1 Modifications of NMP Site Drainage Systems

Modifications to the NMP site drainage system would consist of blocking the Lake Road Culvert 5 and removing Culverts 2 and 3 and replacing them with an open channel (Figure 4-1) (AREVA 2013). The Lake Road culvert is approximately 1.5 ft in diameter with insignificant flows; therefore blocking of the Lake Road culvert would have small impacts to the nearby offsite roads and drainage areas. Creating an open channel would improve the site drainage and would have minimal impact to site operations. Neither modification is expected to generate significant environmental permitting issues.

The following modifications to the site are reflected in FLO-2D model described below:

1. The culvert penetrations at Lake Road were removed/blocked, disconnecting the watershed area south of Lake Road. As a result, the 6-hour PMP hyetograph constructed as per Figure B-5 in NUREG/CR-7046 was used as the precipitation input.
2. Four culverts in the drainage channel were modeled as hydraulic structures within FLO-2D using estimated headwater depth vs. discharge rating tables. The conditions used for the culverts are as follows:
    a. Culverts 1 and 4 are 100% blocked, as per the conceptual model of Case 3 in NUREG/CR-7046, Appendix B.
    b. Culverts 2 and 3 are considered to be removed and replaced with an open channel.

The small flow rate of water from the watershed area south of Lake Road (and now to be blocked from entry through the Lake Rd culverts) will need to be examined to determine the impacts to Lake Rd drainage areas.

4.5.1.1 Impact of Site Drainage System Modifications on Flood Elevations

The duration above elevation 261 ft (USLS35) for both units will decrease from 20 hours to 2 hours based on the proposed site drainage system modifications. However, this is still above the CLB of 20 minutes for both units. The LIP flood elevation will exceed floor elevation by 0.4 ft (USLS35) (approximately five inches) for Unit 1, and the time duration above elevation 261 ft (USLS35) for both units will be 2 hours; however, the time above elevation 261 ft (USLS35) is still greater than 20 minutes. The plant will either take temporary or permanent measures to mitigate the flooding into safety related structures.

4.5.2 Flood Mitigation Measures

The LIP flood elevation will exceed the floor elevation of the Unit 1 diesel generator building rollup doors by 0.4 feet (approximately five inches) for 2 hours. The plant will either take temporary or permanent measures to mitigate the flooding into the building.

4.5.2.1 Temporary Flood Mitigation Measures

The calculated flooding duration change and the flooding elevation change identified during the flood hazard reevaluation will impact the amount of water ingress into both units. Station procedures will be revised to direct installing temporary flood protection measures to protect essential station equipment from the reevaluated flood event. Reasonable simulation practices will be used to perform a walk-through of the procedures in order to validate the procedures can be executed by station personnel as specified, using the prescribed tools and equipment, within the anticipated time frames at the following locations:

- NMP1: Diesel Generator 102 and 103 room roll-up doors, Power Board 103 room, Battery Board 11 and 12 rooms, and Valve Board 11 room exterior doors
- NMP2: Control Building, Electrical Tunnel, and Standby Gas Treatment room exterior doors

4.5.2.2 Permanent Measures

Permanent flood mitigation measures do not require action by plant personnel during a flood event. Permanent measures could consist of construction of berms around the diesel roll up doors on NMP1, and for both units replace building exterior doors to water tight doors or construct water-tight barriers in front of the affected doors that house equipment required for safe shutdown.

Permanent flood mitigation measures will be determined during the integrated assessment for external flooding.

Figure 4-1 Culvert Locations

4.6 References

NOTE: Refer to the Project Manager's approval (on the signature page of this report) verifying that the Constellation Nuclear Energy Group (CENG) references are valid sources of design input created in accordance with the CENG's QA program.

AREVA 2013. AREVA Document No. 32-9199660-000, Additional FLO-2D Simulation of Local Intense Precipitation-Induced Flooding at Nine Mile Point, 2013.

CENG 2012. Nine Mile Station Procedure EPIP-EPP-26, Natural Hazard Preparation and Recovery.

NMP 1984. Calculation No. WH-C-001, Stormwater Inflow into Buildings from PMF.

NMP 2012a. Operating Procedure N1-OP-64, Rev. 00602, Meteorological Monitoring .

NMP 2012b. Operating Procedure N2-OP-102, Meteorological Monitoring .

NRC 1999. NRC Technical Evaluation Report, The High Winds, Floods, Transportation and Other Events (HFO) Portion of the Nine Mile Point Unit 1 IPEEE Submittal for NRC Generic Issue GI 80-20, 1999.

NRC 2011. U.S Nuclear Regulatory Commission, "Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights From the Fukushima Dai-ichi Accident," Enclosure 4, "Recommendation 2.3: Flooding (ADAMS Accession No. ML12056A050).

USAR 2010. Nine Mile Point Unit 2 - Updated Safety Analysis, Revision 19, October 2010.

5.0 ADDITIONAL ACTIONS

None.

APPENDIX A: SIMULATION MODEL USE DESCRIPTIONS

This appendix was prepared as per Sections 5.3 and 5.5 of NUREG/CR-7046 (NRC 2011).

A.1 FLO-2D Computer Program – 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 NMP and PMF in streams and rivers near NMP. For the LIP calculation, rainfall runoff was calculated internally by FLO-2D and translated into overland flow within FLO-2D.

This appendix was prepared as per Sections 5.3 and 5.5 of NUREG/CR-7046 (NRC 2011).

A.1.1 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

*.exe File	Size
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.1.2 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 ft) 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.1.3 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):

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

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

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.1.4 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 softwares 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.1.5 Model Validation

As per Section 5.5 of NUREG/CR-7046 (NRC 2011), accuracy of computer models should be validated using site-specific data. Historical observed flood flow / elevation data at NMP is not available. In lieu of site-specific data, the validation of the FLO-2D software used two benchmark case studies presented in the FLO-2D model validation report (FLO-2D 2011). FLO-2D's model validation report has gained acceptance from a variety of federal, state, and local regulatory agencies and the model itself has been accepted by FEMA. Example 1, a simple flume model, was validated by comparing the results with a hand calculation as shown in Table A-1:

Table A-1: Comparison of Results – Example 1

METHOD OF COMPUTATION	FLO-2D v.2009.06	HAND CALCULATION
Flow Depth (ft)	6.8	6.8
Velocity (ft/s)	5.9	5.9

Example 2 was a case study for the Truckee River performed by FLO-2D (FLO-2D 2011). The Truckee River FLO-2D model was originally created and calibrated by others to conduct a flood hazard delineation project for the Truckee River in response to recorded flooding of the Truckee River through Reno and the City of Sparks, Nevada between December 31, 1996 and January 6, 1997. The simulated results by FLO-2D were compared with observed USGS gage data during an actual storm. See Table A-2 and Figure A-1. Upon achieving the identical output results as presented in the FLO-2D model validation report, it was concluded that the model validation was completed to the extent practicable.

Table A-2: Comparison of Results – Example 2

	Node	Benchmark	GZA	% Difference
Maximum Flow Depths (ft)	4936	0.1	0.1	0.0
	4937	6.25	6.25	0.0
	4938	9.09	9.09	0.0
	4968	0.1	0.1	0.0
	4969	4.62	4.62	0.0
	4970	10.75	10.75	0.0

	Node	Benchmark	GZA	% Difference
	4998	4.32	4.32	0.0
	4999	9.02	9.02	0.0
	5000	8.77	8.77	0.0
	5022	3.99	3.99	0.0
	5023	3.98	3.98	0.0
	5024	7.08	7.08	0.0
Total Inflow and rainfall Volume (acres)		101028	101028	0.0
Total Outflow and Storage (acres)		101028	101028	0.0
Maximum Inundated Area (acres)		21125	21125	0.0

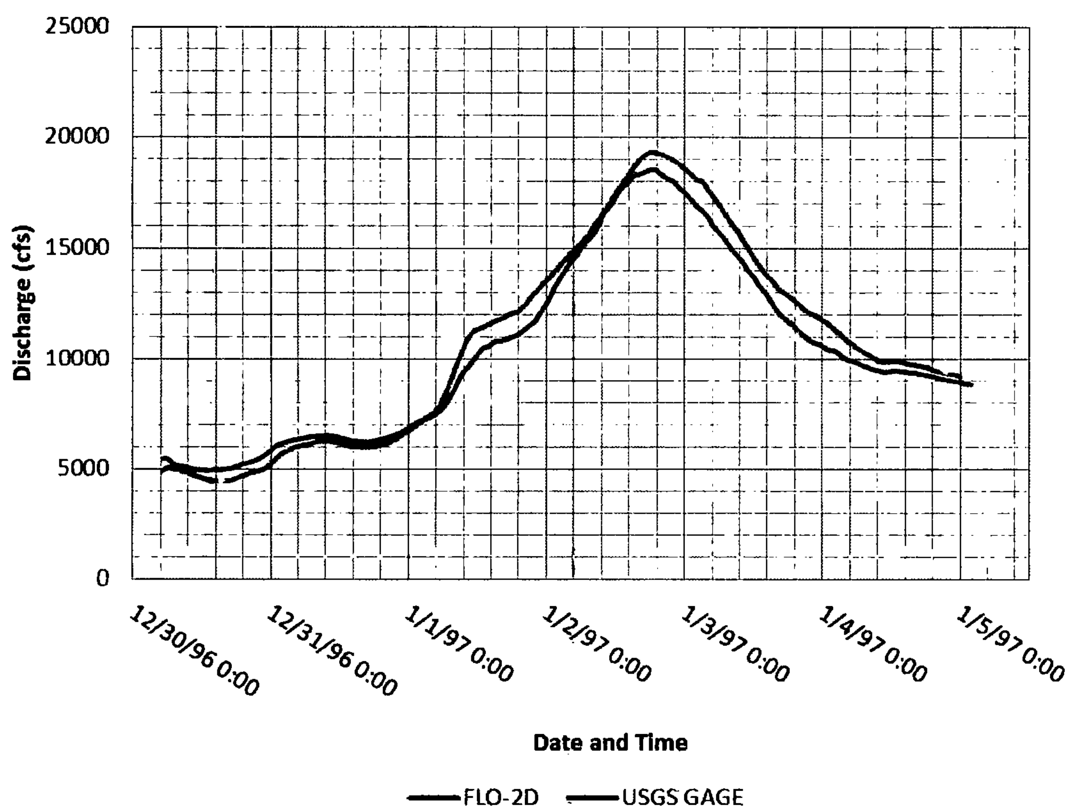

Figure A-1: Observed vs. Predicted Discharge for the 1997 Flood at Truckee River New Vista Gage

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 NMP flood re-evaluation study where 2-dimensional overland flow is predominant.

A.1.7 References

NOTE: Refer to the Project Manager's approval (on the signature page of this report) verifying that the Constellation Nuclear Energy Group (CENG) references are valid sources of design input created in accordance with the CENG's QA program.

AREVA 2012. AREVA Document No. 38-9191747-000, Computer Software Certification – FLO-2D v.2009.06, 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.

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

USACE 2000. HEC-HMS Hydrologic Modeling System Technical Reference Manual, HEC, USACE, March 2000.

USACE 2010. HEC-RAS River Analysis System Hydraulic Reference Manual, HEC, USACE, January 2010.

A.2 SSPP Computer Program – SSPP for Probable Maximum Storm Surge (PMSS) Simulations

The Great Lakes Storm Surge Planning Program Version SSPP v1.4 was designed to simulate the storm surge water surface elevation maximums and minimums on the Great Lakes. SSPP v1.4 was developed by the NOAA Great Lakes Environmental Research Laboratory (GLERL) and has been used by NOAA to calculate maximum and minimum surge heights and elevations for ten areas of the Great Lakes, including Central and Eastern Lake Ontario, Western Lake Erie, Lake St. Clair, Lake Huron, Saginaw Bay, the eastern shore of Lake Michigan, the western shore of Lake Michigan, Green Bay and Lake Superior.

Its intended use on this project is to conservatively simulate storm surge elevations for the Probable Maximum Storm Surge (PMSS) in the vicinity of NMP Units 1 and 2, located along the southeastern shore of Lake Ontario.

The following SSPP v1.4 documentation is filed with the project records:

- SSPP v1.4 User Manual, August 1987

The source code is readily available and distributed by the software vendor. See web page:

http://www.glerl.noaa.gov/ftp/publications/tech_reports/glerl-065/

A.2.1 Software Capability

The SSPP v1.4 computer program was developed by NOAA GLERL in response to requests for a surge planning program from institutions such as the Michigan Department of Natural Resources, the Ohio Department of Natural Resources, the US Army Corp of Engineers, the Wisconsin Sea Grant Institute Advisory Services, the National Park Service, and the St. Lawrence - Eastern Ontario Commission.

The SSPP predicts minimum and maximum water level elevations at select shoreline locations for a given constant wind speed and direction (NOAA Technical Memorandum GLERL-65; Schwab, 1987). The SSPP uses lake water level impulse-response functions calculated from the response of a two dimensional, dynamic numerical model of the Great Lakes (Schwab 1978 and 1981). Water level responses for 15 points in each of ten areas are stored in DATA statements in the program. The responses are multiplied and superposed according to the input wind speed and direction. The maximum level for each of the 15 points during the 12 hours following the onset of the wind is then tabulated. Water level responses have been stored in SSPP for ten areas: (1) Lake Ontario, (2) Central and Eastern Lake Erie, (3) Western Lake Erie, (4) Lake St. Clair, (5) Lake Huron (except Saginaw Bay), (6) Saginaw Bay, (7) Eastern Shore of Lake Michigan, (8) Western Shore of Lake Michigan, (9) Green Bay, and (10) Lake Superior.

The following executable module composes the SSPP computer program:

*.exe File	Size
sspp.exe	65.0 K

SSPP.exe is written in BASIC computer language. The program can be downloaded or executed from Reference 1.

A.2.2 SSPP Model Theory

The SSPP v1.4 is based on the results of a two-dimensional dynamic numerical storm surge model developed by Schwab (Schwab 1978 and 1981). The SSPP uses lake water level impulse-response functions calculated from the response of the two dimensional, dynamic numerical model. The two dimensional numerical model is a linear hydrodynamic model based on depth-integrated shallow water equations. It includes coefficients for friction, Coriolis, pressure gradient and transport. Since the model is based on linear dynamics, the response to an impulsive wind stress from any direction and of any magnitude can be synthesized from the eastward and northward results by linear superposition. The response functions are calculated from a 1 dyn cm-2 spatially invariant impulse.

The SSPP impulse response functions are used to determine time-varying storm surge heights from time-constant lake-average wind forcing. Separate delta function impulses for the east-west and north-south directions define the water level responses and are superimposed to determine the total water level response at the location for which the impulse response functions are defined. The water level response takes the form of a convolution of Green's function (g) with a uniform forcing (τ) as represented by:

$$h_k = \sum_{i=1}^{m} \sum_{j=1}^{n} \vec{g}_{ij} * \vec{\tau}_{ik-j}$$

where h_k is the station dependent water level at time k, g_{ij} is the water level response at time j due to an impulse from forcing station i, $\vec{\tau}$ is the forcing function at station i and time k-j, m is the number of forcing stations, and n is the length of the response function. The surface wind stress forcing ($\vec{\tau}$) is proportional to the square of the wind speed used as forcing as:

$$\vec{\tau}_{ij} = \rho_a c_d \left|\vec{v}_{ij}\right| \vec{v}_{ij}$$

where c_d is the coefficient of drag over the lake surface, ρ_a is the density of air (assumed constant in space and time) and v_{ij} is wind velocity at station i and time j. The resulting time series of water levels provides an event maximum value for the water level during the modeled storm, based on the wind driven surge.

The two dimensional hydrodynamic model utilizes a 3.1 mile (5 kilometer) grid for Lake Ontario with uniform, impulsive wind stress from the west and the south. Using these eastward and northward results, the response to a wind stress of any magnitude, from any direction, can be calculated using linear superposition. The SSPP automatically calculates the maximum and minimum water level during the 12 hours following the onset of the wind, assuming that the wind speed and direction remains constant during that period. The SSPP also assumes that the wind speed and direction is spatially uniform.

A.2.3 Model Inputs and Outputs

Inputs to SSPP are entered through a graphical user interface (GUI), which then provides outputs in the DOS command prompt. The outputs can then be imported to a text editor.

The inputs to the SSPP model include:

1. File Name
2. Selection of the Great Lake to be modeled
3. Ambient Lake Level
4. Wind speed in miles per hour. This should be a representative overwater wind speed for neutral stability conditions at 10 m above the water surface.
5. The direction that the wind is blowing from. An entry of 0 degrees corresponds to a wind blowing from the north, 90 corresponds to a wind from the east, 180 from the south, and 270 from the west.

The outputs to the model include:

1. the maximum and minimum hourly water levels at each of the 15 points (for the period of 12 hours after the onset of the wind).

A.2.4 Model Limitations

As per Section 5.5 of NUREG/CR-7046 (NRC 2011), accuracy of computer models should be validated using site-specific data and is described in the literature below. The methodology of the SSPP v1.4 impulse response model, which is based on the response functions derived from the two dimensional hydrodynamic model, has been verified in Lake Erie (Schwab 1978). The verification shows that the impulse-response methodology has good agreement (low RMS error) for hindcasts of historical events and tends to overestimate the magnitude of the water level response. The hydrodynamic model itself is based on a version of the Princeton Ocean Model (POM) that has been adapted and validated for use in the Great Lakes as described in (Schwab and Bedford 1994; O'Connor and Schwab, 1994; Schwab and Morton, 1984) and is currently used for lake water level forecasting by NOAA. The basic limitations of the hydrodynamic model are given in Schwab (1978 and 1987). Briefly, the model is linear (water level displacements are assumed to be small compared with water depth), bottom friction is proportional to the square of the vertically averaged velocity, and baroclinic effects are ignored.

In SSPP v1.4, the wind is assumed to be spatially uniform and constant in time. Time- or space-variable winds can have an effect on storm surge response (see Schwab 1978). SSPP v1.4 uses a constant drag coefficient of 0.0032 to convert wind speed to wind stress at the water surface. This value is based on the work of Platzman (1963) and Schwab (1978) for storm surges on Lake Erie and Simons (1975) on Lake Ontario and may be somewhat high for the smaller fetches obtained in Lake St. Clair and Green Bay. Drag coefficients vary, increasing in unstable marine boundary layers and decreasing in stable conditions. These limitations have been considered when using the results of SSPP v1.4 for this study.

A.2.5 References

NOTE: Refer to the Project Manager's approval (on the signature page of this report) verifying that the Constellation Nuclear Energy Group (CENG) references are valid sources of design input created in accordance with the CENG's QA program.

GLERL Web Address: http://www.glerl.noaa.gov/ftp/publications/tech_reports/glerl-065/.

(Schwab et al. 1987). "Great Lakes Storm Surge Planning Program (SSPP), NOAA Tech. Memo. GLERL-65, 12 pp, National Oceanic and Atmospheric Administration, Schwab, D.J., and E. Lynn. 1987.

Platzman, G.W. 1963. The Dynamical Prediction of Wind Tides on Lake Erie. Meteorological Monographs, Vol. 4 No. 26, American Meteorological Society, Boston, 44 pp.

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ATTACHMENT (2)

REGULATORY COMMITMENTS CONTAINED IN THIS CORRESPONDENCE

ATTACHMENT (2)

REGULATORY COMMITMENTS CONTAINED IN THIS CORRESPONDENCE

The following table identifies actions committed to in this document. Any other statements in this submittal are provided for information purposes and are not considered to be regulatory commitments.

REGULATORY COMMITMENT	DUE DATE
Perform an integrated assessment for external flooding for the Nine Mile Point Site.	March 12, 2015
Implement interim actions to address higher flooding hazards, relative to the current design basis, as described in this submittal.	August 1, 2013