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.

The bounding flooding mechanism at Ginna is the combination of the PMF on the Deer Creek with the 25-year surge with wind-wave activity on Lake Ontario and the maximum controlled water level on the Lake.

3.1.1 Local Intense Precipitation

The CLB LIP event results in a flood elevation of 254.5 ft in the vicinity of the Screen House, but was not considered to impact safety related SSC which was located at elevation 254.8 ft (Ginna, 2011, Section 2.4.2.2). This event is based on a total rainfall depth of 6.11 inches in one hour and 19.17 inches over 24 hours (Ginna, 2012).

Based on the LIP re-evaluation calculation for the Ginna site, the following conclusions can be reached:

- The maximum water surface elevation due to the LIP at Ginna results from a total rainfall depth of 16 inches within one hour and 22.4 inches within 6 hours. In the immediate vicinity of the main buildings at Ginna, predicted maximum water surface elevations resulting from the LIP range from 255.8 ft at the Screen House to approximately 270.9 ft at the Auxiliary and All-Volatile-Treatment Buildings.
- At Ginna, the water elevation resulting from the LIP would result in flooding of the Screen House (255.8 ft flood level compared to a safe design elevation of 254.8 ft) and may impact safety-related SSC housed within.

See Section 2.1 for detailed information.

3.1.2 Flooding on Rivers and Streams

The CLB PMF event for Deer Creek results in flood elevations of 273.8 ft on the south wall of the Auxiliary Building, 272 ft east of the Auxiliary Building and the Turbine Building, and 256.6 ft in the north yard area near the Screen House. These flood elevations are based on a Deer Creek peak discharge of 26,000 cfs. (Ginna, 2012)

Based on the PMF re-evaluation calculation for the total contributory drainage area for the Deer Creek and the Mill Creek, the following conclusions can be reached:

- The probable maximum flood depth on streams results from the All-Season 72-hour PMP depth of 30.5 inches.
- The re-evaluated, peak PMF elevations at the SSC at Ginna generally range from 258.1 ft to 273.5, corresponding to the PMF peak flow of 28,500 cfs.
- Based on the re-evaluated peak PMF elevations, safety-related SSC would be impacted at the Screen House. Flooding may also occur in the Turbine Building and the All-Volatile Treatment Building, but no safety-related SSC would be affected in these buildings.
- The CLB did not previously include considerations of dynamic flood properties such as velocity or flow vectors.

See Section 2.2 for detailed information.

3.1.3 Dam Breaches and Failures

Flooding hazard from dam failure scenarios was not evaluated as part of the CLB.

Based on dam failure evaluations performed for NMP3NPP and the re-evaluated hazard for Deer Creek and Mill Creek, potential dam breaches and failures of the sources upstream of the Ginna site would not affect the safety-related SSC because:

- Regionally, the effects of lower than normal Lake Ontario water levels due to failure of the dams at the outlet of Lake Ontario in the St. Lawrence River have been considered in plant design. A margin of 16.2 ft. exists above the top of the Ginna intake to account for the lowest water level projected.
- Locally, the effects from the simultaneous failure of the three dams in the Ginna watershed would produce a total peak flow much less than the flow needed to fill the capacity of the Deer Creek channel, resulting in the channel water level remaining below the 270 ft. flood elevation of the Ginna site.
- On site, there are no above grade basins that would contribute to the potential flooding at the site.

See Section 2.3 for detailed information.

3.1.4 Storm Surge

The CLB Storm Surge event results in a surge level elevation of 253.28 ft resulting from a Tropical Storm and associated phenomena (Ginna, 2012).

Based on the PMSS re-evaluation calculation for Ginna, 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 3.2 feet.
- The predicted PMSS elevation is 251.1 feet (NGVD29), or 250.5 feet (IGLD85).
- Due to the armor revetment shoreline protection with a top elevation of 261 ft, this mechanism is not anticipated to impact safety-related SSC at Ginna. The opening in the revetment for the discharge canal was not impacted due to PMSS.

See Section 2.4 for detailed information.

3.1.5 Seiche

Flood hazards due to seiches were not evaluated as part of the CLB.

Based on the seiche re-evaluation for Ginna, the following conclusions are reached for Lake Ontario:

- The natural period of the primary and secondary seiche modes in Lake Ontario are approximately 5.1 and 3.2 hours, respectively.
- 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 axis of the lake (west to east) 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 PMS hazard for Ginna is bound by the PMSS elevation. Historical lake data indicates that amplification of storm surges does not occur.

The following conclusions are reached for Ginna Discharge Canal:

- The discharge canal is small, shallow and generally protected from meteorological forcing.
- The geometry of the discharge canal provides protection from wave induced seiche propagation.
- Significant earthquake-generated seiches in the discharge canal are not considered to be a hazard due to the relatively low anticipated ground motion at the site from the maximum expected earthquake.

See Section 2.5 for detailed information.

3.1.6 Tsunami

Flood hazards due to tsunami events were not evaluated as part of the CLB.

As an inland site, the Ginna site is not subject to oceanic tsunamis; however, tsunami-like waves (identified as 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 90 miles west of the Ginna site.
- Notwithstanding the occurrence of tsunami-like waves, the potential effects on the Ginna site (wave runup and draw down) are negligible because there is sufficient available physical margin to protect safety-related SSC. 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

It is determined in the CLB that ice-induced flooding presents a negligible hazard at the site due to topographic relief between the Deer Creek channel and safety-related SSC (Ginna, 2011).

Based on the historical records and information above, ice induced flooding at the Ginna site, if it occurred, would not affect the safety-related SSC because:

- 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;
- The nearest historical ice jams data on record occurred on separate water bodies greater than 10 mi from the site;

- There are no perennial streams close to or on the site that would contribute to the potential of ice induced flooding; and
- Site topography is such as to prevent ice induce flooding from a nearby intermittent stream due to large areas both south and east of the plant that would direct over flow toward the lake before the water level would rise to plant grade level.

See Section 2.7 for detailed information.

3.1.8 Channel Migration or Diversion

Flooding hazards due to channel migration or diversion were not evaluated as part of the CLB.

Based on the evaluation of historical and geomorphological information, channel migration or diversion is a negligible hazard to safety-related SSC.

See Section 2.8 for detailed information.

3.1.9 Combined Effect Flooding

The CLB evaluated the combined effect of a probable maximum storm surge with coincident wind-wave activity. This results in a wave runup elevation of 260.94 ft, which is below the revetment top elevation of 261 ft (Ginna, 2012).

Based on the Combined Effect evaluation for Ginna, the following conclusions are reached:

- The bounding combined-effect flooding mechanism at Ginna is the combination of the PMF on the Deer Creek with the 25-year surge with wind-wave activity on Lake Ontario and the maximum controlled water level on the Lake. Under this alternative, waves overtop the stone revetment and discharge canal, increasing the PMF water surface elevations at the northern end of the site by 0.1 ft.
- The Probable Maximum Water Elevation at Ginna including wave effects is calculated to be 272.4 ft at the Reactor Containment Building, 272.6 ft at the Auxiliary Building, 258.2 ft at the Turbine Building, 272.4 ft at the Control Building, 271.3 ft at the All-Volatile Building, 272.8 ft at the Standby Auxiliary Feedwater Pump Building, 273.5 ft at the proposed Standby Auxiliary Feedwater Pump Building Annex, 258.2 ft at the Screen House, and 258.4 ft at the Diesel Generator Building.
- Impacts to safety-related SSC due to these flood elevations are discussed in Section 3.3.

See Section 2.9 for detailed information.

3.2 Summary of Walkdown Findings

The walkdown report (Ginna, 2012) found no deficiencies in credited flood protection features. A deficiency was defines as when a flood protection feature is unable to perform its intended flood protection function when subject to a design basis flooding hazard.

3.3 Impacts of Flood Elevations

A comparison of the re-evaluated flood hazards to the CLB flood elevations for each flooding mechanism indicates the following:

- Flood elevations have increased for LIP, PMF, and Combined Effect flooding.
- The flood scenarios indicated above result in potential impact to safety-related SSC located within the Screen House.

- The bounding re-evaluated flood mechanism at Ginna is the combination of the PMF on the Deer Creek with the 25-year surge with wind-wave activity on Lake Ontario and the maximum controlled water level on the Lake as detailed in Section 2.9.

Flood elevations due to the bounding flood scenario at specific buildings containing safety-related SSC compared to design levels and protection elevations are shown in Table 3.3-1.

Based on the modeled water surface elevation, the bounding flood scenario will result in flooding of safety-related SSC in one location, the Screen House building, rendering equipment housed within inoperable. Equipment in the Screen House includes safety-related service water pumps. Flooding of other structures including the Turbine Building and the All-Volatile Treatment Building is anticipated, but not considered a threat to safety-related SSC. Mitigation measures for maintaining ultimate heatsink availability during this flood scenario are discussed in Section 4.

3.4 References

Ginna, 2011. R. E. Ginna Nuclear Power Plant Updated Final Safety Analysis Report (UFSAR) Revision 23, December 6, 2011, see AREVA Document No. 38-9191389-000.

Ginna, 2012. Response to 10 CFR 50.54(f) Request for Information Recommendation 2.3 Flooding, Constellation Nuclear Energy Group, November 27, 2012, NRC Accession No. ML12335A029.

Table 3.4-1: Combined-Effect Flood Elevations Summary

Structure†	Design Basis Flood Levels (ft)*	PMF Peak Elevation (ft)	Protection Elevation (ft)*	Protection Margin (ft)	Maximum Flow Depth (ft)	Maximum Flow Velocity (fps)
Auxiliary Building	272.0 to 273.8	272.6	273.0 to 273.8	+0.4 to 1.2	2.0	2.8
Standby Auxiliary Feedwater Pump Building	273.0	272.8	273.0	+0.2	2.7	4.0
Proposed Standby Auxiliary Feedwater Pump Building Annex	N/A	273.5	273.8	+0.3	3.6	2.8
Screen House	256.6	258.2	254.8	-3.6	4.5	3.3
Diesel Generator Building	256.6	258.4	N/A**	N/A**	4.7	4.4

†Only locations with potential impacts to safety-related SSC are included in this summary. Other structures, such as the reactor containment building, are not included either because they do not contain safety-related SSC or have no direct pathways of external flood water ingress.

*Source: Ginna, 2011 and Ginna, 2012

** The Diesel Generator Building access ways are sealed against water leakage (Ginna, 2012). No flooding of the Diesel Generator Building is anticipated.

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 used for calculation of the LIP-induced PMF, PMF in streams and rivers near Ginna, and combined effect flood scenarios at Ginna. 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 flood levels at various Flood Hazard Re-evaluation Project sites.

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

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

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

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

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

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

A.1.2 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):

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

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

where

h = flow depth;

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

x = one of the eight flow directions;

i = rainfall intensity;

S_f = friction slope based on Manning's equation;

S_0 = bed slope

g = acceleration of gravity

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

A.1.4 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
	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

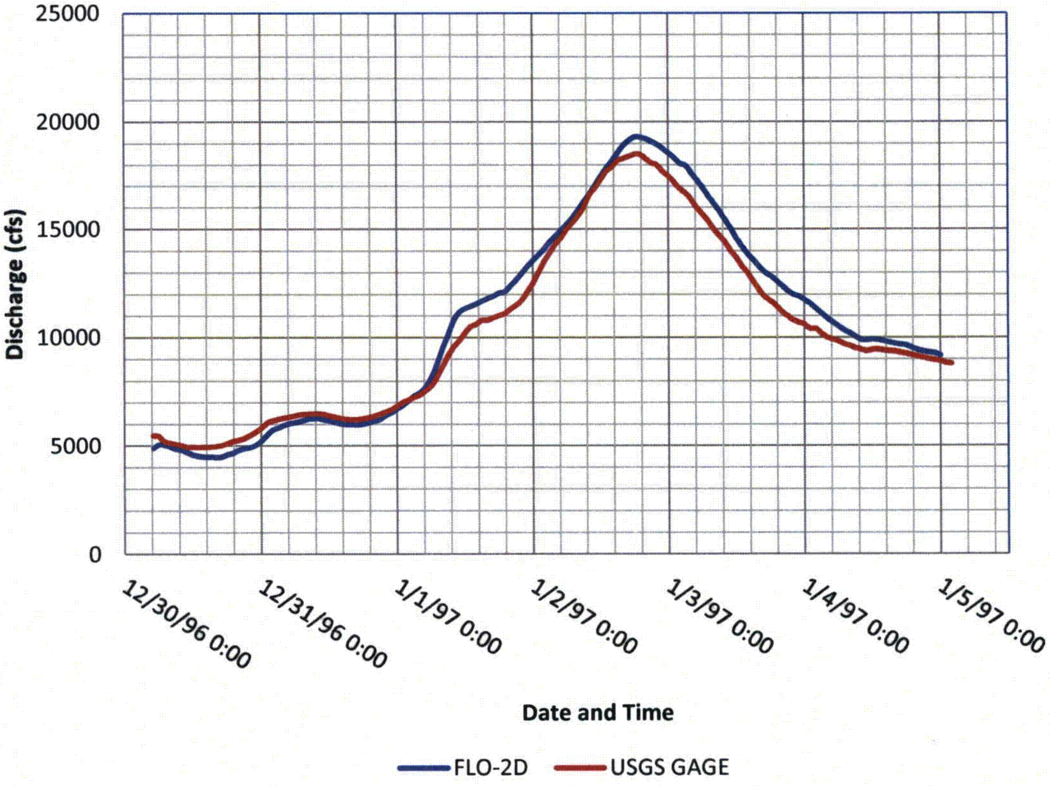

Figure A-1: Observed vs. Predicted Discharge for the 1997 Flood at Truckee River New Vista Gage

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 Ginna, located along the southern 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:

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

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

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