James A. Spina
Vice President, Corporate Site Operations
100 Constellation Way
Suite 200C
Baltimore, Maryland 21202
410-470-5203

CENG
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Constellation Energy
eDF

March 12, 2013

U.S. Nuclear Regulatory Commission
11555 Rockville Pike
Rockville, MD 20852

ATTENTION: Document Control Desk

SUBJECT: **Calvert Cliffs Nuclear Power Plant, Units 1 and 2**
Renewed Facility Operating License Nos. DPR-53 and DPR-69
Docket Nos. 50-317 and 50-318

Flood Hazard Reevaluation Report

REFERENCES:

(a) Letter from E. J. Leeds (NRC) and M. R. Johnson (NRC) to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, dated March 12, 2012, 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

(b) Letter from E. J. Leeds (NRC) to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, dated May 11, 2012, Prioritization of Response Due Dates for Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Flooding Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident

On March 12, 2012, the U.S. Nuclear Regulatory Commission (NRC) issued Reference (a). Enclosure 2 of Reference (a) contains specific requested actions, requested information, and required responses associated with Recommendation 2.1 for flooding reevaluations. The response date for the Calvert Cliffs Nuclear Power Plant Units 1 and 2 (Calvert Cliffs) flood hazard reevaluation was identified in Reference (b) as March 12, 2013, which corresponds to one year from the date of Reference (a).

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NRR

The purpose of this letter is to provide the Calvert Cliffs flood hazard reevaluation using updated flooding hazard information and present-day regulatory guidance and methodologies. The results are compared against the Calvert Cliffs current design basis for protection and mitigation from external flood events.

The report provided in Attachment 1 describes the approach, methods, and results from the reevaluation of flood hazards at Calvert Cliffs. The eight (8) flood-causing mechanisms, and a combined effect flood, identified in Attachment 1 to Enclosure 2 of Reference (a), are described in the report along with the potential effects on Calvert Cliffs.

Two reevaluated flood mechanisms, local Probable Maximum Precipitation (PMP) and Probable Maximum Storm Surge (PMSS) for Calvert Cliffs Units 1 and 2 exceeded the current design basis flood.

For a local PMP event, the immediate actions that will be put in place are the use of sand bags, or the use of other commercially available flood barriers like inflatable barriers, self inflating flood bags etc., at the Auxiliary Building access/entrance points as well as procedure changes to ensure the access pathway doors are closed for all doors susceptible to the reevaluated local PMP level. Based on a short flooding duration of 90 minutes, and the short duration of the maximum flood height of two (2) ft, it is reasonable to assume that these actions will be sufficient to preclude any effect on safety-related systems, structures, or components (SSC).

The preliminary estimate for the required amount of sandbags is 512 bags, to construct a three (3) foot high barrier at the six (6) personnel access doors and the three (3) roll up doors on the west side of the Auxiliary Building, which is the area of concern for a PMP event. If conventional sand bags are used, it is estimated it would take two (2) men eight (8) hours to complete the deployment, assuming filled bags are prestaged. As an alternative, Floodstop Flood Barriers can be deployed by a single individual in 30 minutes per door. These barriers are lightweight and sized based on the access needing protection. The time estimate for full deployment is approximately five (5) hours. As stated below, it is anticipated that support personnel will be available based on a 24 hour warning time associated with a PMP event. These time estimates will be validated through demonstration prior to June 1, 2013.

For the PMSS event, to mitigate the reevaluated storm surge elevation and minimize the water ingress into the intake structure, Calvert Cliffs will provide panel covers, which will be placed over the intake structure ventilation louvers. These covers would only be put in place prior to the arrival of the hurricane at the Calvert Cliffs site. The lead time available for the installation of these covers will coincide with a unit shutdown to comply with Plant Technical Procedure ERPIP-3.0 Attachment 20, Severe Weather.

The intake structure has 18 ventilation louvers that will require protection during a PMSS event. A design under consideration involves the use of a watertight membrane held in place by existing hooks and straps to provide a seal to prevent water ingress. Deployment of these seals can begin well in advance of the actual storm surge impacting the site. The current estimate is two (2) persons for eight (8) hours to complete the installation of all eighteen barriers. Once the final design is complete, this estimate will be validated by demonstrating deployment of at least one louver cover prior to June 1, 2013.

The local PMP and the PMSS events do not cause an immediate flooding concern at Calvert Cliffs. The time between the prediction of a severe precipitation event including local PMP and the potential flooding event will be greater than 24 hours, giving the plant time to initiate potential flood mitigation measures. The notification time for a storm surge including the PMSS resulting from a severe hurricane including the Probable Maximum Hurricane (PMH) is greater than 24 hours and allows for interim actions to mitigate these impacts.

The reevaluated PMP and PMSS flood causing mechanisms for Calvert Cliffs are not bounded by the current design basis for external flood at the site. Therefore, Constellation Energy Nuclear Group will perform an integrated assessment for external flooding in accordance with Reference (a).

Interim actions to address any higher flooding hazards relative to the design basis are as follows:

1. Revise station procedures to direct installing sandbags, or the use of other commercially available flood barriers like inflatable barriers, self inflating flood bags etc., at the access/entrance points as well as procedure changes to ensure the access pathway doors are closed for all doors susceptible to the reevaluated local PMP level. This action has been entered into the Corrective Action Program with a completion date of June 1, 2013. This due date coincides with the start of the hurricane season, which is the probable time period of a PMP event.

2. Revise station procedures to direct installing covers over the intake structure ventilation louvers to prevent water ingress, resulting from the new PMSS level, into the intake structure which houses safety-related SSCs. These covers would only be put in place prior to the arrival of the hurricane at the CCNPP site. This action has been entered into the Corrective Action Program with a completion date of June 1, 2013. This due date coincides with the start of the hurricane season, which is the probable time period of a PMSS event.

This letter contains regulatory commitments as listed in Attachment 2.

If there are any questions regarding this submittal, please contact Everett (Chip) Perkins everett.perkins@cengllc.com at 410-470-3928.

I declare under penalty of perjury that the foregoing is true and correct. Executed on March 12, 2013.

Sincerely,

James A. Spina

JAS/EMT/bjd

Attachments: (1) Calvert Cliffs Flood Hazard Reevaluation Report
(2) Regulatory Commitments Contained in this Correspondence

cc: B. K. Vaidya, NRC
W. M. Dean, NRC
Resident Inspector, Calvert Cliffs
S. Gray, DNR

ATTACHMENT (1)

CALVERT CLIFFS FLOOD HAZARD REEVALUATION REPORT

Constellation Energy Nuclear Group, LLC
March 12, 2013

Calvert Cliffs Nuclear Power Plant Units 1 and 2

Flooding Hazard Reevaluation Report

Revision 000
February 2013

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Acronyms

ANS	American Nuclear Society
ANSI	American National Standards Institute
CBBT	Chesapeake Bay Bridge Tunnel
CCNPP	Calvert Cliffs Nuclear Power Plant
CEM	Coastal Engineering Manual
CLB	Current Licensing Basis
COLA	Combined License Application
EDG	Emergency Diesel Generator
FSAR	Final Safety Analysis Report
HEC	Hydrologic Engineering Center
HEC-HMS	Hydrologic Engineering Center – Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center – River Analysis System
HHA	Hierarchical Hazard Assessment
HMR	Hydro-Meteorological Report
MDNR	Maryland Department of Natural Resources
MLW	Mean Low Water (tide)
MSL	Mean Sea Level
NAD27	North American Datum 1927
NGVD 29	National Geodetic Vertical Datum of 1927
NOAA	National Oceanographic and Atmospheric Administration
NRC	Nuclear Regulatory Commission
NRCS	Natural Resources Conservation Service
NTTF	Near-Term Task Force
NWS	National Weather Service
PMF	Probable Maximum Flood
PMH	Probable Maximum Hurricane
PMP	Probable Maximum Precipitation
PMSS	Probable Maximum Storm Surge
PMT	Probable Maximum Tsunami
SBO	Station Black-Out
SSC	Structures, Systems, and Components
UFSAR	Updated Final Safety Analysis Report
USACE	U.S. Army Corps of Engineers
USGS	United States Geological Survey
UTM	Universal Transverse Mercator

Units of Measure

ac-ft	Acre-feet
ac	Acres
cfs	Cubic feet per second
cu ft	Cubic feet
cu mi	Cubic mile
ft	Feet
ft/s	Feet per second
gpm	Gallons per minute
hr	Hours
in.	Inches
m	Meters
mb	Millibars
mi	Miles
mi/hr	Miles per hour
min	Minutes
°F	Degrees Fahrenheit
s	Seconds
sq mi	Square miles

BACKGROUND

Following the 2011 Great Tohoku Earthquake and tsunami and the resulting events at the Fukushima Dai-ichi Nuclear Power Plant, the Nuclear Regulatory Commission (NRC) established the Near-Term Task Force (NTTF). The NTTF was tasked with conducting a systematic and methodical review of NRC regulations and processes and evaluating the need for additional improvements to these programs. As a result of the review, the NTTF developed a comprehensive set of recommendations, which is documented in the enclosure to SECY-11-0093 (Reference 1).

These recommendations were enhanced by NRC staff following interactions with stakeholders and are documented in SECY-11-0124, Recommended Actions to Be Taken without Delay from the NTTF Report, dated September 9, 2011 (Reference 2), and SECY-11-0137, Prioritization of Recommended Actions to be Taken in Response to Fukushima Lessons Learned, dated October 3, 2011 (Reference 3).

As part of the staff requirements memorandum (SRM) for SECY-11-0124, dated October 18, 2011, the NRC approved the staff's proposed actions, including the development of three information requests under Title 10 of the Code of Federal Regulations Part 50.54(f). The information collected would be used to support the NRC staff's evaluation of whether further regulatory action should be pursued in the areas of seismic and flooding design and emergency preparedness.

In addition to NRC direction, the Consolidated Appropriations Act, Public Law 112-074, was signed into effect on December 23, 2011. Section 402 of the law requires a reevaluation of licensees' design basis for external hazards.

In response to the aforementioned commission and congressional directions, the NRC issued a request for information letter to all power reactor licensees and holders of construction permits under 10 CFR 50.54(f) on March 12, 2012 (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 NTTF Review of Insights from the Fukushima Dai-ichi Accident [Reference 4]). The March 12, 2012, 50.54(f) letter describes that consistent with NTTF Recommendation 2.1, the NRC would implement hazard evaluation in two phases.

- In Phase 1, through the 50.54(f) letter, the NRC requests that licensees reevaluate the seismic and flooding hazards at their nuclear power plant sites using updated seismic and flooding hazard information and present-day regulatory guidance and methodologies and, if necessary, perform a risk evaluation.

- In Phase 2, based on the Phase 1 results, the NRC will determine whether additional regulatory actions are necessary (e.g., update the design basis and structures, systems and components important to safety) to provide additional protection against the updated hazards.

Enclosure 2 of the March 12, 2012, 50.54(f) letter identifies the requirements for the external flooding hazard reevaluations associated with NTTF Recommendation 2.1 and requests information to:

- Reevaluate seismic and flooding hazards at operating reactor sites with respect to NTTF Recommendation 2.1, as amended by SRM on SECY-11-0124 and SECY-11-0137, and the Consolidated Appropriations Act, for 2012, Section 402.

- Facilitate NRC's determination whether there is a need to update the design basis and structures, systems, and components (SSCs) that are important to safety to protect the updated hazards at operating reactor sites.

- Collect information to address Generic Issue (GI) 204 regarding flooding of nuclear power plant sites following upstream dam failures.

This report was prepared in response to the March 12, 2012, 50.54(f) letter to provide requested information on the reevaluation of external flooding hazards at Calvert Cliffs Nuclear Power Plant (CCNPP) Units 1 & 2 using present-day methodologies, data, and regulatory and industry guidance.

Flooding hazards from external sources for the CCNPP site and vicinity have been evaluated recently in support of the Combined License Application (COLA) for a future unit (Unit 3) (Reference 5) to be located immediately to the south and southeast of Units 1 & 2 within the plant's property boundary. The approach and methods used for the CCNPP Units 1 & 2 external flooding reevaluation are the same as the Unit 3 COLA analyses, which are consistent with the standards and requirements of present-day regulatory and industry guides, in particular, NUREG/CR-7046 (Reference 6), NUREG/CR-6966 (Reference 7), NUREG 0800 (Reference 8), ANSI/ANS-2.8-1992 (Reference 9), and NEI 12-08 (Reference 10). In applicable cases, the analysis performed for the CCNPP Unit 3 COLA, augmented by recent site-specific information, is adopted for this reevaluation. The results of the flooding hazard reevaluation are compared to the current design basis for the plant, which is documented in the Updated Final Safety Analysis Report (UFSAR) (Reference 11) and CCNPP Units 1 & 2 Flooding Walkdown Report (Reference 12) prepared in response to NTTF Recommendation 2.3.

Chapter 1 of this report provides site-specific information related to flood hazard. Chapter 2 provides reevaluation of flood hazards for each flood-causing mechanism. Chapter 3 compares current and reevaluated flood-causing mechanisms. Interim flood protection measures and actions taken for the higher flooding hazards are summarized in Chapter 4, and additional actions beyond interim flood protection measures are described in Chapter 5.

Background References

1. U.S. Nuclear Regulatory Commission, *Recommendations for Enhancing Reactor Safety in the 21st Century, The Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident*, SECY-11-0093, ADAMS Accession No. ML111861807, July 12, 2011.

2. U.S. Nuclear Regulatory Commission, *Recommended Actions to be Taken Without Delay from the Near Term Task Force Report*, SECY-11-0124, ADAMS Accession No. ML11245A158, September 9, 2011.

3. U.S. Nuclear Regulatory Commission, *Prioritization of Recommended Actions to be Taken in Response to Fukushima Lessons Learned*, SECY-11-0137, ADAMS Accession No. ML11272A111, October 2011.

4. 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*, ADAMS Accession No. ML12053A340, March 12, 2012.

5. Unistar Nuclear Services, LLC. *Calvert Cliffs Nuclear Power Plant Unit 3, Combined License Application*, Rev. 8, March 2012.

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

7. U.S. Nuclear Regulatory Commission, *Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America*, NUREG/CR-6966, March 2009.

8. U.S. Nuclear Regulatory Commission, *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition — Site Characteristics and Site Parameters (NUREG-0800, Chapter 2), Sections 2.4.1 through 2.4.10*, March 2007.

9. American National Standards Institute/American Nuclear Society, *Determining Design Basis Flooding at Power Reactor Sites*, ANSI/ANS-2.8-1992, Nuclear Standard 2.8, 1992.

10. Nuclear Energy Institute, *Overview of External Flooding Reevaluations*, NEI 12-08, Rev. 0, August 2012.

11. Calvert Cliffs Nuclear Power Plant Inc. *Updated Final Safety Analysis Report*, Rev. 43, 2012.

12. Constellation Energy Nuclear Group, LLC, *Flooding Walkdown Report*, Report No: SL-011462, Revision 0, November, 2012.

1 SITE INFORMATION RELATED TO FLOODING HAZARDS

This section presents site information and data as requested in Enclosure 2 of the Nuclear Regulatory Commission (NRC) request for information letter under 10 CFR 50.54(f) on March 12, 2012. Following Near-Term Task Force (NTTF) Flooding Recommendation 2.1, the NRC 50.54(f) letter requires that the licensee provide:

- Detailed site information (both designed and as-built), including present-day site layout; elevation of pertinent structures, systems, and components (SSCs) that are important to safety; and site topography, as well as pertinent spatial and temporal data sets.
- Current design basis flood elevations for all flood-causing mechanisms.
- Flood-related changes to the licensing basis and any flood protection changes (including mitigation) since license issuance.
- Changes to the watershed and local area since license issuance.
- Current licensing-basis flood protection and pertinent flood mitigation features at the site.
- Additional site details, as necessary, to assess the flood hazard (i.e., bathymetry, walkdown results)

The requested information and data and current design basis flood elevation and flood-related changes are described in Subsections 1.1 through 1.6.

Calvert Cliffs Nuclear Power Plant (CCNPP) Units 1 & 2 adopts the mean sea level (MSL) datum as the plant's reference vertical datum. The MSL datum is also referred to as the National Geodetic Vertical Datum of 1929 (NGVD 29). In this report, the NGVD 29 datum is referenced primarily; however, the two definitions could be used interchangeably. Tidal datums are referenced as they appear in the source. Directions are described relative to true north in this report, unless specified otherwise. English units are used consistently throughout this report except for instances where the base information is developed in SI units. In such a case, the corresponding English unit is provided in parenthesis.

1.1 Detailed Site Information

1.1.1 Location

The CCNPP site is located in Calvert County, Maryland, approximately 10.5 mi southeast of Prince Frederick, Maryland, and on the western shore of the Chesapeake Bay, approximately 110 mi north from the Chesapeake Bay entrance. The CCNPP site covers approximately 2,057 acres. The CCNPP Units 1 & 2 plant and ancillary facilities are located on approximately 932 acres. Figure 1.1-1 shows the CCNPP Units 1 & 2 plant layout, including the proposed CCNPP Unit 3. Figure 1.1-2 shows the CCNPP Units 1 & 2 site and vicinity areas.

1.1.2 Site and Facilities

The topography at the CCNPP site is gently rolling with steeper slopes along stream banks. Local relief ranges from the sea level up to an approximate elevation of 130 ft, with an average relief of approximately 100 ft. Along the northeastern (relative to true north) perimeter of the CCNPP site, the Chesapeake Bay shoreline consists mostly of steep cliffs with a narrow beach area. The CCNPP site is well drained by short, ephemeral streams. A drainage divide, which is generally parallel to the shoreline, extends across the CCNPP site. The area to the northeast of the divide, which lies within the Maryland Western Shore Watershed, comprises about 20 percent of the CCNPP site property and drains into the Chesapeake Bay. The CCNPP Units 1 & 2 plant is located east of the divide. The southwestern area within the Patuxent River Watershed is drained by tributaries of Johns Creek, which flow into St. Leonard Creek, located west of Maryland State Highway 2/4 and subsequently flow into the Patuxent River. The Patuxent River empties into the Chesapeake Bay approximately 10 mi (16.1 km) to the southeast from the mouth of St. Leonard Creek. All streams that drain the CCNPP site that are located east of Maryland State Highway 2/4 are non-tidal. Figure 1.1-2 shows the general topography of the site, the local drainage routes near the CCNPP site, and the drainage divide. The figure also shows the location of the proposed CCNPP Unit 3 plant (Reference 1.1-1).

Southeast of CCNPP Units 1 & 2 is an abandoned recreational area known as Camp Conoy that was used by CCNPP employees and their families. The Calvert Cliffs State Park is located farther to the southeast, outside of the CCNPP property boundary. The Flag Ponds Nature Park is located northwest of the CCNPP site.

In the western shore, Maryland's Critical Area Commission law requires a 1,000 ft (305 m) critical area along the Chesapeake Bay shoreline (Reference 1.1-2). The CCNPP Units 1 & 2 power block is located within the critical area.

The CCNPP Units 1 & 2 safety-related and important-to-safety SSCs were constructed across three nearly level terraces as described in the Flooding Walkdown Report (Reference 1.1-18): (1) the safety-related intake structure is located at a deck elevation of 10 ft NGVD 29, (2) safety-related and important-to-safety SSCs in the main plant area are located at a grade elevation of about 45 ft NGVD 29, and (3) plant substation (switchyard) and administrative buildings are located at a grade elevation of about 70 ft NGVD 29. All open slopes between the intake level and the main plant level are riprap protected, while the open slope between the switchyard level and the main plant level are protected with ripraps and gabion mattresses.

Safety-related SSCs at the CCNPP Units 1 & 2 main plant area, as indicated in the CCNPP Units 1 & 2 License Renewal Application (Reference 1.1-3), include the Containment Buildings, Auxiliary Building, and Emergency Diesel Generator (EDG) Building. The Turbine Building is located east of the Auxiliary Building, and although it is classified as Seismic Category II, it houses the safety-related Auxiliary Feedwater System pumps located at a sub-grade elevation of 12 ft NGVD 29. The Turbine Building also provides access to CCNPP Units 1 & 2 Control Rooms. The CCNPP Units 1 & 2 intake structure is classified as safety-related as it houses the saltwater pumps that are essential for safe shutdown of these units. An augmented Quality Building houses the Station Blackout diesel generator. The Turbine Building for the CCNPP Units 1 & 2 is oriented parallel and adjacent to the shoreline of the Chesapeake Bay, with the twin Containment Structures and the Auxiliary Building located on the west, or landward, side of the Turbine Building. The service building and the intake and discharge structures are on the east, or bay side, of the Turbine Building. Critical elevations for the safety-related and important-to-safety SSCs are summarized in Table 1.1-1; these SSCs are shown in Figure 1.1-3.

The safety-related facilities for CCNPP Units 1 & 2, with the exception of the saltwater cooling pumps, are located in the main plant area where the grade elevation is about 45 ft NGVD 29 (Reference 1.1-4). This grade elevation is substantially higher than the flood elevations from a design storm surge or tsunami event in the Chesapeake Bay as reevaluated in Sections 2.4 and 2.6, respectively. There is no major surface water course near the plant, so probable maximum flooding pertaining to rivers and streams would not be a risk concern to the safety functioning of the plant as described in Section 2.2. As described in Section 2.3, dam break flooding from the Patuxent River watershed will not affect the site. Therefore, high water level from local intense precipitation would be the controlling flood elevation at the safety-related and important-to-safety SSCs of the CCNPP Units 1 & 2, except for the intake structure. Local intense precipitation-induced flooding is reevaluated in Section 2.1.

The safety-related CCNPP Units 1 & 2 intake structure located on the Chesapeake Bay shoreline, as shown in Figure 1.1-3, could be affected by storm surges and tsunamis. The intake structure provides water to both circulating water and saltwater cooling pumps. Water from the bay is drawn to the intake structure through a 40-ft to 51-ft deep and 560-ft wide dredged channel approximately 4,500 ft offshore from the intake structure. The longitudinal slope of the intake channel near the intake structure is maintained at 10 horizontal to 1 vertical with an invert elevation of -26 ft NGVD 29 at the intake structure. A baffle wall, which extends to a depth of -28 ft NGVD 29 over the intake channel to form a forebay for the intake structure, limits the withdrawal to mostly bottom water. The baffle wall is constructed of fixed and removable panels, with the top-of-wall elevation at +5.0 ft NGVD 29. The lowest elevation in the intake channel at the baffle location is -51 ft NGVD 29.

CCNPP Units 1 & 2 uses once-through cooling for the plant non-safety-related circulating water system cooling that cools the exhaust steam of the main turbine and steam generator feed pump turbines. Six circulating water pumps per unit, having a combined volumetric capacity of 1,200,000 gallons per minute, take suction from and discharge to the Chesapeake Bay through a three-shell condenser.

The saltwater cooling system provides bay water to the safety-related component cooling heat exchangers, the service water heat exchangers, and the emergency core cooling system pump room air coolers. There are three vertical centrifugal pumps per unit, only two of which are to be online during normal plant operation. These pumps take suction from the circulating water intake structure and discharge through the heat exchangers to the Chesapeake Bay.

The intake structure includes a 50-ft-wide, open deck and has openings for the trash rakes and racks, stop logs, and traveling screens. Behind this open deck is an enclosure housing where the circulating water pumps and saltwater cooling pumps are located. The roof of the pump building is at elevation 28.5 ft NGVD 29 and has watertight hatches to provide access to the pumps for maintenance. An intake structure air supply unit is mounted on each saltwater pump hatch, and an air exhaust vent is mounted on each circulating water pump hatch. To minimize entry of moisture into the pump room, each air supply unit and air exhaust vent housing is provided with louvers for high moisture separation efficiency. The personnel door located at the north end of the intake structure is watertight.

The maximum storm surge elevation in the Chesapeake Bay from a probable maximum hurricane (PMH) event constitutes the design basis flood elevation for the CCNPP Units 1 & 2 intake structure. Details of the present-day storm surge flooding evaluation for the intake structure are described in Section 2.4.

1.1.3 Hydrological Characteristics

The CCNPP Units 1 & 2 site is located on the Calvert Peninsula within the Chesapeake Bay watershed. The Chesapeake Bay constitutes the main water body influencing the flooding of CCNPP Units 1 & 2. The Chesapeake Bay, having a watershed area in excess of 64,000 sq mi, is the largest estuary in the United States.

The Calvert Peninsula is formed by the Chesapeake Bay to the east and the Patuxent River to the west. It is approximately 5 mi from the CCNPP site. The Patuxent River flows near the CCNPP site from the northwest to the southeast. Drainage in the vicinity of the CCNPP site includes several small streams and creeks, which fall within two sub-watersheds of the Chesapeake Bay, with the drainage divide running nearly parallel to the shoreline. These sub-watersheds include the Patuxent River Watershed and the Chesapeake Bay Sub-Watershed (Maryland Western Shore Watershed) (Reference 1.1-5). Figures 1.1-4 (Reference 1.1-6) and 1.1-6 show the Chesapeake Bay Watershed and Sub-Watersheds and the Lower Patuxent River Watershed along with the CCNPP site location.

1.1.4 Maryland Western Shore Watershed

The Maryland Western Shore Watershed is approximately 1,670 sq mi (Reference 1.1-5), most of which is located in the northern part of the watershed as shown in Figure 1.1-4. In the southern part, the watershed becomes a narrow strip along the Chesapeake Bay shoreline, referred to as the Lower Western Shore Basin, which drains water directly to the Chesapeake Bay from approximately 305 sq mi (Reference 1.1-7) of land. At the CCNPP site, all of the plant safety-related and important-to-safety SSCs are located in this watershed. Also, southeast of the CCNPP Units 1 & 2, this part of the watershed includes steep cliffs along the Chesapeake Bay shoreline. It is drained by two unnamed creeks, Branch 1 and Branch 2, located southeast of CCNPP Units 1 & 2 as shown in Figure 1.1-2. Drainage in these creeks moves away from the CCNPP Units 1 & 2 plant area and does not affect the safe functioning of the plant.

1.1.5 Patuxent River Watershed

Part of the CCNPP Units 1 & 2 site also falls in the Lower Patuxent River Watershed. The Patuxent River is the largest river completely contained in Maryland, draining an approximate area of 932 sq mi as shown in Figure 1.1-4. This area includes portions of St. Mary's, Calvert, Charles, Anne Arundel, Prince George's, Howard, and Montgomery Counties (Reference 1.1-7).

The Lower Patuxent River Sub-Watershed within Calvert County is approximately 174 sq mi. It covers over 50 percent of the land in the county (Reference 1.1-8). The main stem of the Patuxent River is influenced by tidal fluctuation in the Chesapeake Bay. The tidal influence is observed over nearly the entire length of the river in the lower watershed, with the head of tide located south of Bowie, Maryland.

The United States Geological Survey (USGS) maintains stream gauging stations on the Patuxent River at Bowie, Maryland (USGS Station No. 01594440), approximately 60 mi upstream from the river mouth (Reference 1.1-9). The drainage area at the gauging station is 348 sq mi, which is approximately 37 percent of the total drainage area of the Patuxent River. The station is located in the non-tidal reach of the river. Monthly streamflows and mean, maximum, and minimum daily streamflows at the Bowie gauge are presented in Table 1.1-2 through Table 1.1-5. Maximum, mean, and minimum monthly streamflow discharges are also presented in Figure 1.1-5.

The Lower Patuxent River Watershed in Calvert County, Maryland, is further divided into 13 sub-watersheds (Reference 1.1-8) as shown in Figure 1.1-6. Part of the CCNPP site is located within the St. Leonard Creek sub-watershed, which has an area of approximately 35.6 sq mi (Reference 1.1-8). Streams and water courses in the sub-watershed include Johns Creek and its tributaries that drain the CCNPP site.

The USGS operated a gauging station on St. Leonard Creek (USGS Station No. 01594800) near St. Leonard, Maryland, from 1957 to 1968 and from 2000 to 2003 (Reference 1.1-10). The gauging station had a drainage area of 6.73 sq mi comprising approximately 19 percent of the St. Leonard Creek sub-watershed area. Monthly streamflows and mean, maximum, and minimum daily streamflows from the gauge are presented in Table 1.1-6 through Table 1.1-9. Mean, maximum, and minimum monthly streamflow discharges are also presented in Figure 1.1-7.

1.1.6 The Chesapeake Bay Estuary

The Chesapeake Bay is one of the largest and most productive estuarine systems in the world. The Chesapeake Bay main stem, defined by tidal zones, is approximately 195 mi long from its entrance at the Atlantic Ocean near Norfolk, Virginia, to the mouth of the

Susquehanna River near Havre de Grace, Maryland. The Chesapeake Bay varies in width from about 3.5 mi near Aberdeen, Maryland, to 35 mi at the widest point near the mouth of the Potomac River, with an approximate width of 6 mi near the CCNPP site. The average depth of the bay, including tidal tributary channels, is about 21 ft. It has an open surface area of nearly 4,480 sq mi, and including its tidal estuaries, has approximately 11,684 mi of shoreline. The Chesapeake Bay is long enough to accommodate one complete tidal wave cycle at all times (Reference 1.1-11, Reference 1.1-12).

The Chesapeake Bay receives freshwater flows from an area in excess of 64,000 sq mi (Reference 1.1-12). Flow circulation in the Chesapeake Bay is mainly governed by astronomical tides entering the bay through the bay mouth near Norfolk, Virginia, gravitational flow due to freshwater inflow from the rivers, and wind-driven and atmospheric-pressure-driven circulation. The USGS provides estimates of monthly freshwater inflow to the Chesapeake Bay based on a methodology (Reference 1.1-13) that uses index stream gauging data from the Susquehanna, Potomac, and James Rivers (Reference 1.1-14). Estimated monthly freshwater inflow to the bay from 1951 to 2000 is provided in Table 1.1-10. The average annual freshwater inflow to the Chesapeake Bay for this period was approximately 77,500 cfs (Reference 1.1-14).

The mean tidal range in the bay varies from approximately 2.55 ft near the Atlantic Ocean entrance (Chesapeake Bay Bridge Tunnel [CBBT], Virginia), decreasing to approximately 1.04 ft near Cove Point, Maryland, near the CCNPP site, and increasing to nearly 1.9 ft near the northern head waters (Havre De Grace, Maryland) (Reference 1.1-15). The mean tidal range near the entrance of the Patuxent River (Solomons Island, Maryland) is about 1.17 ft, while the range at the upstream stations in Lower Marlboro, Maryland, is 1.79 ft (Reference 1.1-15). Tides in the Chesapeake Bay are mainly semidiurnal, with two nearly equal tide peaks and two troughs each over a day. Tidal currents in the bay follow a distribution similar to that of the mean tidal ranges. The spring tidal current, as estimated by the National Oceanic and Atmospheric Administration (NOAA) at the entrance of the Chesapeake Bay, is about 1.7 knots. At the entrance of Baltimore Harbor, the current magnitude reduces to approximately 1.1 knots but increases in the Chesapeake and Delaware Canal near Chesapeake City to about 2.5 knots (Reference 1.1-15).

The Chesapeake Bay is periodically affected by storm surges generated in the Atlantic Ocean. Between 1851 and 2005, 11 hurricanes affected the Chesapeake Bay region with intensities greater than Category I in the Saffir-Simpson Hurricane scale (Reference 1.1-16). Three typical storm tracks can be identified for the hurricanes affecting the Chesapeake Bay:

- Storms with landfall in Georgia or the South Carolina coast that progress over land west of and away from the Chesapeake Bay, generally producing high rainfall
- Lower outer bank hurricanes with landfall in southern North Carolina that progress along the Virginia eastern shoreline east of the Chesapeake Bay, typically producing drawdown in the Upper Bay

- Upper outer bank hurricanes with landfall in northern North Carolina that follow a path nearly parallel to and west of the Chesapeake Bay, producing high surges in the Upper Bay

Further details of PMHs and their impact on Chesapeake Bay hydrology are provided in Section 2.4.

Meteorologically induced seiches are observed in the bay, as described in Section 2.5, which may increase tidal magnitude. However, seiches are not likely to coincide with the PMH event and therefore would not be a design basis event.

Although the east coast of the United States is generally free from tsunamis generated in the Atlantic Ocean, historical records establish that tsunamis and tsunami-like events have occurred in this area. The impact of tsunamis on the CCNPP site is discussed in Section 2.6.

Ice sheets may form on the upper reach of the Chesapeake Bay, including the CCNPP site; however, historical ice formation in the Chesapeake Bay has not caused any instances of ice jams or ice induced flooding at the CCNPP site. Section 2.7 provides a detailed discussion on ice formation and its impact on the CCNPP site.

Observation of historical shoreline position data shows shoreline erosion near the CCNPP Units 1 & 2 site. Shoreline protection measures were constructed north and south of the CCNPP Units 1 & 2 intake structure to prevent any further shoreline retreat. Therefore, shoreline erosion is not expected to affect the safe functioning of the CCNPP Units 1 & 2 plant. Channel diversion potential is described in Section 2.8.

1.1.7 Dams and Reservoirs

There are no dams or reservoirs on St. Leonard Creek or its tributaries. There are two dams on the Patuxent River. These are Rocky Gorge Dam and Brighton Dam, located approximately 75 and 85 mi from the mouth of the Patuxent River, respectively. Details of the dams are provided in Table 1.1-11 (Reference 1.1-17). Potential failure of these dams would not cause any flooding concerns at the CCNPP site, which is discussed further in Section 2.3.

1.1.8 References

1.1-1 Unistar Nuclear Services, LLC. *Calvert Cliffs Nuclear Power Plant Unit 3, Combined License Application*, Revision 8, March 2012.

1.1-2 Critical Area Commission for the Chesapeake and Atlantic Coastal Bays, *Critical Primer*, Website: http://www.dnr.state.md.us/criticalarea/section2.html, Date accessed: December 6, 2006.

1.1-3 Baltimore Gas and Electric Company, *Calvert Cliffs Nuclear Power Plant Inc., License Renewal Application*, 1998.

1.1-4 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

1.1-5 The Chesapeake Bay Watershed, *Chesapeake Bay Program*, Website: http:// www.chesapeakebay.net/wspv31/(ojtvye45vgw30j55kcanqc45)/WspAboutPrint.a spx? basno=1, Date accessed: August 31, 2006.

1.1-6 A. Gellis, W. Banks, M. Langland, and S. Martucci, *Summary of Suspended-Sediment Data for Streams Draining the Chesapeake Bay Watershed*, Water Years 1952–2002 (Scientific Investigation Report 2004-5056), U.S. Geological Survey, 2005.

1.1-7 Maryland Department of Natural Resources, *Maryland Tributary Strategy Lower Western Shore Basin Summary Report for 1985-2004*, 2006.

1.1-8 Center for Watershed Protection, *Lower Patuxent River Watershed Restoration Action Strategy In Calvert County, MD*, 2004.

1.1-9 U.S. Geological Survey, *Patuxent River near Bowie, MD (USGS 01594440)*, National Water Information System: Web Interface, Website: http://waterdata.usgs.gov/md/nwis/inventory/?site_no=01594440&agency_cd=U SGS&, Date accessed: February 07, 2013.

1.1-10 U.S. Geological Survey, *St. Leonard Creek near St Leonard, MD (USGS 01594800)*, National Water Information System: Web Interface, Website: http://waterdata.usgs.gov/md/nwis/monthly?site_no=01594800&agency_cd=USG S&referred_module=sw&format=sites_selection_links, Date accessed: February 07, 2013.

1.1-11 E. Langland and T. Cronin, *A Summary Report of Sediment Processes in Chesapeake Bay and Watershed (Water Resources Investigation Report 03-4123)*, U.S. Geological Survey, 2003.

1.1-12 Chesapeake Bay Program, *Did You Know? Bay Factoids, About the Bay*, October 20, 2004, Website: http://www.chesapeakebay.net/info/factoids.cfm, Date accessed: November 30, 2006.

1.1-13 C. Bue, 1968, *Monthly Surface-Water Inflow to Chesapeake Bay (USGS Survey Open-File Report)*, U.S. Geological Survey, Website: http://md.water.usgs.gov/publications/ ofr-68-Bue10/ofr-68-bue10.html, Date accessed: September 22, 2006.

1.1-14 U.S. Geological Survey, *Estimated Streamflow Entering Chesapeake Bay, USGS Chesapeake Bay Activities*, Website: http://md.water.usgs.gov/monthly/bay.html, Date accessed: March 5, 2007.

1.1-15 National Oceanic and Atmospheric Administration, Tidal Range (information extracted from the National Oceanic and Atmospheric Administration for tidal range at various locations along the Chesapeake Bay), Website: http://tidesandcurrents.noaa.gov, Date accessed: March 6, 2007.

1.1-16 National Oceanic and Atmospheric Administration, *Chronological List of All Hurricanes which affected the Continental United States: 1851-2005*, Website:

http:// www.aoml.noaa.gov/hrd/hurdat/ushurrlist.htm, Date accessed: December 1, 2006.

1.1-17 U.S. Army Corps of Engineers, *National Inventory of Dams, Detailed Information for Brighton and Rocky Gorge Dam*, Website: http://crunch.tec.army.mil/nidpublic/webpages/ nid.cfm, Date accessed: November 21, 2006.

1.1-18 Constellation Energy Nuclear Group, LLC, *Flooding Walkdown Report*, Report No: SL-011462, Revision 0, November, 2012.

Table 1.1-1 Safety-related and Important-to-Safety SSCs and Associated Critical Elevations

Safety-Related and Important-to-Safety SSCs	Surrounding Grade Elevation[1] (ft NGVD29)	Entrance (Roof) Elevation [2] (ft NGVD 29)	UFSAR Section [3]
Containment Buildings	45.0	45.0[4]	1.2.2
Auxiliary Building	45.0	45.0	1.2.2
Emergency Diesel Generator Building	42.0 – 45.0	45.5	1.2.2
Intake Structure	10.0	10.0 (28.5)[5]	1.2.2 2.8.3
Turbine Building[6]	45.0	45.0	1.2.2
Station Blackout Diesel Generator Building[7]	42.0 – 45.0	45.5	1.2.2

Notes:

(1) Approximate grade elevation near the identified SSC

(2) Roof elevation is relevant for the intake structure as water levels exceeding the roof elevation may enter the safety-related intake structure through louvered ventilation hatches

(3) Section number in CCNPP Units 1 & 2 UFSAR (Reference 1.1-4) where a description of the structure is provided

(4) Personnel and equipment hatches for the Containment Buildings

(5) Personnel access door at 10.0 ft NGVD 29 elevation is watertight

(6) Turbine Building is a Seismic Category II structure

(7) Station Blackout Diesel Generator Building is an augmented quality structure

Table 1.1-2 Monthly Mean Streamflow for the Patuxent River at Bowie, MD, USGS Station No. 01594440, Patuxent River near Bowie, MD (1977 through 2012)

YEAR	Monthly mean Discharge in cfs (Calculation Period: 1977-07-01 -> 2012-09-30)											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1977	-	-	-	-	-	-	131.1	126.5	77.8	220.7	314.3	748.2
1978	1,316	358.6	854.2	372.7	884	233.8	298.5	293.2	130	109.6	201.8	347.9
1979	1,290	1,232	817.5	523.8	460.8	611.6	220	531.9	1,358	1,093	458.8	384.5
1980	496.6	262.8	693.9	806	670.3	308.5	210.3	157.3	106.7	201.1	181.9	135.7
1981	119.2	319.5	173.2	188.1	236.6	209.7	145.6	90.7	116.9	117.4	107.6	158.3
1982	211.9	507.8	328	344.8	206.2	439	124.5	111.8	147.2	130.5	177.5	204.8
1983	173.2	317.8	683	1,247	719.9	766.9	176.6	137.5	110.4	285.8	424.2	1,030
1984	407	658.3	843.1	843.2	657.5	262.1	371.6	343.3	182.8	155.7	225.5	306.1
1985	173.7	536.1	203.9	167.4	238.4	193.6	134.5	104.5	211.4	163	276.1	214.9
1986	218.4	379.5	294.3	328.9	153.8	116.4	102.3	121.5	65.2	80.4	251.6	489.1
1987	428.3	286.2	365.1	459.8	291.2	192.4	176.1	86.1	379	160	316.4	368.6
1988	453.2	566.2	364.2	351.9	730.1	190.7	189.8	130.4	119	114.1	278.6	180
1989	287.1	326.9	532.1	453	1,291	845.6	491.8	304.5	243.4	391.4	348.5	182
1990	462.1	424.5	374.8	581.8	578.4	324.7	209.9	306.3	125.6	332.6	305.6	459.4
1991	720.5	266.4	650.5	376.8	194.7	114.9	103	111.4	133.5	145.8	148.1	295.3
1992	217.8	251.6	399.5	260.9	221.2	234.6	248.4	153.7	188	167.7	350.5	537.5
1993	473.3	335.2	1,358	1,021	429.5	268.6	126.1	132.9	138.2	141.9	392.8	539.2
1994	657.6	930.1	1,318	648.5	347.4	182.9	239.7	319	202.3	153.4	193.9	237.6
1995	389.5	228.1	397.6	198.6	308.2	178.8	156.9	168.3	127.6	381.3	491	332.1
1996	1,035	549.5	566.6	598.3	575.9	654.4	579.2	474.8	701.6	614.1	747.2	1,357
1997	652.3	683.3	870.9	531.6	391.7	319	136.1	177.3	136.1	180.1	448.5	231
1998	605.6	890.7	1,124	648.5	669.1	361.7	163.3	124.8	111.2	114.6	123.4	128.1
1999	377.6	237.5	392	258.6	169.6	126.5	97.3	200.1	722.9	263.2	229.1	341
2000	269	420.7	511.9	581.3	271	318.9	293.9	225.3	362.9	166.1	171.1	312.5
2001	324.8	390.8	506.4	404.5	350.5	595.5	268.5	186.2	169.2	122.2	151.3	166.8
2002	177.4	141.6	244.2	291.5	269.9	135.1	116.5	98.6	124.4	239.7	371.2	477.7
2003	431.2	786.1	1,014	548.4	715.9	1,320	509.9	328.2	1,066	652.9	937.3	1,256
2004	449.5	919.2	507	697.3	437.5	347.3	364.4	336.8	254.7	178.3	343.5	372.1
2005	510.8	383.1	700.6	746.2	414.4	321.9	500	219	107.8	663.9	250	419.3
2006	460.3	473.3	282.4	317.1	223.4	927.8	431.8	129.8	361.6	398.5	684.2	299.2
2007	381.8	376.6	734.5	744.8	209.8	167.7	108.4	112.5	76.6	256.8	145.3	240.5
2008	199	467.3	297.9	439.4	928.3	325.8	205.1	113.2	247.3	154.1	182	338.6
2009	295.7	178.8	212.7	485.3	621.2	879.6	156	192.8	175	372	432.2	1,150
2010	518.2	774.6	1,012	463.8	338.6	180.7	209.5	327.9	191.1	447.1	246.3	246.8
2011	191.3	381.1	645.4	568.6	302.2	145.7	201.6	498.6	1,296	400.9	452.8	784.4
2012	476.1	305.7	479.1	253.8	217.4	229.4	215.1	158.5	131.4	-	-	-
Mean	453	473	593	507	449	372	234	212	289	279	325	436
** No Incomplete data have been used for statistical calculation												

Note: '-' indicates no data

Table 1.1-3 Mean Daily Streamflow for the Patuxent River at Bowie, MD, USGS Station No. 01594440, Patuxent River near Bowie, MD (1977 through 2012)

Day of	Mean of daily mean values for each day in, cfs (Calculation Period 1976-10-01 -> 2012-09-30)											
month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	349	362	521	606	379	356	257	226	181	321	232	348
2	444	451	537	569	393	397	287	212	198	316	223	369
3	468	454	606	637	395	348	240	199	178	250	235	368
4	411	473	615	606	394	445	261	196	170	237	255	387
5	396	505	615	580	426	497	277	209	195	248	285	377
6	339	431	561	502	493	459	275	227	447	273	277	496
7	344	437	573	437	640	463	264	235	499	210	264	392
8	420	379	523	458	487	520	241	191	638	257	334	405
9	434	362	627	487	451	372	287	176	428	415	423	399
10	365	361	629	568	424	329	209	207	261	266	319	472
11	344	350	614	554	402	269	195	187	220	270	311	475
12	414	407	489	463	523	270	170	280	221	199	326	664
13	429	492	481	472	550	282	247	323	208	180	373	508
14	401	440	571	462	390	344	321	273	211	192	314	637
15	450	458	513	506	383	312	287	216	164	269	324	536
16	372	431	476	723	509	272	195	186	242	215	287	491
17	335	411	534	625	629	293	211	167	417	232	432	472
18	376	432	538	543	516	337	186	184	314	286	329	443
19	479	447	534	518	439	416	187	200	344	304	239	482
20	644	439	535	469	409	442	204	179	259	310	300	410
21	605	426	607	478	392	593	229	216	188	270	301	352
22	474	463	780	473	365	506	213	220	248	275	295	375
23	435	621	602	440	420	342	208	173	349	213	396	391
24	481	747	712	397	476	319	210	162	354	265	361	417
25	606	630	578	401	450	308	202	154	243	257	290	490
26	649	661	487	402	535	403	269	182	314	307	311	537
27	708	644	639	439	537	408	235	195	348	375	341	444
28	564	539	746	501	375	331	272	341	311	490	358	358
29	532	439	795	493	352	277	234	284	276	381	557	367
30	403		710	407	427	238	181	191	241	319	445	339
31	368		632		368		188	183		254		325

Table 1.1-4 Maximum Daily Streamflow for the Patuxent River at Bowie, MD, USGS Station No. 01594440, Patuxent River near Bowie, MD (1977 through 2012)

Day of	Maximum of daily mean values for each day for 35 - 36 years of record in, cfs (Calculation Period 1976-10-01 -> 2012-09-30)											
month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	1,140	1,040	2,210	2,040	1,130	1,000	1,010	1,060	615	4,600	1,280	1,250
2	2,460	2,870	2,270	1,680	1,320	1,680	1,680	986	1,780	2,960	837	2,730
3	3,160	1,340	3,090	4,510	1,400	1,220	1,250	744	1,380	2,170	778	2,290
4	1,360	2,790	3,330	2,780	1,190	2,940	1,810	917	785	2,480	936	2,000
5	995	3,320	4,480	1,850	1,260	2,930	1,450	1,070	900	2,370	1,090	1,930
6	1,170	2,550	2,290	1,520	2,350	2,710	1,820	646	7,350	2,470	1,430	4,430
7	932	4,390	2,120	1,000	8,400	1,790	1,830	1,520	7,500	1,430	1,480	1,850
8	1,960	2,700	1,660	1,220	4,020	3,950	896	642	13,700	2,420	2,650	4,030
9	1,920	1,400	3,170	1,160	2,100	2,520	2,750	499	7,000	7,310	4,190	2,520
10	1,650	1,330	3,780	2,320	1,990	1,650	660	1,240	1,960	1,740	3,360	3,770
11	777	1,400	4,140	3,430	1,460	1,160	899	817	1,180	3,350	1,730	2,240
12	2,180	1,310	1,570	1,600	5,120	1,110	599	1,880	1,490	1,220	1,200	5,240
13	2,310	3,750	2,870	1,700	5,930	1,270	1,210	1,460	938	815	1,590	2,670
14	1,560	1,760	5,120	1,500	1,790	2,070	3,800	1,940	1,520	816	934	5,220
15	3,960	2,140	2,050	2,480	1,580	1,610	1,230	756	602	1,560	2,470	2,800
16	1,270	2,270	1,240	4,780	3,630	1,220	631	638	2,740	881	1,360	1,900
17	1,080	1,300	3,860	3,180	4,560	1,280	1,460	654	7,110	1,140	4,080	1,680
18	1,500	1,620	3,350	1,890	2,940	1,480	741	887	1,840	1,600	1,710	2,360
19	1,910	1,370	1,870	1,660	1,550	3,160	534	1,700	4,940	2,030	593	5,700
20	6,350	1,600	1,750	1,220	1,010	3,000	901	568	3,240	2,860	3,010	3,470
21	4,170	1,600	3,190	1,990	1,640	4,280	1,510	1,300	1,040	1,840	2,180	1,410
22	3,920	1,300	3,440	1,990	1,200	3,630	1,100	1,050	1,870	2,300	1,330	1,380
23	2,850	5,600	2,140	1,280	3,000	1,480	975	572	3,450	575	1,880	1,780
24	2,610	4,540	3,770	968	2,170	2,110	768	526	4,890	1,410	1,790	1,460
25	4,650	4,500	2,450	1,880	1,830	1,370	1,350	488	1,900	900	1,030	2,500
26	4,430	8,000	1,350	982	2,690	5,710	1,560	1,030	2,110	1,080	1,390	2,880
27	8,860	8,470	3,720	931	2,580	7,570	1,260	1,560	1,400	2,530	1,290	4,640
28	4,430	4,430	3,420	1,830	1,310	5,160	1,940	4,220	2,330	3,020	1,720	1,940
29	4,110	918	3,440	1,980	893	3,170	1,850	3,680	1,520	2,000	5,190	1,560
30	1,340		3,620	1,390	2,530	1,510	718	833	1,600	2,600	2,720	1,370
31	1,080		2,010		1,500		638	800		1,490		1,550

Table 1.1-5 Minimum Daily Streamflow for the Patuxent River at Bowie, MD, USGS Station No. 01594440, Patuxent River near Bowie, MD (1977 through 2012)

Day of	Minimum of daily mean values for each day for 35 - 36 years of record in, cfs (Calculation Period 1976-10-01 -> 2012-09-30)											
month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	113	105	132	183	122	121	79	69	67	59	82	98
2	115	158	136	182	122	118	88	68	71	64	107	119
3	123	151	169	171	176	115	85	66	71	66	107	118
4	114	148	165	159	173	112	82	66	70	63	105	117
5	121	140	177	152	157	109	79	65	74	63	106	115
6	118	136	175	155	150	110	77	73	75	60	107	110
7	118	128	166	153	143	111	76	70	72	57	106	105
8	119	145	161	140	135	111	76	70	72	57	107	105
9	117	145	157	147	126	108	78	70	69	58	108	107
10	115	136	168	158	120	105	81	73	66	57	105	103
11	107	133	159	157	128	105	81	73	65	57	104	98
12	104	134	150	158	128	105	79	73	64	58	102	99
13	105	141	151	156	115	108	79	74	62	63	98	101
14	104	133	147	154	110	102	85	73	61	76	99	103
15	106	132	147	153	106	99	81	73	59	76	99	124
16	111	134	151	153	102	99	80	71	58	77	101	126
17	112	136	151	149	100	98	78	75	56	77	100	123
18	112	134	149	143	98	94	79	74	56	73	99	120
19	111	132	143	143	105	92	74	70	56	71	97	118
20	112	130	143	140	149	94	73	70	57	71	97	120
21	122	135	139	136	144	105	79	69	60	70	98	106
22	130	135	142	134	139	96	80	66	59	70	95	109
23	130	133	138	131	135	86	79	67	59	71	117	116
24	130	133	137	130	132	87	76	67	57	71	102	115
25	120	134	134	138	120	87	81	66	59	70	96	115
26	120	132	130	127	114	86	78	65	60	84	96	114
27	110	139	131	124	108	81	77	65	60	105	96	114
28	110	132	131	127	119	80	75	64	59	108	94	120
29	100	204	129	127	133	90	73	62	59	106	95	124
30	100		159	123	127	88	77	68	58	96	94	118
31	100		164		123		72	70		86		111

Table 1.1-6 Monthly Mean Streamflow for St. Leonard Creek at St. Leonard, MD, USGS Station No. 01594800, St. Leonard Creek near St. Leonard, MD (1956 through 2003)

YEAR	Monthly mean Discharge in cfs (Calculation Period: 1956-12-01 -> 2003-09-30)											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1956	-	-	-	-	-	-	-	-	-	-	-	6.74
1957	6.02	8.99	12.5	7.7	4.11	2.46	1.57	4.7	7.04	4.15	4.96	11.9
1958	13.3	15.2	22.9	26.1	26.1	13.9	10.8	14.9	9.1	9.65	8.46	8.78
1959	9.2	7.59	8.63	10.1	5.93	4.26	7.81	4.73	2.44	4.18	8.12	7.61
1960	7.98	9.68	8.89	11.8	13.5	9.57	10.5	8.23	13.1	10.4	11	10.9
1961	12.9	24.8	22	20.3	16.5	9.88	6.53	4.63	2.84	3.76	4.37	6.79
1962	7.69	7.51	12.1	14	7.45	6.39	3.85	2.78	3.03	3.79	10.1	6.49
1963	7.16	6.31	12.3	7.46	5.67	8.91	2.3	1.14	2.64	2.31	7.65	5.58
1964	9.67	10.5	9.49	10.8	6.16	3.93	3.45	1.37	1.71	4.06	5.12	5.88
1965	6.58	7.33	9.26	8.59	4.49	4.88	3.56	3.12	2.44	2.69	3.06	3.23
1966	4.33	9.53	5.55	5.49	5.32	1.72	0.8	0.326	4.59	4.42	3.08	4.99
1967	5.19	5.91	6.43	4.29	5.82	2.5	2.14	3.45	1.31	1.73	2.41	7.23
1968	8.75	3.69	9.09	5.43	4.94	5.16	1.17	2.59	1.94	-	-	-
2000	-	-	-	-	-	-	-	-	-	3.35	4.34	6.2
2001	8.94	11.2	11.4	8.65	9.58	9.82	6.91	5.43	2.44	1.91	2.92	3.11
2002	4.25	3.45	4.73	4.32	3.76	1.14	0.074	0.415	1.63	1.93	5.7	6.26
2003	4.89	8.14	11.3	8.21	9.35	9.92	5.49	4.8	8.73	-	-	-
Mean	7.88	9.3	11	10	8.6	6.3	4.5	4.2	4.3	4.2	5.8	6.8
** No Incomplete data have been used for statistical calculation												

Note: '-' indicates no data

Table 1.1-7 Mean Daily Streamflow for St. Leonard Creek at St. Leonard, MD, USGS Station No. 01594800, St. Leonard Creek near St. Leonard, MD (1956 through 2003)

Day of	Mean of daily mean values for each day for 14 - 15 years of record in, cfs (Calculation Period 1956-10-01 -> 2003-09-30)											
month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1	10	8	13	12	9	7	3.5	4.1	3.8	5.3	5.1	4.8
2	8.3	7.1	9.8	11	9.1	9.6	3.6	3.1	3.5	4.1	5.7	4.5
3	7.1	6.4	9.7	10	8.4	14	4.3	3.4	3.6	3.6	6.3	5.9
4	6.7	8.5	8.9	9.8	8	7.9	3.9	3.9	3.4	3.6	4.9	6.6
5	5.9	9.3	11	10	9.6	7.2	3.7	4.8	2.9	3.5	4.6	5.5
6	8.1	8.9	12	9.9	8.5	6.2	4.2	4.5	2.6	4.6	6.7	6.2
7	9.6	9	10	11	11	7.4	3.4	3.3	6.7	4.4	11	5.6
8	7.1	10	11	12	9.9	7.7	4.3	5	2.9	3.6	5.2	5.5
9	7.6	9.6	11	11	9.1	5.8	4.3	2.9	2.7	3.9	4.7	7
10	7.6	8	8.6	9.8	7.5	6.5	4.3	3.1	3.9	3.2	8	6
11	6.6	8.2	8.4	13	9.3	5.2	4.9	3.8	6.2	2.9	4.9	6.7
12	6.4	8.7	13	11	8.5	5.3	5.4	3.8	10	2.8	4.9	9.5
13	6.6	9.9	10	13	7.5	5.8	5	4.8	4.6	2.8	5	7
14	15	8.7	10	11	7.3	7.6	5.8	3.9	4.4	3.7	4.6	7.6
15	10	8.2	8.9	9.3	7	5.5	7.4	2.9	3	3.7	4.7	6.3
16	7.7	9.9	9	9.4	7.4	5.4	5.6	3	3.6	3.4	4.5	8.2
17	6.5	9.3	11	9.2	6.5	5.6	3.4	3.5	4.4	4.4	5.3	8.5
18	6.1	8.8	12	9	7.3	5.2	3.3	2.5	4.1	3.7	5.4	7.8
19	6.5	14	12	9.1	7.8	5.7	3.2	3	5.5	6.1	5.6	5.7
20	9.7	9	17	9.8	8.1	6.3	3	4.8	4.9	5.5	5.2	5.9
21	8.9	8.6	16	9	6.9	6.8	3.4	3	8.4	4	4.7	7
22	8.4	9.3	15	9.4	8.2	5.4	3.1	3.5	3.9	6.1	5.9	5.2
23	8.3	11	13	12	8.5	5.2	3.6	3.2	4.1	4.1	5.2	6.3
24	7.8	9.1	11	9	6.7	6	4.5	4.8	3	4.3	7.2	6.6
25	8.9	10	10	9.1	7.3	6.2	3.5	17	3.1	3.5	7.3	6.7
26	6.7	11	11	8.4	11	5.1	3.9	6.5	2.9	4.1	8.3	7.4
27	6.7	12	11	8.8	9.1	5.5	5.4	4.6	4.8	3.6	5.3	6.8
28	7	11	9.5	11	11	4.1	3.4	3.3	3.4	5.6	5	7.6
29	6.6	8.4	9.1	9.3	13	4.1	4.5	3.4	4.6	6.2	6.5	9.9
30	6.6		11	10	7.8	3.6	12	2.8	4.6	4.5	6.2	9.9
31	6.3		11		8.9		4.1	3.3		4.1		6

Table 1.1-8 Maximum Daily Streamflow for St. Leonard Creek at St. Leonard, MD, USGS Station No. 01594800, St. Leonard Creek near St. Leonard, MD (1956 through 2003)

Day of	Maximum of daily mean values for each day for 14 - 15 years of record in, cfs (Calculation Period 1956-10-01 -> 2003-09-30)										
month	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1	54	28	43	39	22	16	8.7	14	11	29	21	9.4
2	24	18	20	31	27	33	8.7	9.4	13	10	22	8.7
3	18	13	18	25	22	71	15	9.1	10	8.7	22	23
4	12	40	18	23	25	23	11	13	13	9.1	18	19
5	11	30	29	41	54	14	9.8	16	9.4	7.9	15	16
6	27	22	22	30	38	13	16	21	9.1	24	19	19
7	41	25	25	24	51	28	10	11	60	19	60	13
8	11	30	28	43	32	27	13	38	11	9.6	14	17
9	19	50	32	21	28	16	12	9.4	9.1	13	10	32
10	17	29	18	23	25	30	13	7.9	23	7.9	49	24
11	11	22	17	63	46	15	14	8.7	27	6.9	13	16
12	13	22	57	28	32	16	25	15	115	6.6	11	26
13	11	49	21	50	21	24	16	23	18	6.9	15	15
14	90	26	28	35	20	46	19	18	17	14	10	15
15	33	20	18	24	20	12	53	9.1	7.9	7.5	9.8	11
16	16	35	20	23	19	11	34	8.7	14	6.9	9.8	42
17	13	30	30	21	19	18	9.4	15	16	15	17	35
18	12	28	40	19	18	11	8.3	7.5	16	8.5	14	21
19	13	57	29	19	30	17	7.9	9.8	27	41	10	12
20	35	24	95	20	35	15	7.5	28	30	23	9.4	12
21	23	24	48	22	21	22	9.1	11	69	9.8	9.4	25
22	21	26	34	21	24	17	9.1	13	11	40	25	9.5
23	15	47	47	73	31	14	19	13	24	19	11	15
24	21	27	31	30	17	24	22	23	11	16	22	12
25	40	26	27	24	30	16	12	140	8.8	8.3	26	16
26	20	22	29	22	45	18	9.8	32	7.2	8.3	36	26
27	13	46	33	24	35	28	20	18	23	7.9	11	16
28	12	41	26	40	44	11	11	16	11	39	9.8	28
29	13	11	22	26	53	10	18	14	17	47	15	33
30	14		27	27	21	9.1	124	12	19	14	20	36
31	14		33		37		16	12		13		12

Table 1.1-9 Minimum Daily Streamflow for St. Leonard Creek at St. Leonard, MD, USGS Station No. 01594800, St. Leonard Creek near St. Leonard, MD (1956 through 2003)

Day of month	Minimum of daily mean values for each day for 14 - 15 years of record in, cfs (Calculation Period 1956-10-01 -> 2003-09-30)											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
1	1.1	3.5	2.1	4.4	3.5	1.4	0.18	0	0	0.2	1.1	2.4
2	1	3.7	2.8	3.9	3.3	1.1	0.13	0	0	0.15	1.2	2.4
3	1.4	3.4	3.7	3.7	3.5	0.64	0.13	0	0	0.1	1.2	2.6
4	2.1	3.4	4.4	3.7	3.5	0.64	0.13	0	0	0	1.3	2.7
5	2.5	2.8	4	3.7	3.3	0.76	0.1	0	0	0	2	2.8
6	4.2	3.2	3.9	3.5	3.3	1.4	0.08	0.06	0	0	2.1	2.8
7	3.8	3	3.9	3.5	3.1	1.6	0	0	0	0	2.1	2.8
8	3.6	3.5	3.9	3.3	2.9	1.8	0	0	0	0	2.1	2.9
9	3.4	3.6	3.8	3.3	2.9	1	0	0	0	0	2.1	2.9
10	3.7	3.4	4.1	3.3	2.9	1	0	0	0	0.1	2.1	2.8
11	3.3	3.2	4	3.3	2.7	0.64	0	0	0.01	0.2	2.1	2.9
12	3	3	3.9	3.6	3.1	0.64	0	0	0	1	2.1	2.9
13	3.3	2.8	4.4	3.6	3.1	0.9	0	0	0	1.1	2.1	2.6
14	3.3	2.8	4.2	3.4	2.7	1	0.13	0	0	0.5	2.1	3
15	3.2	3	4.1	3.4	2.7	1.1	0.13	0	0.13	0.39	2.3	2.6
16	3	3.2	3.9	2.6	2.9	1	0.06	0	0.4	1.5	2	2.1
17	2.9	3.8	4	2.3	2.6	0.9	0.06	0	0.2	1.5	2.1	2.3
18	2.6	3.6	4.1	1.9	2.3	0.76	0.01	0	0.2	1.5	2.1	3.1
19	2.8	3.5	4.1	2.4	2.6	0.52	0	0	0.23	1.5	2.3	2.7
20	3.1	2.9	3.9	3.4	3.1	0.64	0	0	0.13	0.62	2	2.6
21	3	2.9	3.9	2.9	2.7	0.77	0	0	0.13	0.48	2	2.1
22	3.5	2.8	3.9	4.1	2.7	0.65	0	0.01	0.06	0.35	2	1.9
23	3.5	2.3	3.9	3.7	2.4	0.46	0	0.06	0.06	0.35	2.4	2.1
24	3.1	2.2	4.1	3.7	3.2	0.46	0.13	0	0.01	0.35	2.4	3.3
25	3.4	2.3	3.9	3.3	2.8	0.2	0.01	0	0	0.62	2.6	3.5
26	3.7	2.5	4	3.9	2.7	0.2	0.1	0	0.34	1.8	2.6	2.9
27	3.8	2.9	4.4	4.4	2.5	0.23	0.2	0	0.4	1.7	2.4	2.3
28	3.6	1.8	3.9	4.6	2.1	0.2	0.3	0	0.4	1.6	2.3	1.9
29	3.5	5	3.9	3.9	2.1	0.67	0.23	0	0.52	1.3	2.3	2.1
30	3		3.9	3.7	1.8	0.47	0.01	0	0.32	1.2	1.7	1.5
31	2.5		4.5		1.7		0	0		1.2		1.2

Table 1.1-10 Estimated Monthly Mean Inflow to the Chesapeake Bay Based on Three Reference Stations (1951 through 2000)

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1951	119,400	175,400	148,100	179,100	66,000	87,900	42,100	22,700	16,300	13,600	41,600	82,200	82,100
1952	173,500	123,300	182,100	180,100	142,100	47,000	33,500	30,400	38,800	16,800	68,300	97,100	94,400*
1953	136,000	111,100	170,700	129,000	123,600	74,400	23,700	17,000	12,700	10,800	17,900	48,000	72,800
1954	39,700	71,800	135,100	95,200	99,900	45,400	19,100	14,000	13,600	41,600	51,100	78,300	58,700
1955	83,300	79,400	208,800	90,300	46,300	44,900	19,100	93,400	26,800	79,700	74,000	33,300	73,400
1956	27,400	107,900	161,400	161,500	82,800	45,000	49,900	39,500	30,700	36,400	69,400	101,900	76,000
1957	76,400	109,900	114,800	183,800	62,700	37,500	19,500	11,900	17,900	19,700	30,600	93,000	64,400
1958	89,100	72,900	160,900	238,900	154,400	51,500	43,000	40,400	24,900	25,400	37,400	37,500	81,400*
1959	72,800	71,900	96,700	138,200	69,800	46,100	20,600	18,900	19,100	55,400	70,500	117,700	66,400
1960	95,500	118,100	84,000	230,700	145,700	92,900	32,100	26,100	42,600	22,100	24,300	20,100	77,400*
1961	30,000	144,300	181,400	202,900	111,000	55,700	31,700	29,200	23,200	38,000	31,500	63,800	78,000
1962	78,500	71,800	207,200	195,300	61,000	38,800	21,900	16,800	13,700	31,500	60,500	41,700	69,800
1963	65,800	43,200	228,600	86,400	55,700	40,600	17,200	12,200	10,600	8,600	18,800	38,200	52,400
1964	103,400	80,600	222,700	127,300	88,700	23,600	16,300	11,400	7,800	13,000	14,000	33,200	61,900
1965	65,200	110,300	118,000	112,900	59,300	23,900	13,000	12,000	11,700	21,300	20,500	25,500	49,000
1966	29,600	110,200	130,100	66,500	105,800	30,700	10,500	9,300	23,600	35,000	30,500	61,400	53,300
1967	61,000	67,000	205,100	101,300	120,900	38,700	30,600	47,800	27,500	51,000	67,000	104,600	77,200
1968	62,800	86,600	129,100	64,800	81,200	86,000	31,500	16,900	23,700	19,700	67,900	52,600	60,100
1969	46,900	58,800	68,800	93,100	57,300	39,100	36,200	80,700	23,800	17,300	44,300	62,200	52,300
1970	66,200	132,000	95,800	218,500	73,500	38,200	39,100	24,400	17,300	29,000	111,800	80,200	76,500
1971	74,100	163,000	167,400	73,300	104,500	68,000	22,900	32,400	32,400	54,500	47,300	108,800	78,600
1972	82,300	107,700	183,500	159,600	145,300	324,600	117,100	42,400	19,900	58,600	131,800	209,000	131,700
1973	108,400	144,800	138,500	174,700	127,000	76,400	44,900	34,500	30,100	34,600	52,300	176,000	94,900
1974	153,900	88,600	109,000	156,000	81,000	56,500	40,100	27,400	46,800	25,200	38,500	99,400	76,800
1975	97,600	155,600	185,000	96,700	121,800	77,700	56,100	30,200	155,100	118,000	77,400	66,000	102,700
1976	118,200	155,400	104,400	85,900	59,400	74,400	41,900	33,600	22,900	173,900	73,400	68,300	84,100
1977	31,100	34,500	195,600	152,600	49,830	23,800	29,700	22,600	44,600	97,400	124,100	155,100	80,400

Table 1.1-10 Estimated Monthly Mean Inflow to the Chesapeake Bay Based on Three Reference Stations (1951 through 2000) Sheet 2 of 2

Year	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1978	171,800	75,800	231,600	158,500	182,700	53,500	39,900	46,100	25,300	22,500	25,600	61,900	91,700
1979	188,700	131,200	253,400	122,200	94,600	81,500	32,600	34,300	98,800	132,600	107,800	87,200	113,700
1980	88,800	42,900	151,000	205,200	104,800	40,800	28,800	21,000	15,000	14,000	24,200	31,100	64,000
1981	17,800	151,900	58,600	69,600	78,900	68,900	36,100	20,600	23,200	31,100	50,000	44,200	53,500
1982	60,900	134,900	169,900	123,000	54,100	147,200	42,400	25,900	15,700	17,300	25,700	58,500	72,400
1983	39,500	100,800	128,500	264,000	149,000	64,400	33,300	16,900	13,000	26,800	55,100	167,000	88,000
1984	56,000	216,300	151,000	251,000	134,000	76,000	62,700	73,600	27,900	22,800	33,200	92,300	99,100
1985	62,100	97,000	95,100	86,000	58,800	40,800	25,700	36,600	21,100	28,700	164,000	104,300	68,000
1986	53,700	125,000	169,000	95,800	52,400	45,900	29,700	31,900	17,400	25,900	72,400	114,200	69,100
1987	68,200	69,500	121,100	226,000	76,500	38,600	36,200	15,400	73,600	33,100	47,600	82,100	73,800
1988	66,500	93,200	78,300	70,300	139,000	36,400	21,000	17,600	25,200	16,900	47,200	31,200	53,400
1989	49,100	54,100	97,300	104,900	223,900	117,800	87,000	39,800	47,700	76,600	71,800	37,500	84,200
1990	94,100	153,600	78,600	104,300	113,500	67,200	50,500	38,100	30,300	135,400	80,400	134,800	89,800
1991	167,200	94,600	156,500	110,400	58,800	24,600	24,400	21,700	13,200	14,700	21,900	51,900	63,300
1992	58,000	54,600	124,800	134,600	77,600	68,400	43,300	37,900	38,200	35,700	92,600	103,600	72,400
1993	125,300	58,500	230,700	380,700	89,000	35,800	20,900	18,000	20,400	24,000	78,600	125,200	100,600
1994	69,500	152,400	298,000	230,500	87,500	45,300	43,800	83,900	37,400	28,500	41,300	83,300	99,800
1995	128,600	55,000	88,100	58,400	61,700	77,000	60,500	20,700	13,200	63,500	80,100	69,800	64,900
1996	244,600	142,200	152,200	139,000	155,900	86,300	54,100	57,600	142,000	97,900	130,000	219,900	135,200
1997	77,500	109,700	160,300	80,500	61,300	62,200	25,000	19,000	20,300	18,600	81,400	55,800	64,000
1998	199,700	235,900	223,100	178,700	152,900	57,300	41,600	20,900	14,100	17,500	14,400	16,400	97,700
1999	80,200	70,500	108,000	98,800	45,000	17,400	13,100	13,600	47,300	48,700	33,200	66,400	53,500
2000	44,500	85,200	141,500	149,000	83,300	70,000	34,000	34,200					

* Minor arithmetic corrections made in the source data since the original publication in Reference 1.1-13.

Table 1.1-11 Details of Brighton and Rocky Gorge Dams

Information	Brighton Dam	Rocky Gorge Dam
Record Number	26707	26722
Dam Name	Brighton Dam	Rocky Gorge Dam
Other Dam Name	Tridelphia Lake Dam	Duckett Dam
State ID	5	20
NID ID	MD00005	MD00020
Longitude (decimal degree)	-77.005	-76.8767
Latitude (decimal degree)	39.1933	39.1167
County	Montgomery	Prince Georges
River	Patuxent River	Patuxent River
Owner Name	Washington Suburban Sanitary Commission	Washington Suburban Sanitary Commission
Year Completed	1943	1953
Year Modified	1999	1986
Dam Length (ft, top of the dam)	995	840
Dam Height (to the nearest ft)	80	134
Maximum Discharge (cfs)	83,000	65,200
Maximum Storage (ac-ft)	27,000	22,000
Normal Storage (ac-ft)	19,000	17,000
Surface Area (acres)	800	773
Drainage Area (mi^2)	77.3	132.0
Down Stream Hazard Potential	High	High
State Regulated Agency	MD Water Management Administration	MD Water Management Administration
Spillway Type	Controlled	Controlled
Spillway Width (to the nearest ft)	260	190

Figure 1.1-1 CCNPP Units 1 & 2 and Proposed Unit 3 Site Arrangements and Layout

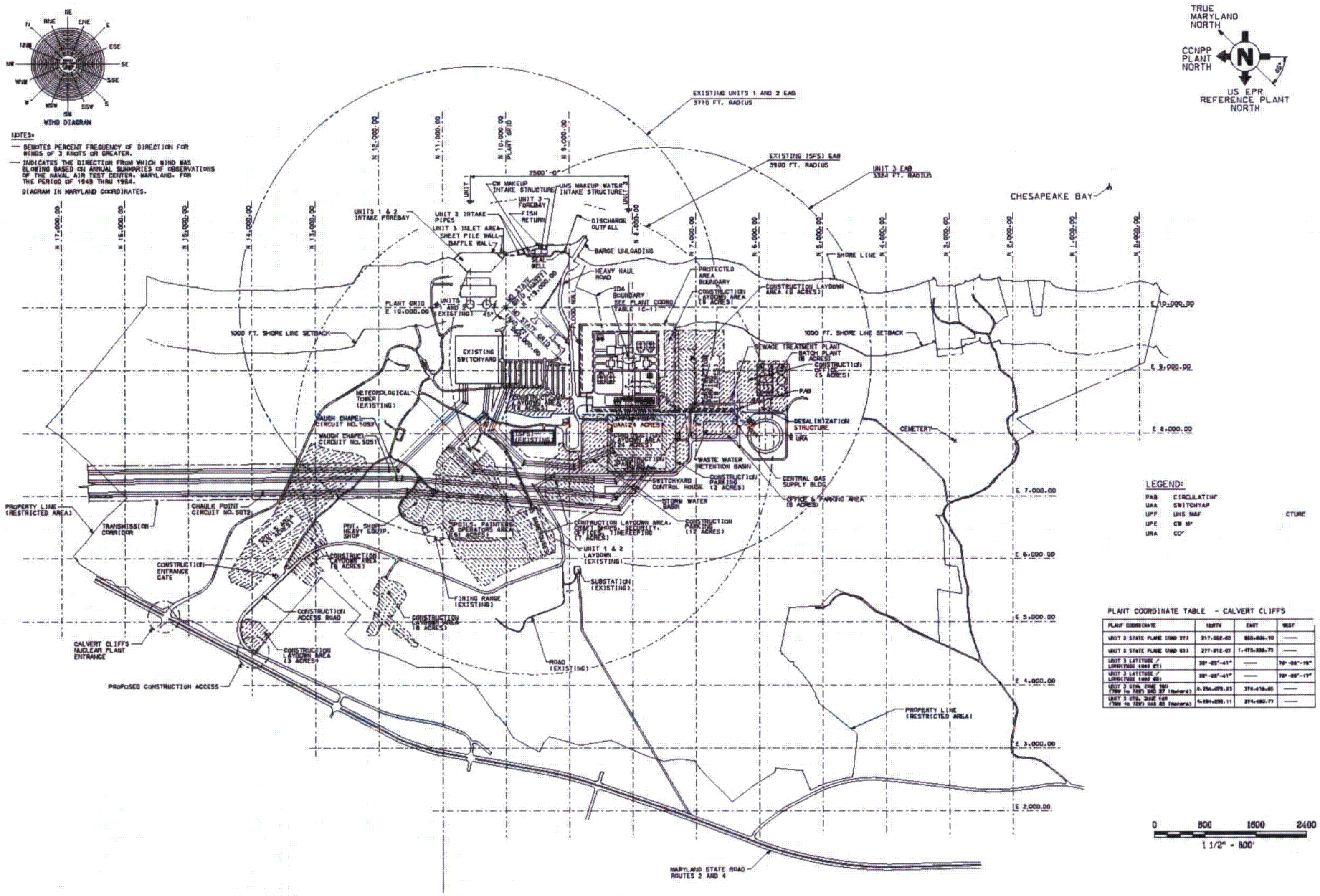


Figure 1.1-2 Site Area Topography and Drainage

(Modified from Reference 1.1-1)

Figure 1.1-3 CCNPP Units 1 & 2 Plant Property and Buildings (Reference 1.1-4)

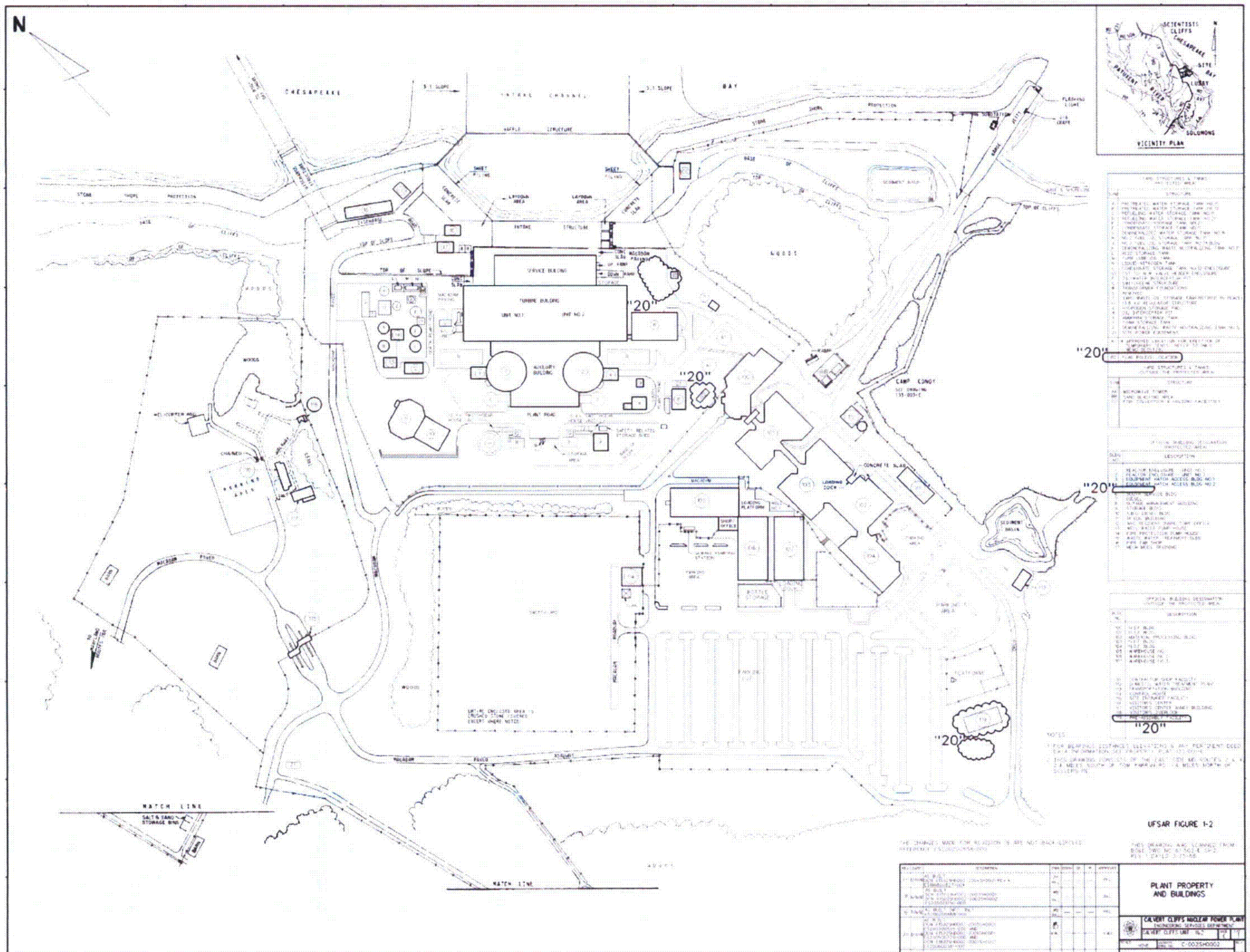


Figure 1.1-4 Chesapeake Bay Sub-Watershed

(Modified from Reference 1.1-1)

Figure 1.1-5 Mean, Maximum and Minimum Monthly Streamflows for the Patuxent River at Bowie, MD, USGS Station No. 01594440, Patuxent River Near Bowie, MD (1977-06-01 through 2005-09-30)

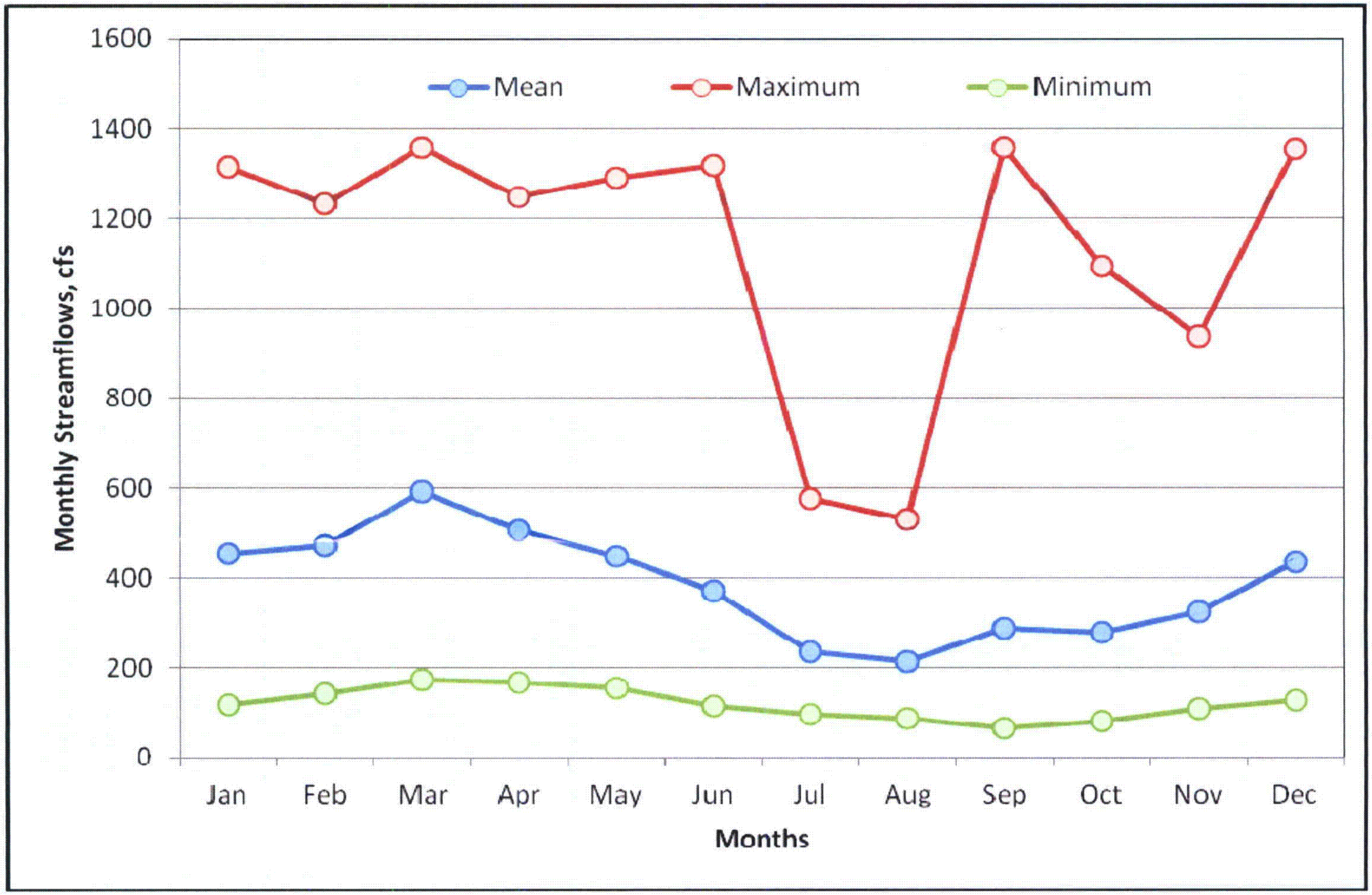
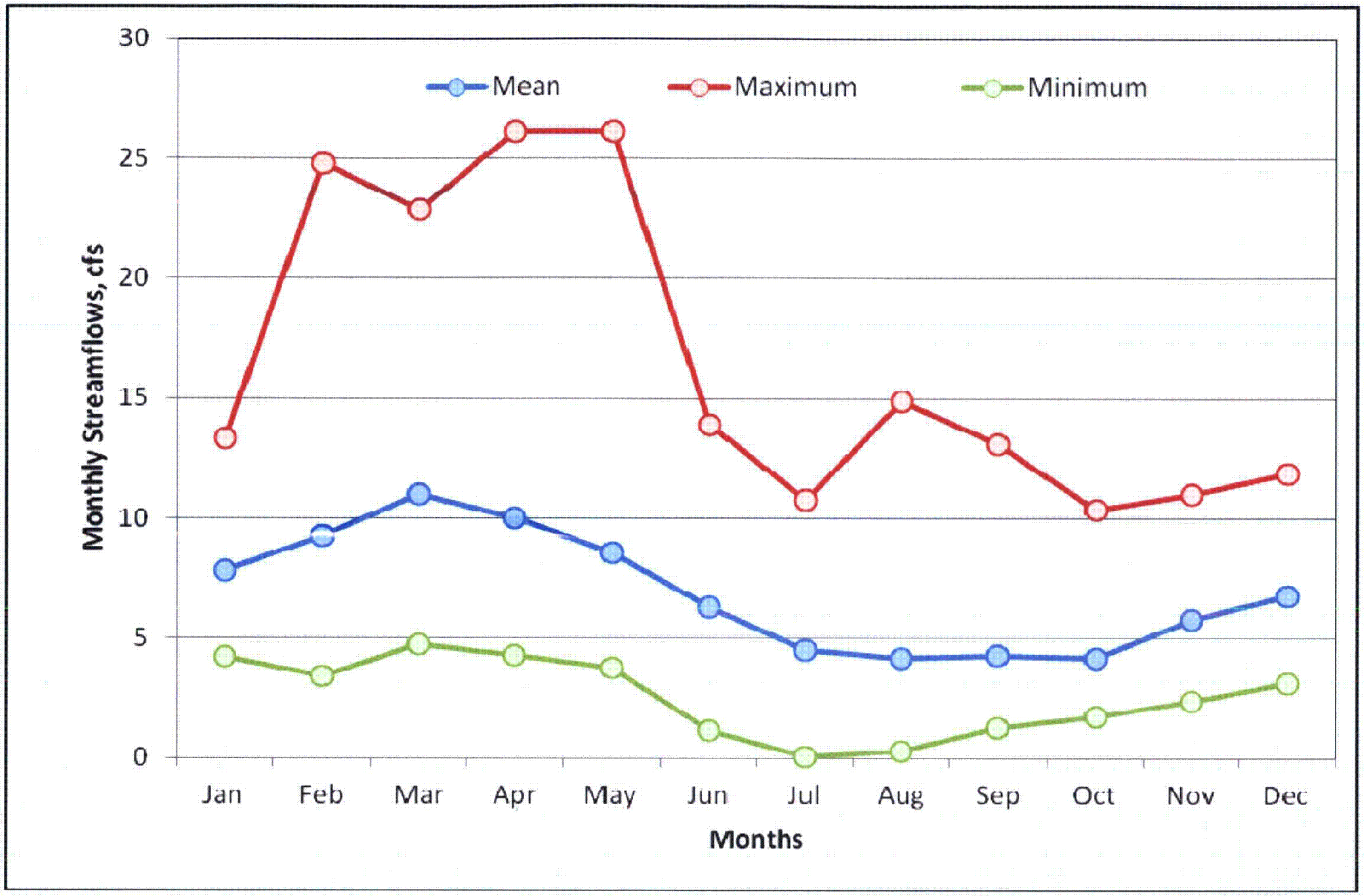


Figure 1.1-6 Sub-Watershed Delineation of the Lower Patuxent River Watershed, which is Indicated as WRAS Project Ares in Legend

(Modified from Reference 1.1-1)

Figure 1.1-7 Mean, Max and Min Monthly Streamflows for St. Leonard Creek at St. Leonard, MD USGS Station No. 01594800, St. Leonard Creek Near St. Leonard, MD (1956-12-01 Through 2003-09-30)



1.2 Current Design Basis Flood Elevations

The current design basis flood elevations for different flood-causing mechanisms are listed in Table 1.2-1. These elevations are documented in Sections 2.5, 2.6, and 2.8 of the CCNPP Units 1 & 2 UFSAR, Revision 43 (Reference 1.2-1). Several flood-causing mechanisms are described in the UFSAR, and flood elevations from controlling events are provided. The design basis flood elevation at the intake structure is given as 27.1 ft NGVD 29, which is due to the probable maximum storm surge (PMSS) and coincidental wind-wave runup on the structure where the wave runup on the intake structure was determined through a series of physical model tests. The design basis flood elevation at the main plant area is due to local intense precipitation with a maximum flood elevation of 44.8 ft NGVD 29 at the EDG Building. The floor elevation of the EDG Building is 45.5 ft NGVD 29. The Station Blackout Diesel Generator Building is located near the EDG Building. Floor elevations and entrances/openings to other safety-related and important-to-safety SSCs at this location are at grade elevation, which is approximately 45 ft NGVD 29.

As described in Section 2.8.3.6 of the CCNPP Units 1 & 2 UFSAR (Reference 1.2-1), the intake structure roof hatches have curbs around them that are at least 6 in. high, providing additional margin against flooding. The roof and hatch covers were constructed for a live loading of 250 psf. The louvered housings for intake structure air supply units and exhaust vents were designed for a live load of 100 psf.

The baffle wall in the intake channel was designed for conditions less than the PMH. However, they would not damage or block the intake structure even if sections of the baffle wall came loose. The concrete stop logs are stored in a recess to the south of the pump house. Thus, they are not considered to be a missile for the intake structure (Section 2.8.3.6, Reference 1.2-1).

Scour at the front edge of the intake structure is not expected as the velocity in this area remains very low (about 1 ft/sec), and the foundation soil is a dense silty sand or sandy silt (Section 2.8.3.6, Reference 1.2-1).

Table 1.2-1 Current Design Basis Flood Elevations

Flooding Mechanism	Flood Critical Plant Structures	Still Water Level (ft NGVD 29)	Wind-wave Runup (1) (ft)	Design Basis Flood Level (2) (ft NGVD 29)	UFSAR Section (3)
Local Intense Precipitation	EDG Building (4)	44.8	Not Applicable	44.8	2.5.2
Flooding in Streams and Rivers	No (5) Flooding Expected	No Flooding Expected	Not Applicable	No Flooding Expected	2.5.2
Upstream Dam Failures	Not (6) Evaluated	Not Evaluated	Not Evaluated	Not Evaluated	Not Evaluated
Storm Surge	Intake Structure	17.6	9.5 (7)	27.1	2.8.3
Seiche	Not Evaluated	Not Evaluated	Not Applicable	Not Evaluated	Not Evaluated
Tsunami	Intake Structure	No Flooding Expected	Not Evaluated	No Flooding Expected	2.6.6
Ice Induced Flooding	Not Evaluated	Not (6) Evaluated	Not Applicable	No Flooding Expected	Not Evaluated
Channel Migration or Diversion	Intake Structure	No Flooding Expected (8)	Not Applicable	No Flooding Expected	2.4.4

Notes:
(1) Wind-wave runup estimated at the intake structure only
(2) The maximum water level from the flooding mechanism considered
(3) Section number in CCNPP Units 1 & 2 UFSAR (Reference 1.2-1)
(4) Emergency Diesel Generator Building
(5) While flooding mechanism is described in the UFSAR, no flooding elevation is established
(6) Flooding mechanism was not considered in the UFSAR
(7) Wave runup adjusted in the UFSAR for the 1 percent wave height from physical scale model data
(8) Shoreline erosion near the site evaluated and shore protection measures constructed

1.2.1 References

1.2-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

1.3 Licensing Basis Flood-Related and Flood Protection Changes

There has been no change to design basis flooding elevations or flooding protection designs beyond what is described in the CCNPP Units 1 & 2 UFSAR (Reference 1.3-1). The Flooding Walkdown Report (Reference 1.3-2) also did not identify any deficiency in the current flooding protection measures.

1.3.1 References

1.3-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

1.3-2 Constellation Energy Nuclear Group, LLC, *Flooding Walkdown Report*, Report No: SL-011462, Revision 0, November, 2012.

1.4 Watershed and Local Area Changes

The CCNPP site is located primarily within the Maryland Lower Western Shore Watershed and St. Lenard River Sub-Watershed of the Lower Patuxent River Watershed. The drainage divide generally runs parallel to the shoreline at the site location with most of the plant area draining toward the Chesapeake Bay. As a result, the site area includes several stream headwaters that facilitate drainage away from the plant naturally. While changes in the watershed area affect overall runoff patterns from the watershed, the impacts to the CCNPP site remain negligible as there has been almost no change to site topography, land use, and vegetation cover.

Overall, land use in Calvert County has changed due to residential and industrial development that kept pace with population growth. Between 1970 and 2005, the county population experienced about a four-fold increase (Reference 1.4-1). Several growth management initiatives at the state and county levels are being implemented for sustainable land preservation. These include zoning laws, transferable development rights, the Maryland Rural Legacy Program, etc. The current land use pattern in Calvert County includes about 38 percent of land permanently preserved, 34 percent of land for low-intensity development, and about 28 percent of land that is non-developed and non-preserved (Reference 1.4-1).

While watershed changes have taken place in the Maryland Lower Western Shore or Lower Patuxent River watersheds since the CCNPP Units 1 & 2 plants were built, these changes did not affect the site, which is located at the headwater areas in these sub-watersheds.

Vertical vehicle barriers were built on the landward sides of the CCNPP Units 1 & 2 plant. These vehicle barriers could potentially change the drainage flow paths near the plant and divert runoff which otherwise would flow towards the plant. These vehicle barriers are included in the site drainage analysis due to a local intense precipitation event as described in Section 2.1.

CCNPP also applied for a combined license for a new unit within the CCNPP site. The new unit, CCNPP Unit 3, would be located south–southeast of the CCNPP Units 1 & 2 plant area (References 1.4-2). While the new unit would modify the current land use pattern at the site, the changes are not expected to impact the flooding behavior of the CCNPP Units 1 & 2 as described in Section 2.1.

1.4.1 References

1.4-1 Maryland Department of Natural Resources, *Preserving Farm and Forestland in Calvert County, Maryland*, Website Address: http://www.dnr.state.md.us/forests/download/ForestryTDRProgram.pdf, Date Accessed: November 14, 2012.

1.4-2 Unistar Nuclear Services, LLC., *Calvert Cliffs Nuclear Power Plant Unit 3, Combined License Application*, Revision 8, March 2012.

1.5 Current Licensing Basis Flood Protection and Mitigation Features

The maximum storm surge elevation in the Chesapeake Bay from a PMH constitutes the design basis flood elevation for the CCNPP Units 1 & 2 intake structure. According to UFSAR Section 2.8.3.6 (Reference 1.5-1), the design of the intake structure includes appropriate protection against the design storm surge and associated wave impacts. Also, procedural measures are in place to address severe weather conditions that may inhibit safe functioning of CCNPP Units 1 & 2 (Reference 1.5-2).

The maximum storm surge elevation, including wind-wave runup, at the intake structure is 27.1 ft NGVD 29. The roof of the intake structure is located at elevation 28.5 ft NGVD 29. The roof of the intake structure has watertight hatches to provide access to saltwater and circulating water pumps for maintenance. An intake structure air supply unit is mounted on each saltwater pump hatch, and an air exhaust vent is mounted on each circulating water pump hatch. The watertight personnel door is located at the north end of the intake structure.

CCNPP Units 1 & 2 plant area drainage facilities are designed to drain intense precipitation events away from the plant. Local probable maximum precipitation (PMP) results in a flood elevation of 44.8 ft NGVD 29 at the EDG Building. Floor elevation of this building is located at 45.5 ft NGVD 29, which eliminates any flooding of the building for a local PMP event. Entrance openings to other safety-related and important-to-safety SSCs in the plant area are located at site grade of 45 ft NGVD 29.

The Flooding Walkdown Report (Reference 1.5-3) summarizes the performances of the plant features credited in the current licensing basis (CLB) for protection and mitigation from external flood events. The site walkdown of flood mitigation features was performed in response to Enclosure 4 of the 50.54(f) letter (NTTF Recommendation 2.3: Flooding) in which the NRC requests that licensees "perform flood protection walkdowns to identify and address plant-specific degraded, nonconforming, or unanalyzed conditions and cliff-edge effects (through the corrective action program) and verify the adequacy of monitoring and maintenance procedures." The walkdown report (Reference 1.5-3) concludes that the flood protection features in aggregate would perform their design function as credited in the CLB.

1.5.1 References

1.5-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

1.5-2 Constellation Energy, *Severe Weather Preparation, Calvert Cliffs Nuclear Power Plant Station Administrative Procedure EP-1-108*, Revision 00400.

1.5-3 Constellation Energy Nuclear Group, LLC, *Flooding Walkdown Report*, Report No: SL-011462, Revision 0, November, 2012.

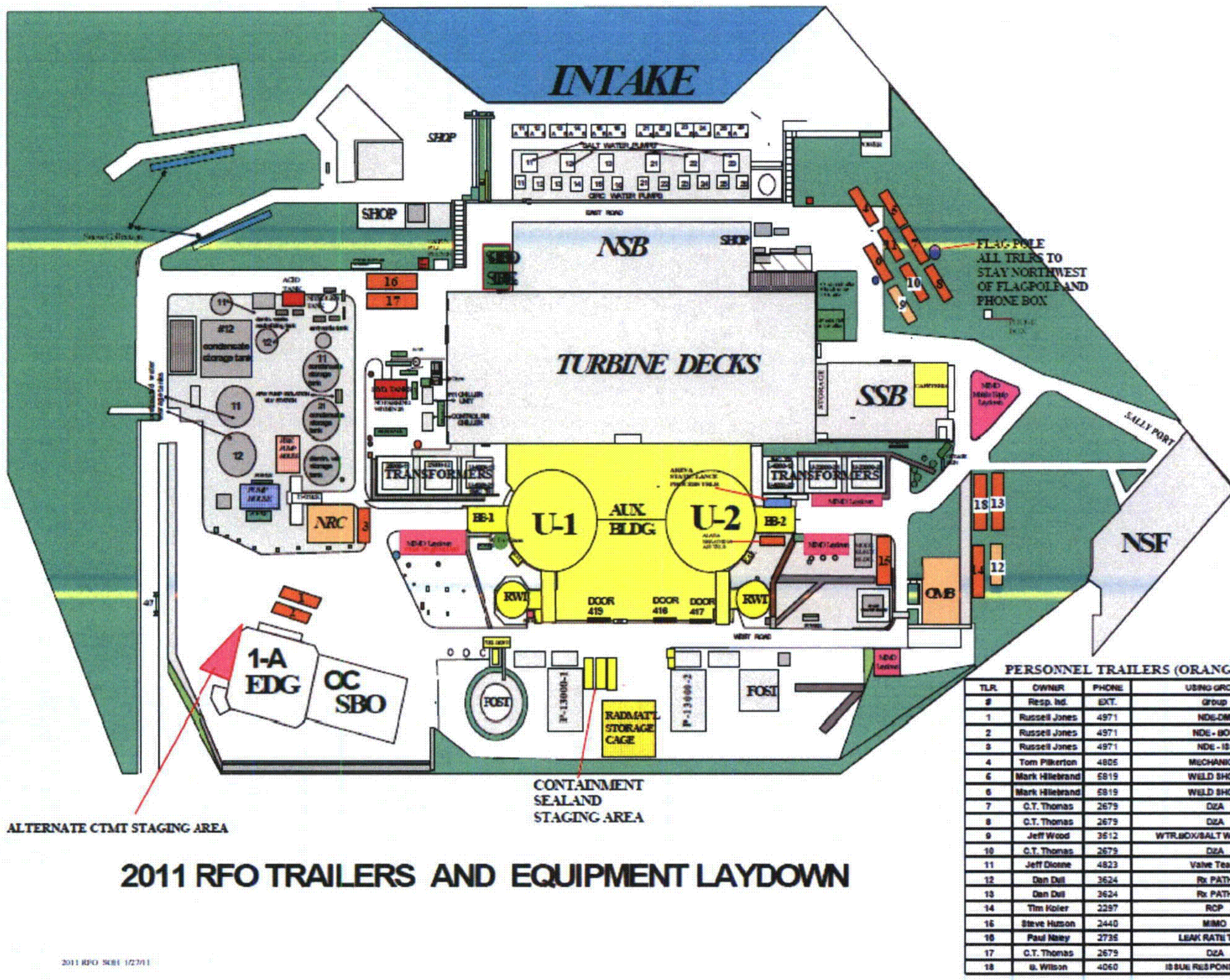
1.6 Additional Site Details

CCNPP Units 1 & 2 typically performs an outage on one unit each year during springtime. The average outage time extends to about 60 days, with a maximum outage of about 90 days. Temporary trailers are placed within the plant's protected area to facilitate work. Figures 1.6-1 and 1.6-2 show schematics of trailer locations during 2011 and 2012 outages, respectively. The effects of plant structures, including storage tanks and typical trailers, are accounted for in the local PMP analysis as structures and ineffective flow areas, described in Section 2.1. The plant procedure for severe weather preparations describes the requirements for securing the temporary trailers (Reference 1.6-1).

1.6.1 References

1.6-1 Constellation Energy, *Severe Weather Preparation, Calvert Cliffs Nuclear Power Plant Station Administrative Procedure EP-1-108*, Revision 00400.

Figure 1.6-1 Schematic Layout of Temporary Trailers during Planned Plant Outage in 2011

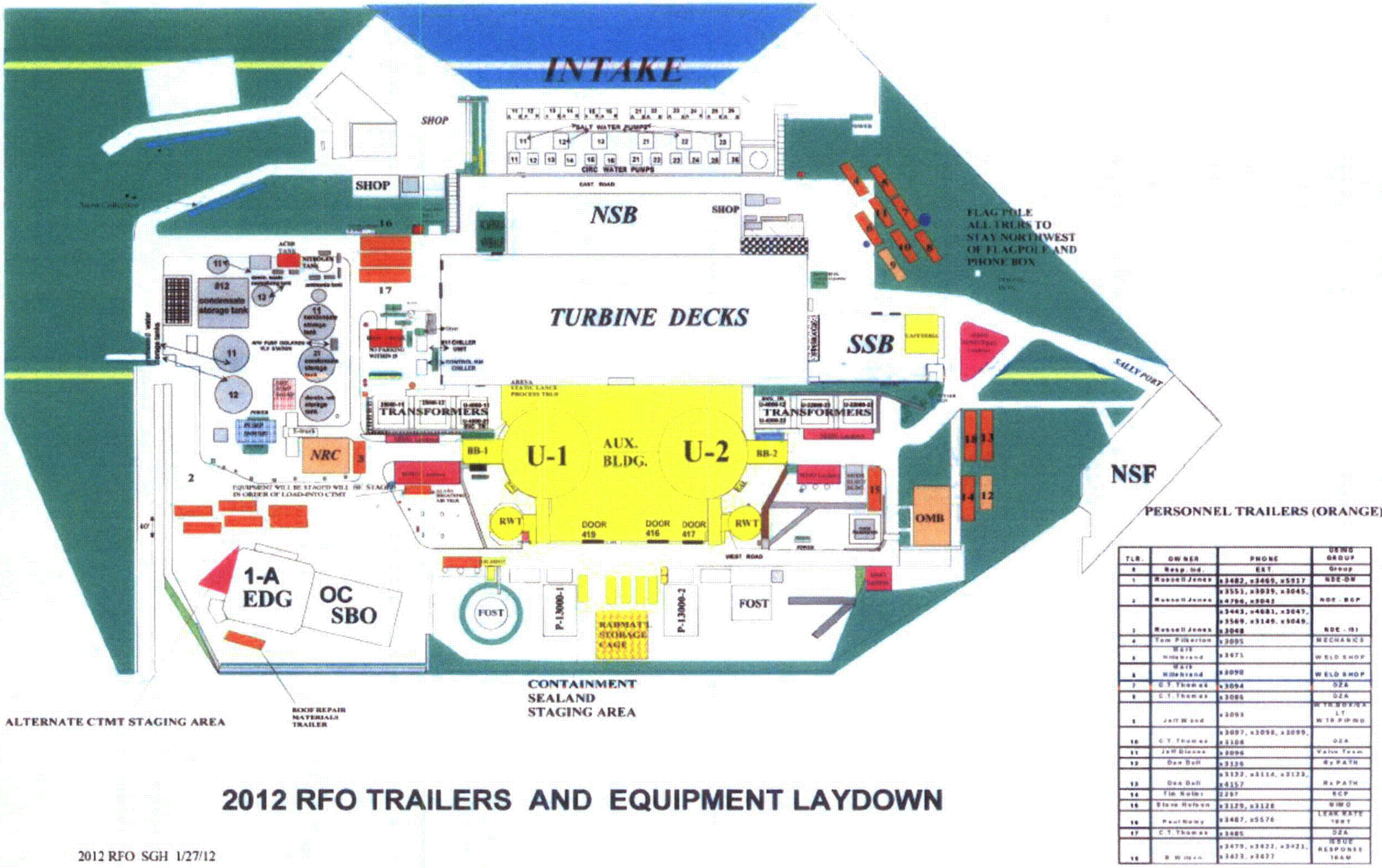


PERSONNEL TRAILERS (ORANGE)

TLR.	OWNER	PHONE	USING GROUP
#	Resp. Ind.	EXT.	Group
1	Russell Jones	4971	NDE-DM
2	Russell Jones	4971	NDE - BOP
3	Russell Jones	4971	NDE - ISI
4	Tom Pilkerton	4805	MECHANICS
5	Mark Hillebrand	5819	WELD SHOP
6	Mark Hillebrand	5819	WELD SHOP
7	C.T. Thomas	2679	D&A
8	C.T. Thomas	2679	D&A
9	Jeff Wood	3612	WTR.BOX/SALT WTR.PIPING
10	C.T. Thomas	2679	D&A
11	Jeff Dionne	4823	Valve Team
12	Dan Dill	3624	Rx PATH
13	Dan Dill	3624	Rx PATH
14	Tim Koller	2297	RCP
15	Steve Hutson	2440	MIMO
16	Paul Nally	2735	LEAK RATE TEST
17	C.T. Thomas	2679	D&A
18	B. Wilson	4060	ISSUE RESPONSE TEAM

Abbreviations: SB: Service Building; EDG: Emergency Diesel Generator Building; SBO: Augmented Quality Station Blackout Diesel Generator Building

Figure 1.6-2 Schematic Layout of Temporary Trailers during Planned Plant Outage in 2012



TLR.	OWNER	PHONE	USING GROUP
#	Resp. Ind.	EXT	Group
1	Russell Jones	x3482, x3469, x5917	NDE-DW
2	Russell Jones	x3551, x3039, x3045, x4786, x3043	NDE - BOP
3	Russell Jones	x3443, x4681, x3047, x3569, x3149, x3049, x3048	NDE - ISI
4	Tom Pilkerton	x3095	MECHANICS
5	Walt Hillebrand	x3671	WELD SHOP
6	Walt Hillebrand	x3090	WELD SHOP
7	C.T. Thomas	x3094	OZA
8	C.T. Thomas	x3086	OZA
9	Jeff Wood	x3093	W TR BOX/NA LT W TR PIPING
10	C.T. Thomas	x3097, x3098, x3099, x3108	OZA
11	Jeff Dionne	x3096	Valve Team
12	Dan Bull	x3126	Rx PATH
13	Dan Bull	x3132, x3114, x3123, x4157	Rx PATH
14	Tim Kuhn	2297	RCP
15	Steve Hutson	x3129, x3128	MIMO
16	Paul Nemy	x3487, x5576	LEAK RATE TEST
17	C.T. Thomas	x3485	OZA
18	B. Wilson	x3479, x3422, x3421, x3423, x3477	ISSUE RESPONSE TEAM

Abbreviations: SB: Service Building; EDG: Emergency Diesel Generator Building; SBO: Augmented Quality Station Blackout Diesel Generator Building

2 FLOODING HAZARD REEVALUATION

Flooding hazards from various flood-causing mechanisms were evaluated for CCNPP Units 1 & 2 in accordance with Enclosure 2 of the NRC's March 12, 2012, 50.54(f) Request for Information Letter (Reference 2.0-1), which identifies the requirements for the flooding hazard reevaluations associated with NTTF Recommendation 2.1. The flooding hazard reevaluation for CCNPP Units 1 & 2 follow the hierarchical hazard assessment (HHA) process described in NUREG/CR-7046 (Reference 2.0-2).

As explained in Attachment 1 to Enclosure 2 of the NRC's 50.54(f) letter, HHA is a progressively refined, stepwise estimation of the site-specific hazards that evaluates the safety of the site with the most conservative, plausible assumptions consistent with available data. Consistent with the HHA approach, flooding mechanisms that are determined to not be the controlling factors of the design basis flood will be screened out using order-of-magnitude analysis or qualitative assessments, where appropriate, with conservative assumptions and physical reasoning based on the physical, hydrological and geological settings of the site.

For the flooding mechanism(s) that will potentially control or affect the design basis, detailed analyses will be performed based on present-day methodologies and standards. The CCNPP Units 1 & 2 flooding reevaluation applies the flooding hazard analysis approaches, regulatory guidance, and methodologies used in support of the preparation of the Combined License Application (COLA) for a future unit at the site (CCNPP Unit 3) (Reference 2.0-3), which is augmented by recent site-specific information. The principal regulations and guidance related to flooding hazard evaluations and the determination of design basis floods include:

- 10 CFR 50 Appendix A (Reference 2.0-4)
- 10 CFR 52.79 (Reference 2.0-5)
- 10 CFR 100.20 (Reference 2.0-6)
- Regulatory Guides 1.59 (Reference 2.0-7), 1.102 (Reference 2.0-8), and 1.206 (Reference 2.0-9)
- Standard Review Plan (NUREG 0800) Sections 2.4.1 to 2.4.7 and 2.4.9 to 2.4.10 (Reference 2.0-10)
- NUREG/CR-7046 (Reference 2.0-2)
- NUREG/CR-6966 (Reference 2.0-11)
- ANSI/ANS-2.8-1992 (Reference 2.0-12)

This chapter describes in detail the reevaluation effort for each plausible flooding mechanism and the potential impacts to the safety-related and important-to-safety SSCs of the plant: flooding impacts due to local intense precipitation (Section 2.1), flooding in streams and rivers (Section 2.2), dam breaches and failures (Section 2.3), storm surge (Section 2.4), seiche (Section 2.5), tsunami (Section 2.6), ice induced flooding (Section

2.7), channel migration or diversion (Section 2.8), and combined effect flood (Section 2.9).

2.0 References

2.0-1 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*, ADAMS Accession No. ML12053A340, March 12, 2012.

2.0-2 U.S. Nuclear Regulatory Commission, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America*, NUREG/CR-7046, November 2011.

2.0-3 Unistar Nuclear Services, LLC. *Calvert Cliffs Nuclear Power Plant Unit 3, Combined License Application*, Revision 8, March 2012.

2.0-4 U.S. Nuclear Regulatory Commission, *Title 10 of Code of Federal Regulations, Appendix A to Part 50—General Design Criteria for Nuclear Power Plants*, 10 CFR 50, Appendix A, December 2012.

2.0-5 U.S. Nuclear Regulatory Commission, *Title 10 of Code of Federal Regulations, Part 52.79 Contents of applications; technical information in final safety analysis report*, 10 CFR 52.79, December 2012.

2.0-6 U.S. Nuclear Regulatory Commission, *Title 10 of Code of Federal Regulations, Part 100.20 Evaluation Factors for Stationary Power Reactor Site Applications on or After January 10, 1997*, 10 CFR 100.20, December 2012.

2.0-7 U.S. Nuclear Regulatory Commission, *Design Basis Floods for Nuclear Power Plants,* Regulatory Guide 1.59, Revision 2, August 1977.

2.0-8 U.S. Nuclear Regulatory Commission, *Flood Protection for Nuclear Power Plants,* Regulatory Guide 1.102, Revision 1, September 1976.

2.0-9 U.S. Nuclear Regulatory Commission, *Combined License Applications for Nuclear Power Plants,* Regulatory Guide 1.206, June 2007.

2.0-10 U.S. Nuclear Regulatory Commission, *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition — Site Characteristics and Site Parameters (NUREG-0800, Chapter 2), Sections 2.4.1 through 2.4.10*, March 2007.

2.0-11 Prasad, R., *Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America - Final Report*, Pacific Northwest National Laboratory, NUREG/CR-6966, U.S. Nuclear Regulatory Commission, March 2009.

2.0-12 American National Standards Institute/American Nuclear Society, *Determining Design Basis Flooding at Power Reactor Sites*, ANSI/ANS-2.8-1992, Nuclear Standard 2.8, 1992.

2.1 Local Intense Precipitation

The impact of local intense precipitation, also referred to as local PMP, is discussed in the CCNPP Units 1 & 2 UFSAR (Reference 2.1-1). According to the UFSAR, the current drainage network in the power block area consists of surface ditches and underground culverts that convey stormwater runoff away from the plant area. Also, grade elevations in the vicinity of the EDG and Station Black Out (SBO) Buildings provide a system of swales that directs the local PMP runoff towards Chesapeake Bay without interfering with the plant stormwater drainage system. The maximum flood water level for the diesel generator buildings was evaluated in the UFSAR (Reference 2.1-1) using the U.S. Army Corps of Engineers (USACE) computer programs HEC-1 and HEC-2. The analysis indicated that during the local PMP storm event the maximum water level near the EDG and SBO Buildings would be 44.8 ft NGVD 29. This water level is below the floor elevation of the EDG and SBO Buildings, which is 45.5 ft NGVD 29. This eliminates any potential flooding of the EDG and SBO Buildings during the local PMP storm event.

The reevaluation of the flooding hazards on the safety-related or important-to-safety SSCs of the CCNPP Units 1 & 2 due to a local intense precipitation (or local PMP) event is described in this section. Guidelines detailed in U.S. Nuclear Regulatory Commission (NRC) NUREG/CR-7046 (Reference 2.1-2), NRC Regulatory Guide 1.59 (Reference 2.1-3), and ANSI/ANS-2.8-1992 (Reference 2.1-4) are the basis for the approach and methodology used in this reevaluation. The analysis is performed assuming underground storm drains and culverts, as well as roof drains are clogged and not functioning during the local PMP storm event.

The local PMP flood hazards for the CCNPP Units 1 & 2 are determined by performing site-specific hydrologic and hydraulic analyses. Natural Resources Conservation Service (NRCS) methods simulated in the USACE computer program HEC-HMS (Reference 2.1-5) are used to determine runoff hydrographs and peak discharges along identified flow paths. Water surface elevations and flow velocities are determined using the USACE computer program HEC-RAS (Reference 2.1-6).

The evaluation considers both the present day (i.e. existing) condition and the effect of future conditions, in particular, when the future Unit 3 (References 2.1-7) is in place.

2.1.1 Site Description

As described in Section 1.1, CCNPP Units 1 & 2 is located on the eastern side of the Calvert Peninsula in Calvert County, Maryland, and on the west shore of the Chesapeake Bay, approximately 110 mi north from the mouth as shown in Figure 2.1-1. The vicinity of the site is characterized by densely wooded, low, flat to gently rolling terrain of low to moderate relief. Ground elevations at the site range from sea level to about 130 ft NGVD 29, with an average elevation of approximately 100 ft NGVD 29. Nearly vertical cliffs, over 100 ft high in places, are located along the shore of the Chesapeake Bay. The plant is located in an area near the east edge of the site where the preexisting ground elevation was about +65 ft NGVD 29.

According to UFSAR Sections 1.2.2, 2.5.2 and 2.8.3.6 (Reference 2.1-1), the Turbine Building for CCNPP Units 1 & 2 is oriented parallel and adjacent to the shoreline of the Chesapeake Bay with the twin Containment Buildings and the Auxiliary Building located on the west, or landward, side of the Turbine Building, where the final grade elevation of the plant is about 45 ft NGVD 29. The Diesel Generator Buildings, consisting of the EDG and the SBO Buildings, are also on the west of the Turbine Building and has a floor grade of 45.5 ft NGVD 29. The service building and the intake and discharge structures are on the east, or bay side, of the Turbine Building. The intake structure has an open deck at elevation 10 ft NGVD 29 on the bay side. The deck is about 50 ft wide and has openings for the trash rakes and racks, stop logs, and traveling screens. Behind the open deck is an enclosure housing for the circulating water pumps and the safety-related saltwater cooling pumps. The roof of the pump room is at Elevation 28.5 ft NGVD 29 and has watertight hatches to provide access to the pumps for maintenance.

The ground of CCNPP Units 1 & 2 plant is primarily concrete paved, with some isolated areas that are covered with gravel or short grass. The general terrain and the predominant directions of storm water flow are depicted in Figure 2.1-2. The proposed layout of the future Unit 3, as documented in the CCNPP Unit 3 COLA (Reference 2.1-7), is also outlined in Figure 2.1-2. The contributing drainage areas to the plant as delineated from topographic data are provided in Figure 2.1-3. As shown in both figures, the ground surface level gradually increases from the intake area on the bay towards the southwest direction, continuing past the Units 1 & 2 power block until it reaches a topographical ridge line approximately 2,500 ft from the shore. The storm water drainage boundary on the northwest side is defined by a continuous row of vehicle barriers placed primarily along a local ridge line. The vehicle barriers on the southwestern side bordering the substation are not considered to be part of the drainage boundary as they are not sitting on topographic high points, a conservative assumption that would maximize PMP runoff to the plant.

The vehicle barriers are either concrete blocks or earth filled concrete walls at least 2.5 ft high and 3 ft wide. Concrete blocks include a low, short opening at the ground level. Narrow openings are present at some locations for personnel access. Along the northwestern boundary of the sub-basin Sub-2 the vehicle barriers are placed along the topographical drainage divide. For other sub-basins, upslope areas are conservatively included in sub-basin delineation, although runoff from upslope areas would likely be hindered by the barriers.

As shown in Figures 2.1-2 and 2.1-3, the total area that drains toward the plant and subsequently to the Chesapeake Bay is about 149.9 acres (0.2342 sq mi), but the contributing drainage area to the Units 1 & 2 power block is only 67.6 acres (0.1058 sq mi). The storm water runoff from the remaining 82.3 acres (0.1284 sq mi) will drain towards the proposed Haul Road for Unit 3 and will eventually discharge to the bay without affecting the Units 1 & 2 facilities.

The proposed Unit 3 will be sited approximately 2,500 ft south-southeast of Units 1 & 2 as shown in Figure 2.1-2. A local PMP drainage evaluation for the future unit has been conducted in support of the CCNPP Unit 3 COLA (Reference 2.1-7, FSAR Section 2.4.2). The COLA analysis shows that the PMP storm water runoff from the Unit 3 site will drain to the Chesapeake Bay via the proposed Haul Road that runs between Units 1 & 2 and Unit 3. There will be no direct runoff from Unit 3 discharging to the Units 1 & 2 area. In addition, because the Haul Road is much lower in elevation, Units 1 & 2 will not be impacted by the backwater effect from the PMP drainage of Unit 3. Other than the proposed Unit 3, no other future or planned change to the plant or the contributing watershed that could have a hydrologic impact on the flooding of CCNPP Units 1 & 2 has been identified. Therefore, a specific local PMP assessment pertaining to a future condition is not evaluated as the impact will be bounded by the hydrologic configuration considered for the existing condition.

2.1.2 Probable Maximum Precipitation

The basis for the local intense precipitation evaluation is the PMP event. The PMP depths for portions of the United States located east of the 105th meridian, where the CCNPP site is located, are obtained from Hydro-Meteorological Report (HMR) 51 (HMR 51) and HMR 52 (References 2.1-8 & 2.1-9) published by NOAA. The estimated depths from HMR 51 are for precipitation durations ranging from 6 to 72 hours and for drainage areas from 10 to 20,000 sq mi. HMR 52 also provides procedures for estimating short duration, point (1 sq mi) PMP depths that reflect quick response of the small drainage area. Table 2.1-1 presents the PMP depths for various durations at the CCNPP Units 1 & 2 site. The PMP depths used in this evaluation are consistent with the PMP depths applied for the FSAR Section 2.4 of CCNPP Unit 3 COLA (Reference 2.1-6). An examination of official rainfall records for Maryland and Virginia since the publication of the HMRs 51 and 52 has not identified any events with precipitation depths of magnitudes approaching the PMP depths listed in Table 2.1-1. Thus, the use of the PMP values from the current HMRs is considered applicable for the CCNPP Units 1 & 2 site.

The 5-, 30-, 60-, 120-, 180-, and 360-minute duration depths are input into the Frequency Storm option of the meteorological model in HEC-HMS simulation to develop the 6-hour storm distribution that conservatively and fully covers the PMP storm event. HEC-HMS uses the depths provided to distribute rainfall depths in equal time increments on either side of the peak depth. The 2 hour and 3 hour duration PMP depths, required as an input to define the PMP storm in the model, are developed by curve fitting of the PMP depths obtained from HMR 51 and HMR 52. The interpolated PMP depths for 2-hour and 3-hour durations are 21.82 in and 23.96 in, respectively, as shown in Figure 2.1-4.

The hydrologic and hydraulic analyses are based on the assumption that the storm drainage culverts and small ditches around the CCNPP Units 1 & 2 site are not functioning under the PMP storm event, and therefore, only overland flow is considered. All the runoff from roofs is included as direct runoff from the sub-basin drainage areas.

2.1.3 Hydrologic Modeling

The drainage area contributing to the CCNPP Units 1 & 2 power block area, as shown in Figure 2.1-3, is determined using the topographic data collected in support of the Unit 3 COLA (Reference 2.1-7). Stormwater runoff from sub-basins Sub-1, Sub-2, and Sub-3 is collected near the southern corner of the Units 1 & 2 power block at Junction J-2 of the HEC-HMS model. Junction J-2 is directly connected to junction Diversion from which the flow is diverted and divided into two downstream reaches around the power block. A portion of the flow (designated as the direct flow in the HEC-HMS model) from junction Diversion goes to reach Downstream-1, which flows along the southeastern side of the power block toward Outlet-1, where it is joined by the runoff collected from sub-basin Sub-5. The remaining flow (designated as the diverted flow in the model) from junction Diversion goes to reach Downstream-2, which flows along the southwestern and northwestern sides of the power block towards junction Outlet-2, with sub-basins Sub-4 and Sub-6 draining to junctions J-3 and Outlet-2, respectively, as shown on the sub-basin drainage area map (Figure 2.1-3) and the HEC-HMS model schematic (Figure 2.1-5).

The HEC-HMS computer model was set up to simulate the PMP storm event with the diversion junction element and to estimate the split flows into the two downstream reaches: Downstream-1 and Downstream-2. The split flows are obtained based on momentum balance at the bifurcation in a HEC-RAS model setup to perform the iterations until the water surface elevations at the upstream ends of the two reaches reached the same level. The rating curve of inflow vs. diverted flow at the bifurcation, generated in the HEC-RAS model for a series of inflow values, is used to establish the initial flows in the two downstream reaches in the HEC-HMS model.

The methodologies suggested by the U.S. Natural Resources Conservation Service (NRCS) as given in TR-55 Manual (Reference 2.1-10) are used to estimate the times of concentration (Tc) for the various sub-basins. The Tc flow path is divided into three segments: sheet flow, shallow concentrated flow, and open channel flow which is not applicable in this evaluation since there is no well-defined open channel in the power block. The selected surface roughness coefficients of 0.15, 0.24, and 0.40, corresponding to the surface cover conditions of short grass, dense grass, and light woods for various part of the site, are used, based on recommendations in Table 3-1 of Reference 2.1-10. The shallow concentrated flow is assumed for the rest of the flow paths and the time of travel is estimated using the velocity equations of $V=16.1345(\text{slope})^{0.5}$ for an unpaved surface condition and $V=20.3282(\text{slope})^{0.5}$ for a paved surface condition (Reference 2.1-10).

To account for non-linearity effects during extreme flood condition, the computed Tc was reduced by 25 percent in accordance with USACE guidance from EM-1110-2-1417 (Reference 2.1-11). The estimation of the Tc values including sheet flow and shallow concentrated flow parameters for all the sub-basins is summarized in Table 2.1-2. Sub-basin lag time, which is estimated as 60 percent of Tc (Reference 2.1-10) is an input to the HEC-HMS model along with the sub-basin drainage areas and the local PMP intensities with a 6 hour storm duration in 5-minute time increment. ANSI/ANS-2.8-1992 (Reference 2.1-4) requires that prior to the PMP event, an event equivalent to the 40 percent PMP has occurred, with 3 to 5 dry days between the events, which will render the ground saturated. To simulate saturated ground conditions, all areas are conservatively assumed impervious. Therefore, a runoff curve number of 98, representing the impervious surface (Reference 2.1-10) regardless of soil types and accounting for no infiltration during the PMP storm event, is conservatively used for the entire drainage area. The NRCS dimensionless unit hydrograph option is utilized for the development of the peak discharges from the various sub-basins in the HEC-HMS model (Reference 2.1-5).

A schematic of the HEC-HMS model setup is shown in Figure 2.1-5, and the resulting peak discharges during the local PMP event are presented in Table 2.1-3. The hydrographs of each sub-basin, junction, and outlet are shown in Figure 2.1-6a through Figure 2.1-6l.

2.1.4 Hydraulic Modeling

The computer program HEC-RAS developed by the USACE (Reference 2.1-6) is used to simulate the water levels in the CCNPP Units 1 & 2 power block area. As described earlier, the model reaches are set up around the power blocks after the flow is separated into two streams at Junction Diversion. The reaches modeled in HEC-RAS include one upstream reach flowing into Junction Diversion, two downstream reaches coming out of Junction Diversion; one drains to Outlet-1 and the other one drains to Outlet-2.

Cross-sections in HEC-RAS model are generated around the CCNPP Units 1 & 2 power block and are selected generally perpendicular to the predominating flow direction along the flow paths as shown in Figures 2.1-7a and 2.1-7b. The cross sections include the flow blocking effects of structures which are modeled as obstructions, and the effects of ineffective areas that restrict the flow conveyance areas. The interval of the cross sections is less than 100 ft.

The peak discharges given in Table 2.1-3 is simulated as the flow input in the HEC-RAS model. The flow rates are prorated along the reach lengths at each successive cross section, to better represent the flow distribution along each flow path.

The junction is modeled using momentum balance computation method in the HEC-RAS model. In this option, HEC-RAS uses momentum equations to determine the junction losses. The intersection angles of 0 degree to Downstream-1 and 90 degree to Downstream- 2 are used respectively in the junction model to indicate the angles at which the reach Upstream branches off into the two downstream reaches. In addition, both friction forces and the weight of the water are considered in determining the junction losses. A Manning's "n" value of 0.020 is assumed for the channel and over bank areas representing roughly covered impervious pavement of the site (Reference 2.1-12).

Lateral weirs are used to simulate the overflows leaving the reaches of Downstream-1 and Downstream-2. Lateral Weir 1 is located between the cross sections 278 and 131 along the left ends (looking downstream) on Downstream-1. Lateral Weir 2 is located between the cross sections 1075 and 555 along the right ends (looking downstream) on Downstream-2.

The junction boundary condition was used at the junction Diversion. Normal depth is used as the upstream boundary condition for Reach Upstream, and critical depth condition is used as the downstream boundary for the Downstream-1 and Downstream-2. Simulations in the HEC-RAS model are performed in the steady-state condition for a mixed flow regime. Figure 2.1-8 shows the schematic of the HEC-RAS model.

The maximum water levels estimated from the HEC-RAS model for the PMP storm event are shown on Table 2.1-4. The maximum water surface profiles along Downstream-1 and Downstream-2 are shown in Figure 2.1-9. Figure 2.1-10 shows the maximum PMP water levels for each cross section along with the approximate locations of openings/entrances to the safety-related structures or structures that contain safety-related equipment or equipment important to safety. Rating curves representing the relationship of discharges vs. water levels at selected cross sections are developed to estimate the flooding durations in conjunction with the corresponding hydrographs.

The flooding duration is defined as the time period when the water level is above the floor elevation at the entrance or opening of a specific structure. The rating curves of the model cross sections corresponding to these openings/entrances of the power block structures are presented in Figure 2.1-11a through Figure 2.1-11j.

2.1.5 Effect of Local PMP

The flood critical safety-related structures and structures that are important to safety in the CCNPP Units 1 & 2 plant that have entrances or openings potentially subject to flood over flow are: the Diesel Generator Buildings, the Auxiliary Building, the Turbine Building. The floor elevation of these structures is at 45 ft NGVD 29, with the exception of the Diesel Generator Buildings which has a floor elevation of 45.5 ft NGVD 29 (Reference 2.1-13). The floor elevations of the safety-related structures and structures important to safety, the corresponding HEC-RAS model cross sections, the maximum predicted PMP water levels, and the estimated flooding durations are shown in Table 2.1-5.

Based on the hydrographs developed from the HEC-HMS model, the peak flows occur in the middle of the 6 hour PMP storm duration. As shown in Figures 2.1-6a through 2.1-6l, flow rates increase rapidly within one and a half hours, and by approximately 800 cfs at the Junction J-3, for instance, two and a half hours after the PMP storm occurred.

The maximum PMP flood levels predicted by the HEC-RAS model vary from 47.0 ft to 45.0 ft NGVD 29 along Downstream-1 which is on the southeastern side of Unit 2 and from 47.0 ft. to 45.1 ft NGVD 29 along Downstream-2 which is on the southwestern and northwestern sides of Unit 1. The model results indicate that entrances of the Auxiliary Building and the Turbine Building will experience flood depths from 0.1 ft to about 2 ft during a local PMP event. The Diesel Generator Buildings will not be flooded as the floor elevation of 45.5 ft NGVD 29 is above the predicted flood elevation at the entrance location.

The flood duration is estimated for various locations in the power block area at the entrances/ openings to the flood critical structures, using the hydrographs from HEC-HMS (Figures 2.1-6a to 2.1-6l) and the rating curves (Figures 2.1-11a to 2.1-11j) of discharges versus water surface elevations. The floor elevations of majority of the structures in the power block, except for the Diesel Generator Buildings, are at 45 ft NGVD 29 (Reference 2.1-13). For instance, the flow rate corresponding to a water level of 45 ft NGVD 29 at Cross Section 1509 (Figure 2.1-11b) where one of the entrances of the Auxiliary Building is located, is about 160 cfs, above which flooding into the Auxiliary Building will occur. From the hydrograph in Figure 2.1-6k, the duration of flooding at Cross Section 1509 is estimated to be approximately one and a half hours (when flow rate is above 160 cfs).

The predicted flood flow velocities from the HEC-RAS model are shown in Table 2.1-4. For the Downstream-2 reach between Cross Sections 1799 and 583 that cover most of the power block area, the flow velocity (in the main channel) ranges from 0.8 ft/s and 8.8 ft/s. Similarly, for the Downstream-1 reach between Cross Sections 140 and 689, which cover the southeastern side of the power block, the flow velocity ranges from 1.2 ft/s to 10.0 ft/s. Since the power block is mostly concrete paved, there is no safety concern as a result of scouring. The contributing drainage area to the CCNPP Units 1 & 2 power block is generally gradual and is mostly covered with an impervious surface, with some woods areas. Therefore, associated risk of sediment and debris being brought to the site is relatively low under the PMP storm event. There is a relatively steep slope between the Units 1 & 2 power block and the substation. The slope is covered with riprap and protected by gabion baskets. Erosion is not expected to be a factor affecting the safety of the plant.

The PMP runoffs from sub-basins Sub-1, Sub-2, and Sub-3 first converged at the southern corner of the power block and then separated towards Downstream-1 and Downstream-2 (at the junction 'Diversion'). The bed slope upstream of the junction is relatively steep. The channel velocity in this reach is as high as 13.9 ft/s and a hydraulic jump is expected. The ground surface, however, is concrete paved so that there would be no adverse risk from erosion.

After leaving the power block, the PMP discharges from Downstream-1 and Downstream-2 flow overland to the Chesapeake Bay on the northwestern and southeastern sides of the intake structure (see Figure 2.1-7b). The intake structure where the safety-related saltwater cooling pumps are housed will be subjected to direct precipitation. There is no parapet or other installation on the roof that will impede the free draining of the precipitation collected on the roof over the sides of the structure to the deck. With an intake deck elevation of 10 ft NGVD 29 and a roof elevation at 28.5 ft NGVD 29, runoff from Downstream 1 and Downstream 2 would have no impact to the intake structure and there will be no risk to the safety functioning of the saltwater pumps as a result of the local PMP event.

2.1.6 Conclusions

The model results indicate that entrances to the Auxiliary Building and the Turbine Building will experience flood depths from 0.1 ft to about 2 ft during a local PMP event. The model generated maximum duration for this flooding is 90 minutes. Based on this the reevaluated local PMP flood elevations are above the CLB and are further evaluated in Section 3.1 and Chapter 4.

The Emergency and SBO Diesel Generator Buildings will not be flooded as the floor elevation of 45.5 ft NGVD 29 is above the predicted flood elevation at the entrance location.

With an intake deck elevation of 10 ft NGVD 29 and a roof elevation at 28.5 ft NGVD 29, peak flow rates from Downstream 1 and Downstream 2 would have no impact on the intake structure and there will be no risk to the safety functioning of the saltwater pumps as a result of the local PMP event.

2.1.7 References

2.1-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

2.1-2 U.S. Nuclear Regulatory Commission, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America*, NUREG/CR-7046, Office of Nuclear Regulatory Research, November 2011.

2.1-3 U.S. Nuclear Regulatory Commission, *Design Basis Floods for Nuclear Power Plants*, Regulatory Guide 1.59, Revision 2, Office of Standards and Development, August 1977.

2.1-4 American National Standards Institute/American Nuclear Society, *Determining Design Basis Flooding at Power Reactor Sites*, ANSI/ANS-2.8-1992, Nuclear Standard 2.8, 1992.

2.1-5 U.S. Army Corps of Engineers, *HEC-HMS Hydrologic Modeling System*, Version 3.5, Hydrologic Engineering Center, August 2010.

2.1-6 U.S. Army Corps of Engineers, *HEC-RAS River Analysis System*, Version 4.1.0, Hydrologic Engineering Center, March 2008.

2.1-7 Unistar Nuclear Services, LLC, *Calvert Cliffs Nuclear Power Plant Unit 3, Combined License Application*, Revision 8, March 2012.

2.1-8 National Oceanic and Atmospheric Administration, *Probable Maximum Precipitation Estimates, United States East of the 105th Meridian*, Hydro-meteorological Report No. 51, June 1978.

2.1-9 National Oceanic and Atmospheric Administration, *Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian*, Hydro-meteorological Report Number 52, August 1982.

2.1-10 U.S. Department of Agriculture, *Urban Hydrology for Small Watersheds*, Technical Release 55, Soil Conservation Service (now known as Natural Resources Conservation Service), June 1986.

2.1-11 U.S. Army Corps of Engineers, *Flood-Runoff Analysis*, EM 1110-2-1417, August 1994.

2.1-12 V. Chow, *Open-Channel Hydraulics*, 1959.

2.1-13 Constellation Energy Nuclear Group, LLC, *Flooding Walkdown Report*, Report No: SL-011462, Revision 0, November, 2012.

2.1-14 National Oceanic and Atmospheric Administration, NOAA Atlas 14, Volume 2, Version 3, Website http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_printpage.html?lat=38.4329&lon=-76.4418&data=depth&units=english&series=pds, Accessed: August 22, 2012.

Table 2.1-1 Local Intense PMP Depths at the CCNPP Site

Time (Minute)	PMP Depth (inch)
360	28.00
180	23.96
120	21.82
60	18.48
30	13.86
15	9.70
5	6.15

Table 2.1-2 Time of Concentration Calculations

Sub-Basin Name	Sub-1	Sub-2	Sub-3	Sub-4	Sub-5	Sub-6
Sheet Flow Characteristics						
Manning's roughness Coefficient	0.24	0.24	0.15	0.15	0.15	0.4
Two-Year 24-hour Rainfall (in) (Reference 2.1-4)	3.38	3.38	3.38	3.38	3.38	3.38
Flow Length (ft)	100	100	100	100	100	100
Elev. (Upstream) (ft, NGVD29)	145.1	124.8	71.3	53	63	113
Elev. (Downstream) (ft, NGVD29)	143.7	120.9	68.4	46.5	53.5	105.2
Ground Slope (ft/ft)	0.014	0.039	0.029	0.065	0.095	0.078
Sheet Flow Travel Time (hr.)	**0.267**	**0.177**	**0.137**	**0.099**	**0.085**	**0.202**
Shallow Concentrated Flow Characteristics						
Surface Description (Reference 2.1-11)	unpaved	unpaved	paved	paved	unpaved	unpaved
Flow Length (ft)	2400	1650	630	640	1432	650
Elev. (Upstream) (ft, NGVD29)	143.7	120.9	68.4	46.5	53.5	105.2
Elev. (Downstream) (ft, NGVD29)	73	59.9	49.4	43.3	41	30.2
Watercourse Slope (ft/ft)	0.0295	0.037	0.0302	0.005	0.0087	0.1154
Average Velocity (ft/s)	2.769	3.102	3.53	1.437	1.507	5.481
Shallow Concentrated Flow Travel Time (hr.)	**0.241**	**0.148**	**0.05**	**0.124**	**0.264**	**0.033**
Channel Flow Characteristics						
Channel Flow Travel Time (hr.)	NA	NA	NA	NA	NA	NA
Total Time of Concentration (hr.)	**0.508**	**0.325**	**0.187**	**0.223**	**0.349**	**0.235**
75 % of Tc (hr.)	**0.381**	**0.244**	**0.14**	**0.167**	**0.262**	**0.176**
75 % of Tc (min.)	**22.8**	**14.6**	**8.4**	**10**	**15.7**	**10.6**
Lag Time (min.)	**13.7**	**8.8**	**5.0**	**6.0**	**9.4**	**6.3**

Note: NA – Not Applicable

Table 2.1-3 PMP Peak Flow Rates for Sub-Basins

	Hydrologic Element	Drainage Area (sq mi)	Peak Discharge (cfs)	Time of Peak from Start of Storm (hr)
Sub-basins	Sub-1	0.0256	544.5	3:15
	Sub-2	0.0225	598.2	3:10
	Sub-3	0.0068	236	3:06
	Sub-4	0.0087	278.7	3:07
	Sub-5	0.0202	521.4	3:10
	Sub-6	0.0219	686.2	3:07
Reaches	R-1	0.0256	544.5	3:15
	R-2	0.0481	1094.3	3:11
	R-3	0.0087	928	3:08
Junctions	J-1	0.0481	1094.3	3:11
	J-2	0.0549	1240.9	3:10
	J-3	0.0087	928	3:08
	Diversion	0.0549	567.4	3:10
Outlets	Outlet-1	0.0751	1088.8	3:10
	Outlet-2	0.0306	1602	3:08

Table 2.1-4 HEC-RAS Output Table

Reach	River Station	Q Total	Invert Elevation	Water Surface Elevation (WSE)	Critical WSE	Energy Grade (EG) Elevation	EG Slope	Channel Velocity	Flow Area	Top Width	Froude #
		(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Upstream	130	1241	49.3	50.5	51.2	52.8	0.04397	13.9	103	12	2.3
	64	1241	46.4	48.3	48.8	50.0	0.02337	13.8	125	9.9	1.8
	20	1241	44.3	49.0	46.9	49.1	0.00041	3.3	510	2.4	0.3
Downstream-2	1,799	671	43.3	47.0	44.6	47.0	0.00007	1.5	539	1.2	0.1
	1,722	707	43.4	47.0	44.0	47.0	0.00002	0.8	912	0.8	0.1
	1,592	767	43.3	46.9	44.9	47.0	0.00013	2.0	451	1.7	0.2
	1,509	806	43.5	46.9	-	47.0	0.00009	1.6	576	1.4	0.2
	1,412	851	43.5	46.9	44.5	47.0	0.00008	1.5	566	1.5	0.1
	1,336	887	43.5	46.9	45.1	46.9	0.00026	2.7	365	2.4	0.3
	1,254	925	43.4	45.8	45.8	46.7	0.00450	8.8	126	7.4	1.0
	1,161	1000	43.2	45.3	44.6	45.4	0.00098	3.7	331	3	0.5
	1,103	1047	43.1	45.1	44.7	45.3	0.00163	4.9	270	3.9	0.6
	1,075	1069	43.0	45.1	44.5	45.3	0.00108	4.0	333	3.2	0.5
	1050	Lateral Weir									
	1,028	830	42.0	45.1	44.1	45.2	0.00027	2.4	432	1.9	0.3
	989	607	42.0	45.2	43.7	45.2	0.00013	1.8	440	1.4	0.2
	921	427	41.0	45.2	42.9	45.2	0.00003	1.1	645	0.7	0.1
	888	387	41.1	45.2	43.0	45.2	0.00002	1.2	627	0.6	0.1
	805	349	42.0	45.2	42.9	45.2	0.00003	0.8	587	0.6	0.1
	757	387	42.0	45.2	43.5	45.2	0.00008	1.4	455	0.8	0.1
	677	452	41.7	45.1	44.5	45.2	0.00034	3.0	307	1.5	0.3
	583	527	41.3	44.6	44.6	45.1	0.00212	7.3	120	4.4	0.7
	555	550	41.1	42.8	43.4	44.8	0.03113	14.6	58	9.4	2.0
	537	564	39.7	40.5	41.1	43.4	0.11294	16.7	42	13.4	3.4
	505	590	36.0	37.0	37.8	40.5	0.07027	19.4	44	13.5	3.5
	459	627	31.2	32.9	34.0	37.8	0.05008	22.1	42	15	3.1
	441	642	28.0	30.8	32.0	36.6	0.07329	23.4	37	17.3	2.9
	417	661	30.0	31.6	32.4	34.8	0.02941	17.0	53	12.6	2.4
	390	661	28.6	30.0	30.9	33.7	0.04869	19.0	45	14.8	3.0
Downstream-1	689	570	43.2	47.0	44.5	47.0	0.00005	1.2	552	1	0.1
	648	607	43.4	46.9	44.7	47.0	0.00012	1.9	362	1.7	0.2
	555	691	43.7	46.9	44.8	47.0	0.00009	1.6	543	1.3	0.2
	489	750	43.7	46.8	-	46.9	0.00063	4.0	271	2.8	0.4
	408	823	43.7	46.2	46.1	46.8	0.00312	7.6	147	5.6	0.9
	382	846	43.7	45.9	45.9	46.6	0.00402	8.0	130	6.5	0.9
	321	901	43.7	45.0	45.3	46	0.02716	10.0	117	7.7	1.5
	278	940	43.1	45.0	44.8	45.3	0.00396	5.6	228	4.1	0.7
	250	Lateral Weir									
	231	929	42.5	44.6	44.5	45	0.00508	6.6	184	5	0.8
	196	944	41.9	44.6	44.3	44.9	0.00288	5.8	210	4.5	0.7
	166	971	41.6	44.6	44.1	44.8	0.00207	5.2	235	4.1	0.6
	149	986	41.0	44.5	44.1	44.8	0.00177	5.5	241	4.1	0.5
	140	994	40.8	44.5	44.1	44.8	0.00176	5.8	233	4.3	0.5
	131	1002	40.7	44.2	44.2	44.8	0.00311	7.4	169	5.9	0.7
	123	1009	39.9	42.7	43.2	44.6	0.01229	12.5	94	10.7	1.4
	115	1017	36.9	39.4	40.6	44.1	0.04576	21.1	63	16.2	2.5
	109	1022	34.0	36.5	38.0	43.5	0.06779	25.5	54	18.9	3.1

Table 2.1-5 Comparison of Elevations of Building Entrances and Water Levels

Safety-Related Facility	Opening / Floor Elevation (ft)	Associated Cross-Section	Reach	PMP Peak Water Elevation (ft)	Freeboard (ft)	Duration of Flooding at Openings (hr)
South Service Building	45.0	489	Downstream-1	46.8	- 1.8	1.5
Turbine Building	45.0	382	Downstream-1	45.9	- 0.9	0.8
Auxiliary Building	45.0	1722	Downstream-2	47.0	- 2.0	1.5
Auxiliary Building	45.0	1509	Downstream-2	46.9	- 1.9	1.5
Auxiliary Building	45.0	1412	Downstream-2	46.9	- 1.9	1.3
Auxiliary Building	45.0	1336	Downstream-2	46.9	- 1.9	1.0
Turbine Building	45.0	1075	Downstream-2	45.1	- 0.1	0.3
Diesel Generator Buildings	45.5	1075	Downstream-2	45.1	0.4	0.0

Figure 2.1-1 Site Location

Figure 2.1-2 General Site Terrain and Drainage Flow Directions

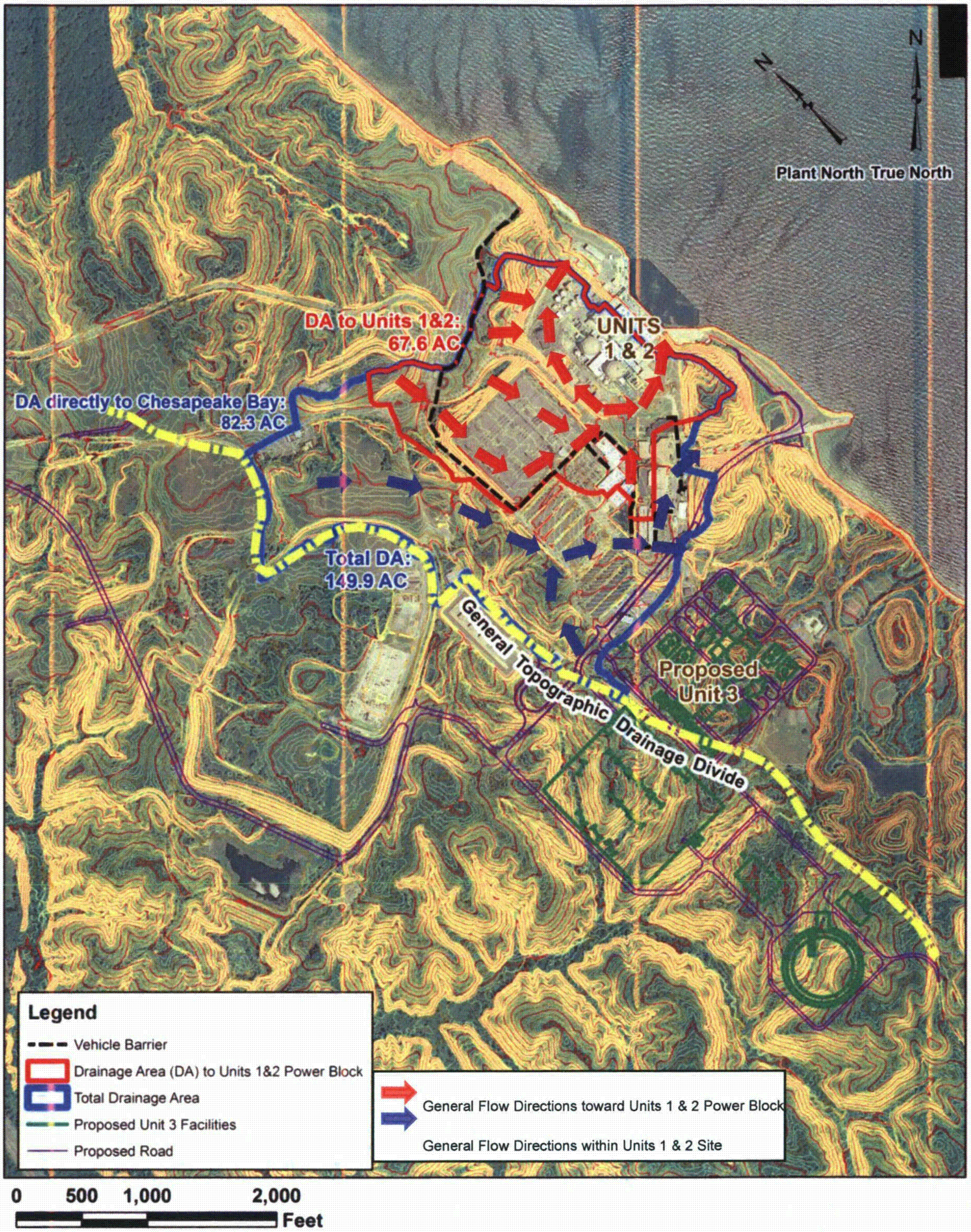
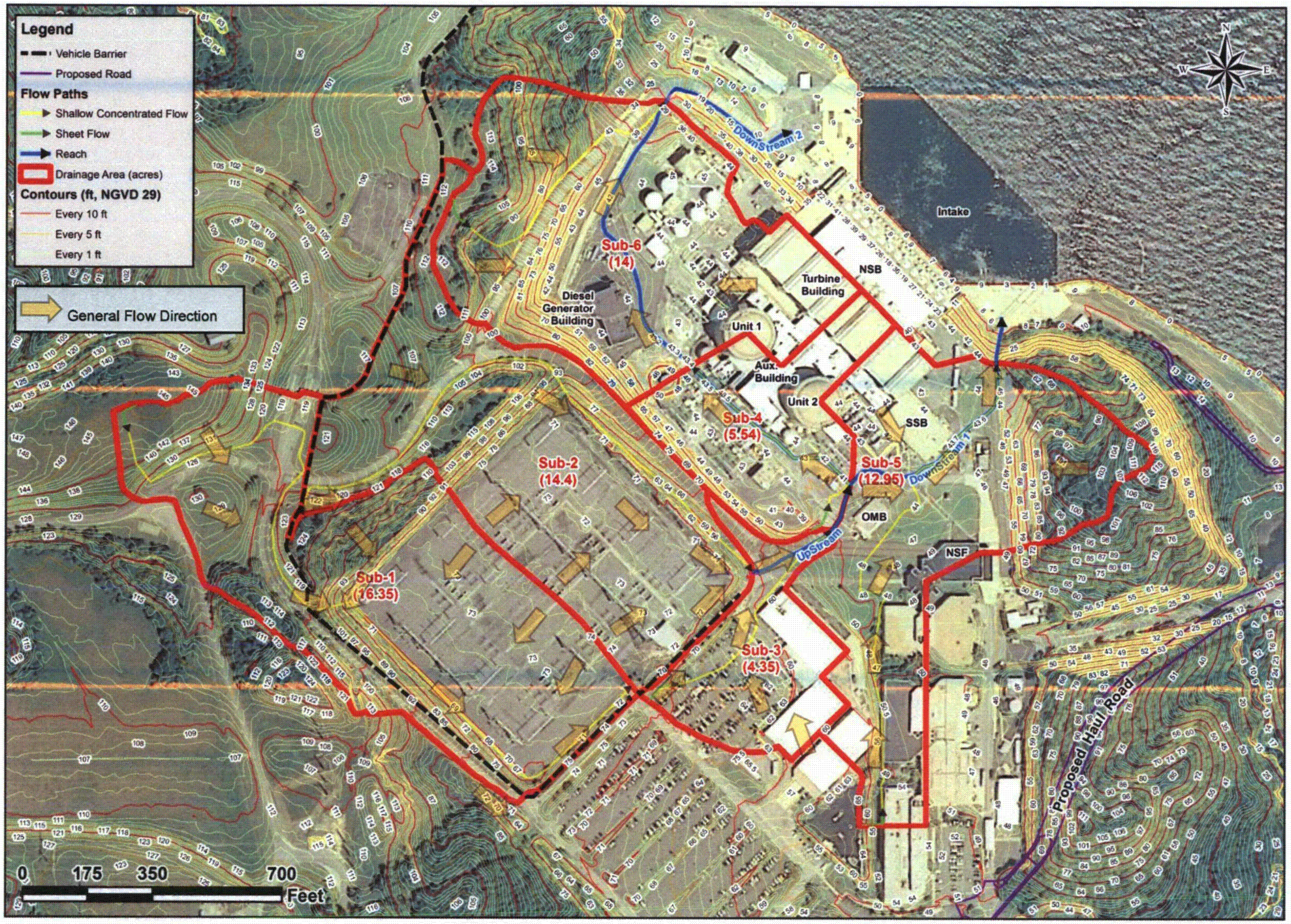


Figure 2.1-3 Sub-Basin Drainage Area Map



Note: Unit 1, Unit 2: Containment Buildings, NSB, SSB: North and South Service Buildings, OMB: Outage Management Building, NSF: Nuclear Security Facility

Figure 2.1-4 Fitting of PMP depths from HMR51 & HMR52

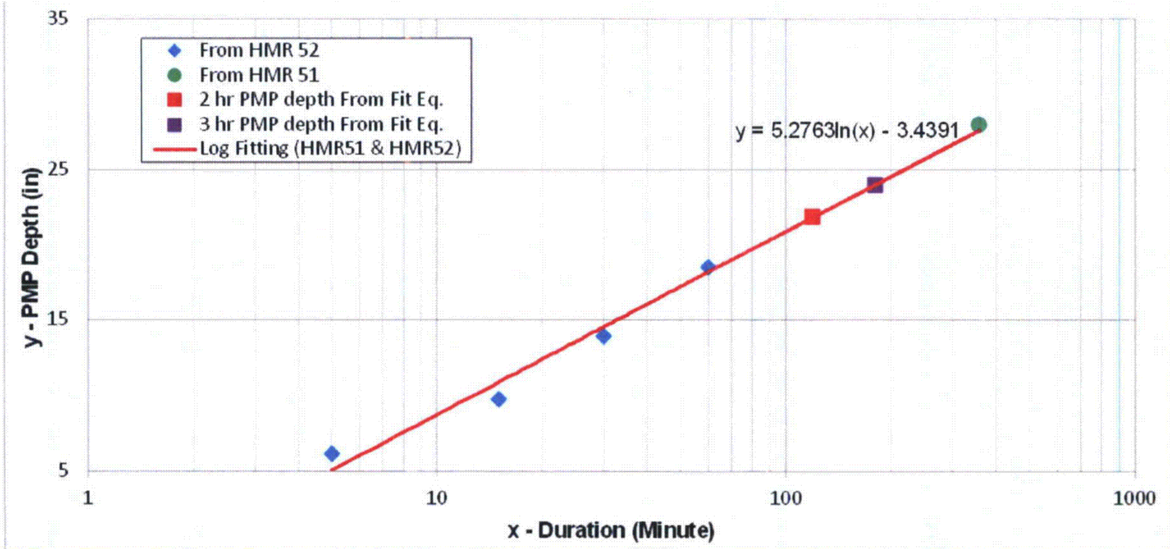


Figure 2.1-5 Schematic of HEC-HMS Model

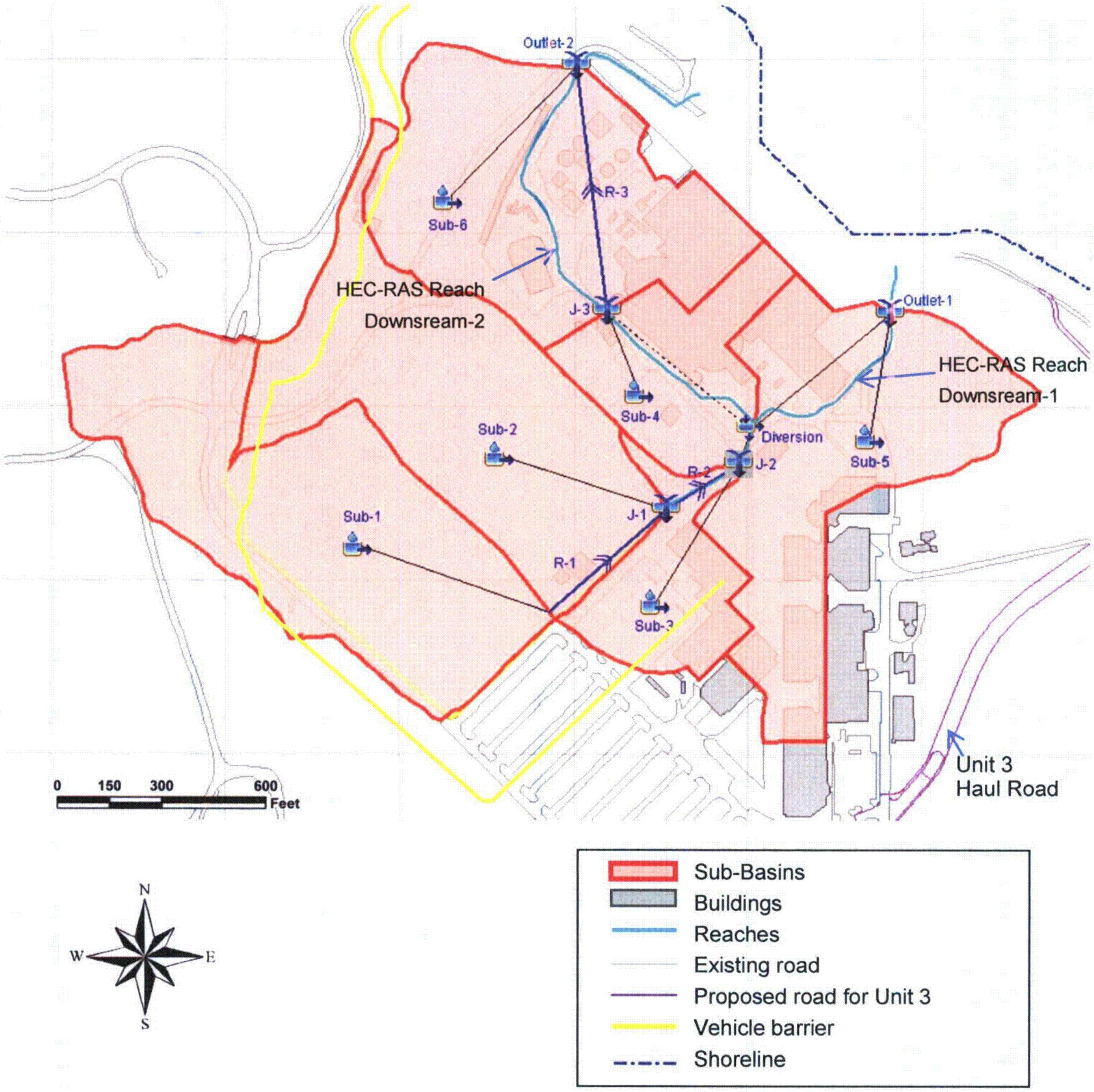


Figure 2.1-6a Hydrograph at Sub-1

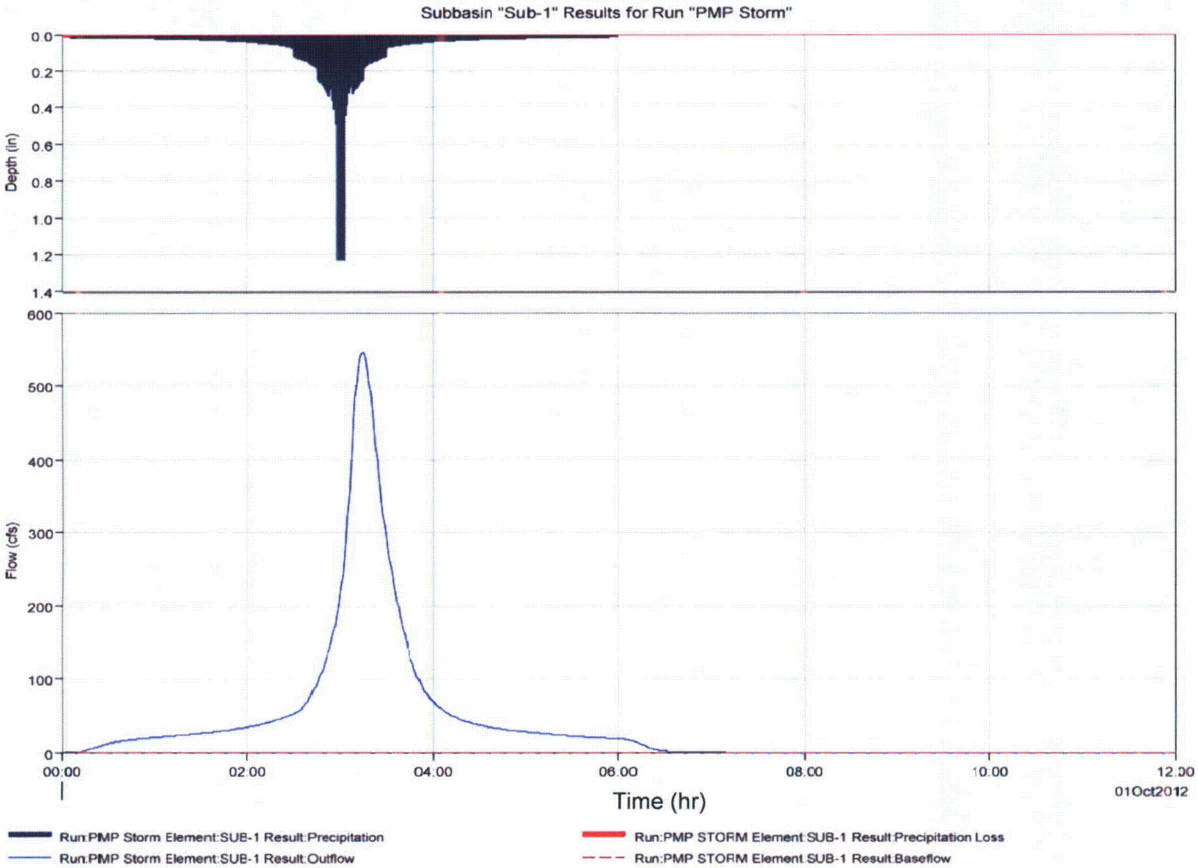


Figure 2.1-6b Hydrograph at Sub-2

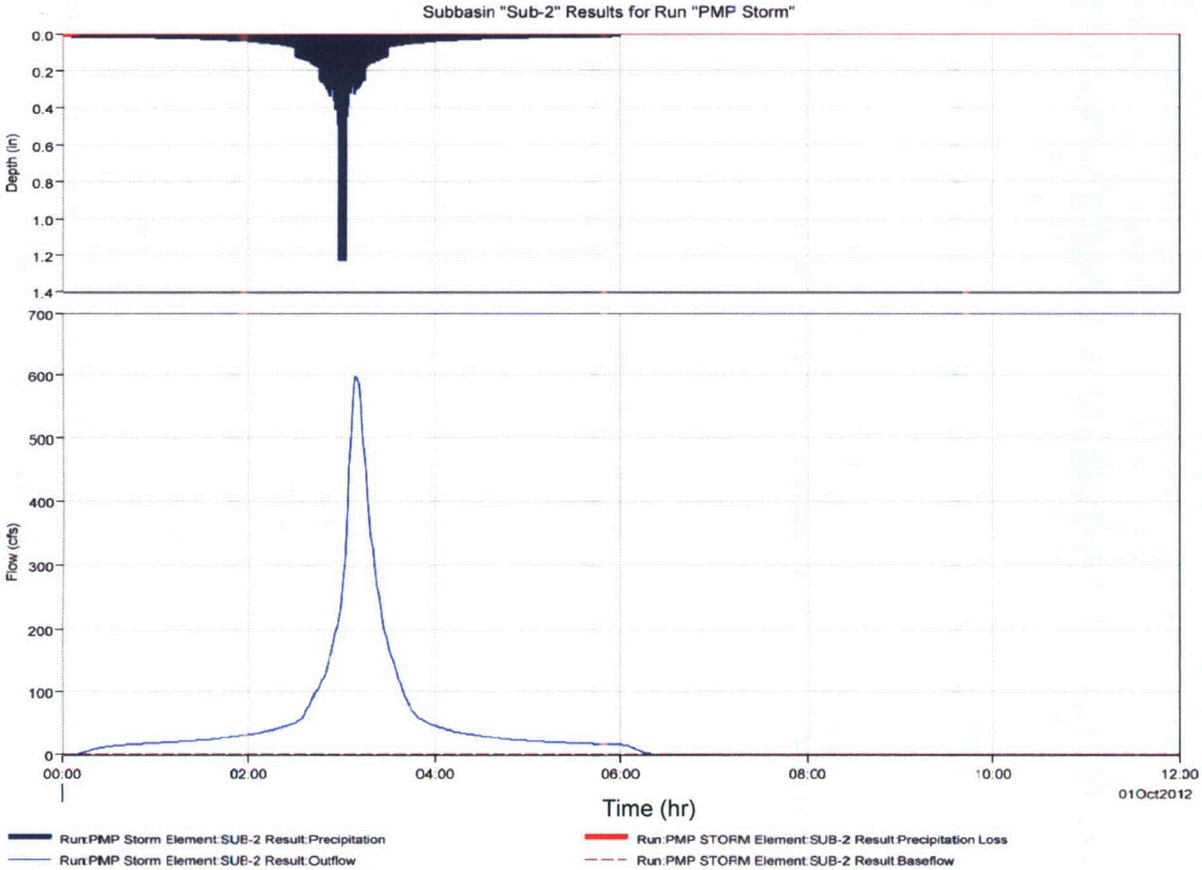


Figure 2.1-6c Hydrograph at Sub-3

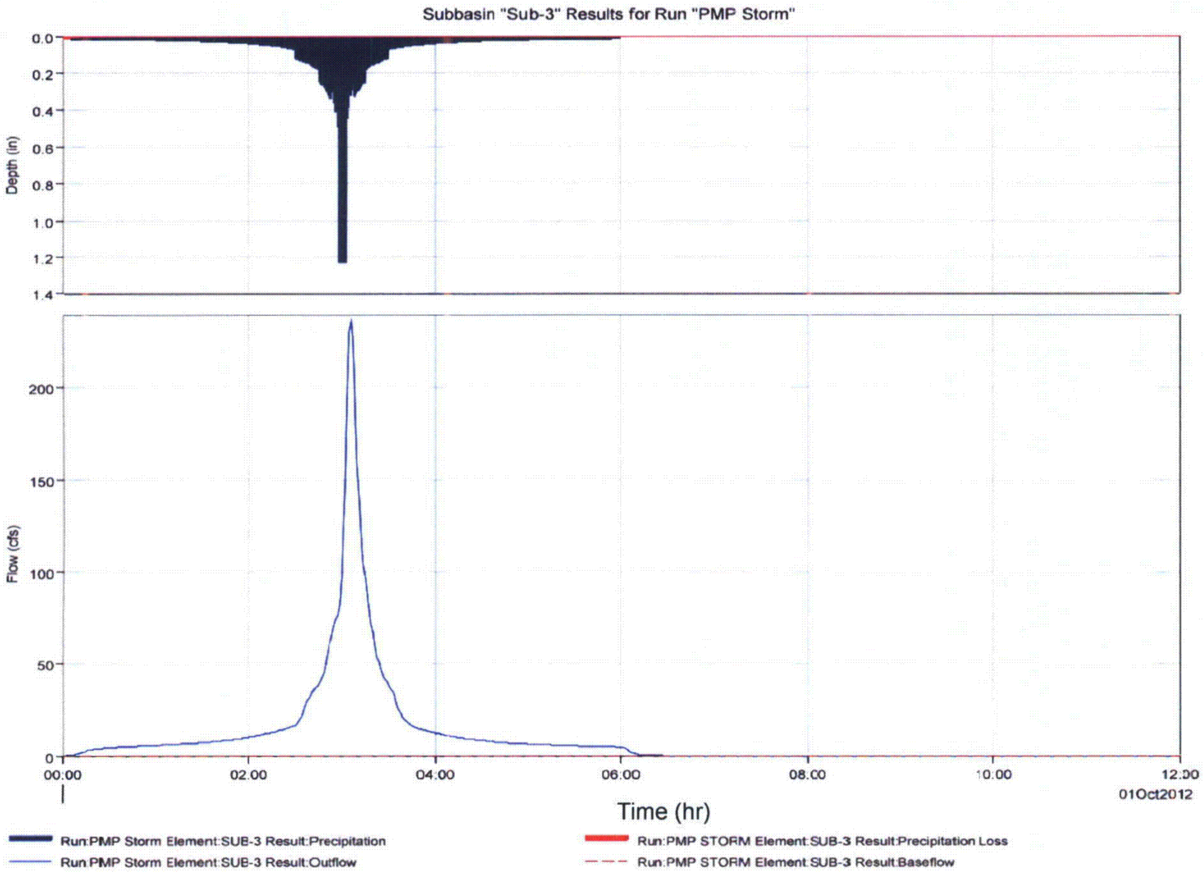


Figure 2.1-6d Hydrograph at Sub-4

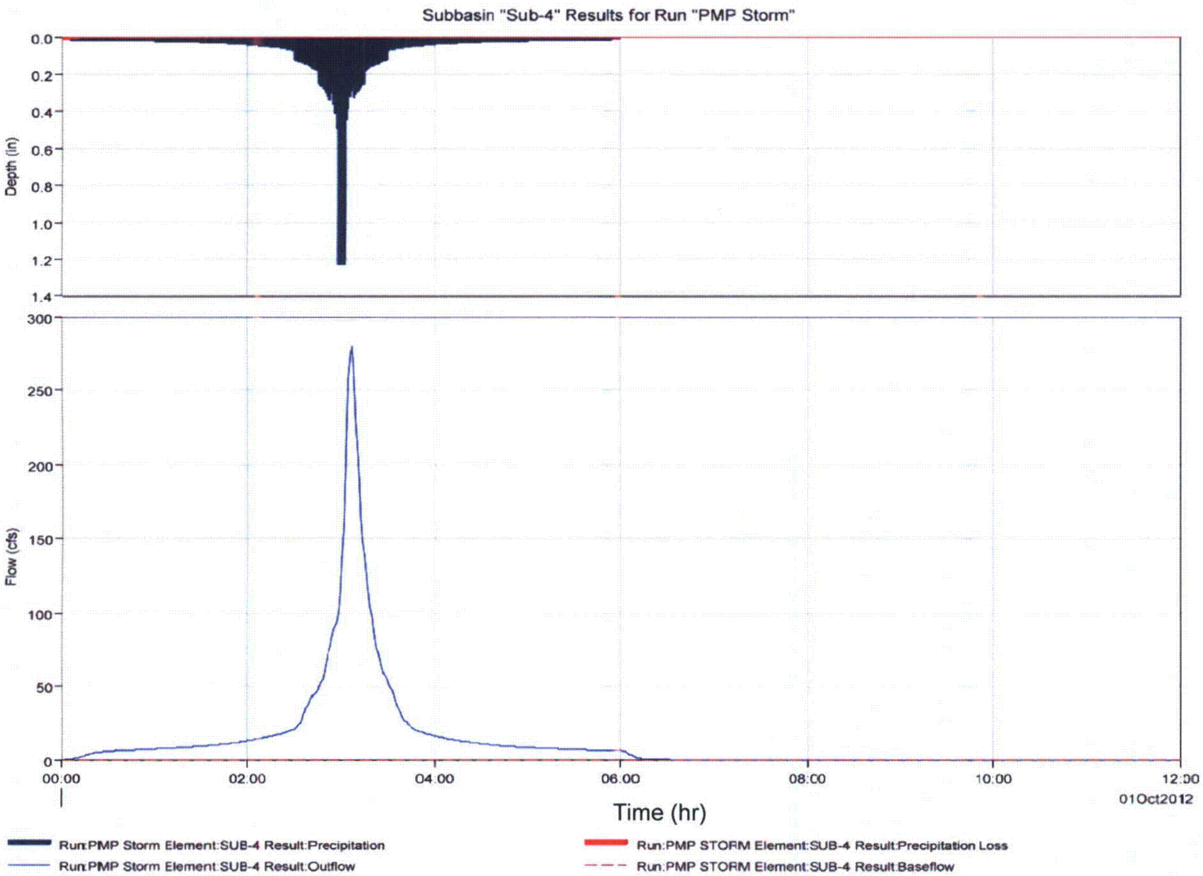


Figure 2.1-6e Hydrograph at Sub-5

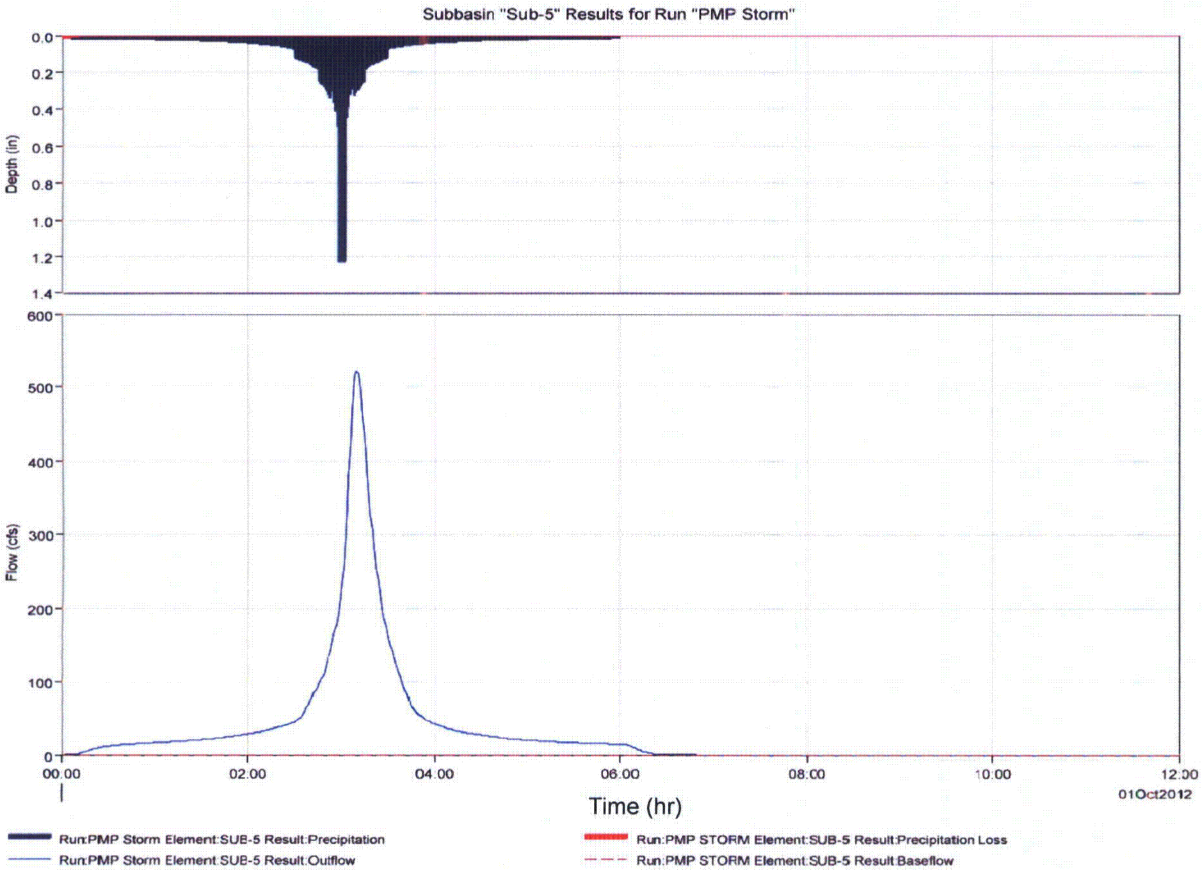


Figure 2.1-6f Hydrograph at Sub-6

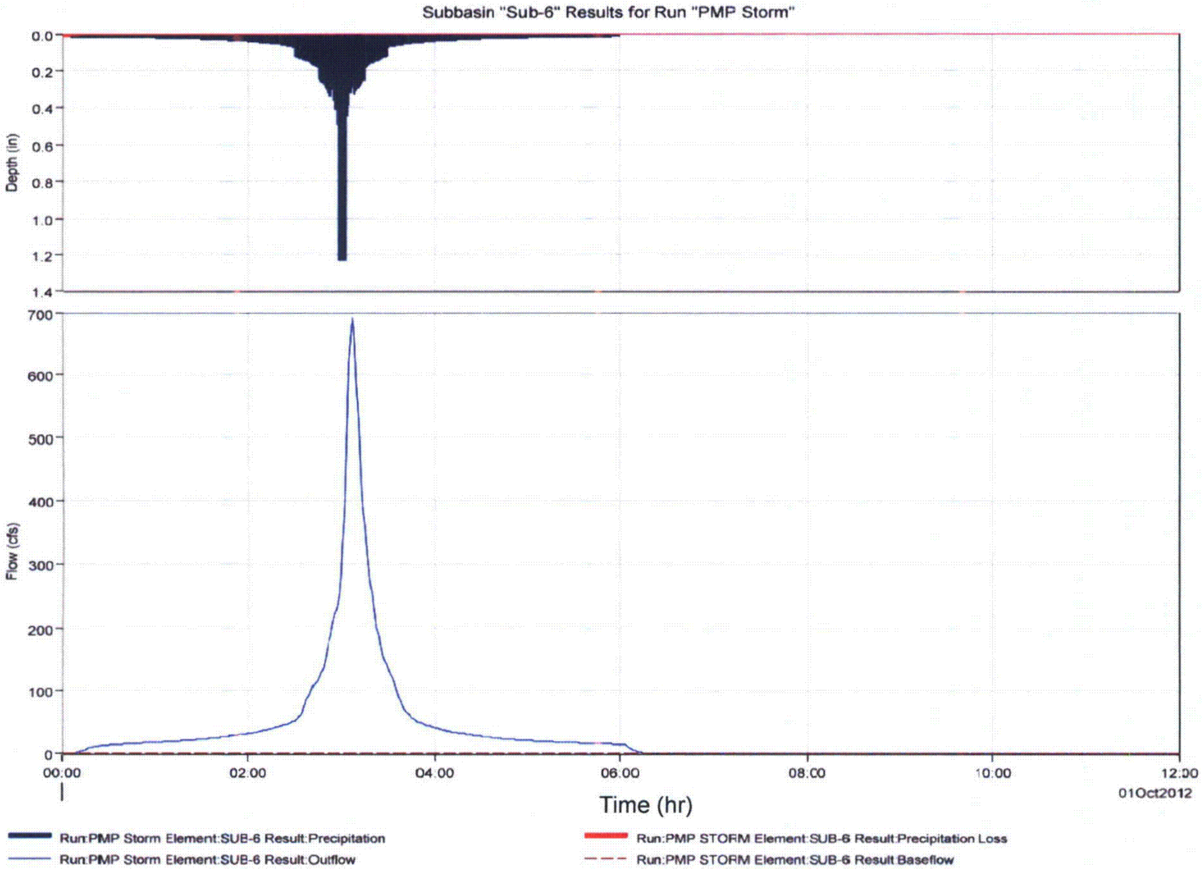


Figure 2.1-6g Hydrograph at Junction J-1

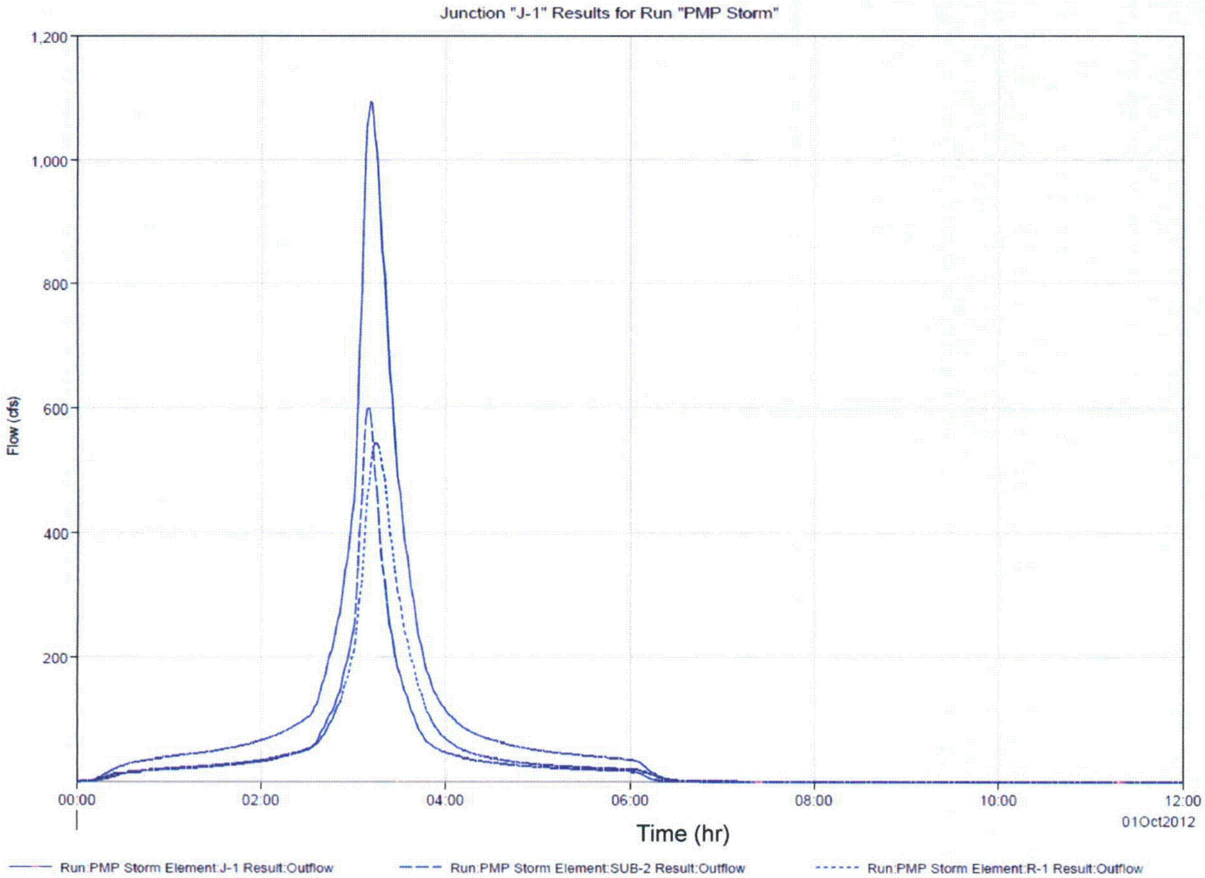


Figure 2.1-6h Hydrograph at Junction J-2

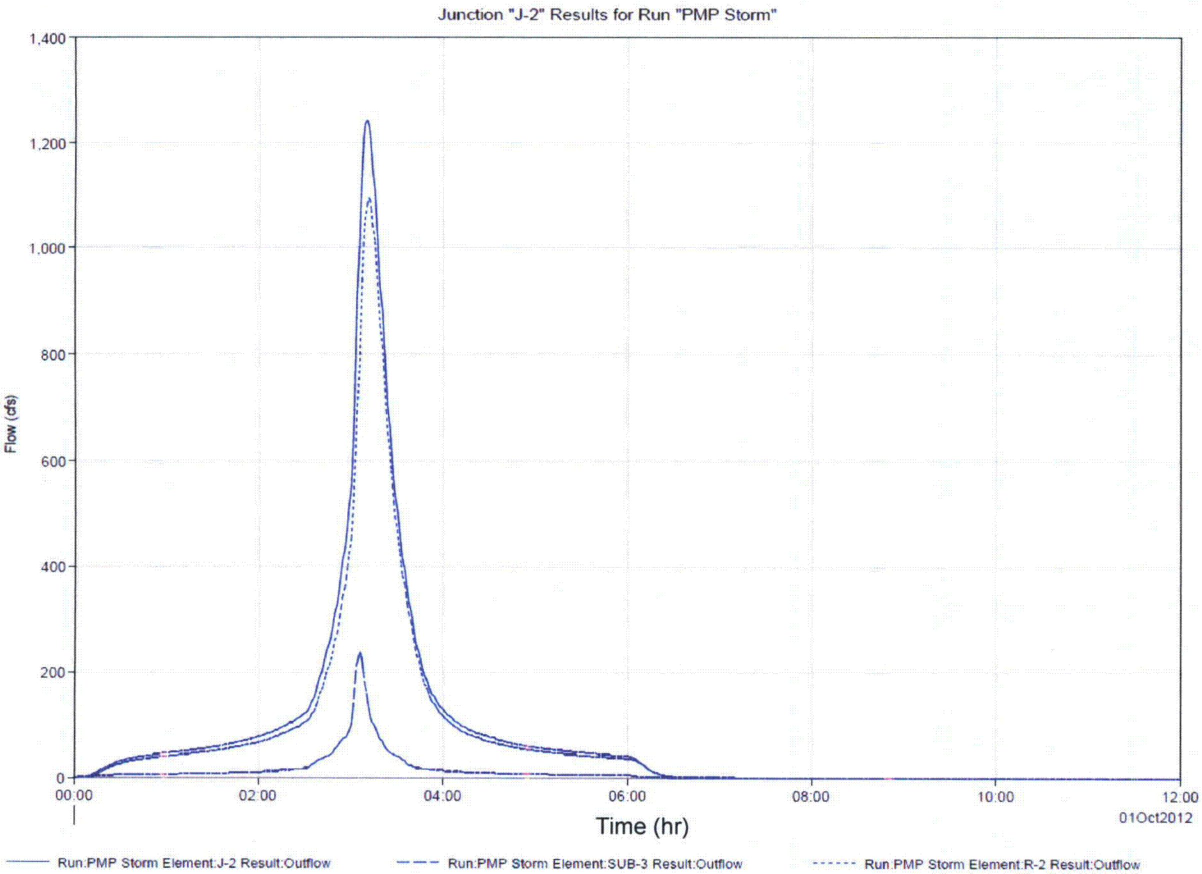


Figure 2.1-6i Hydrograph at Diversion

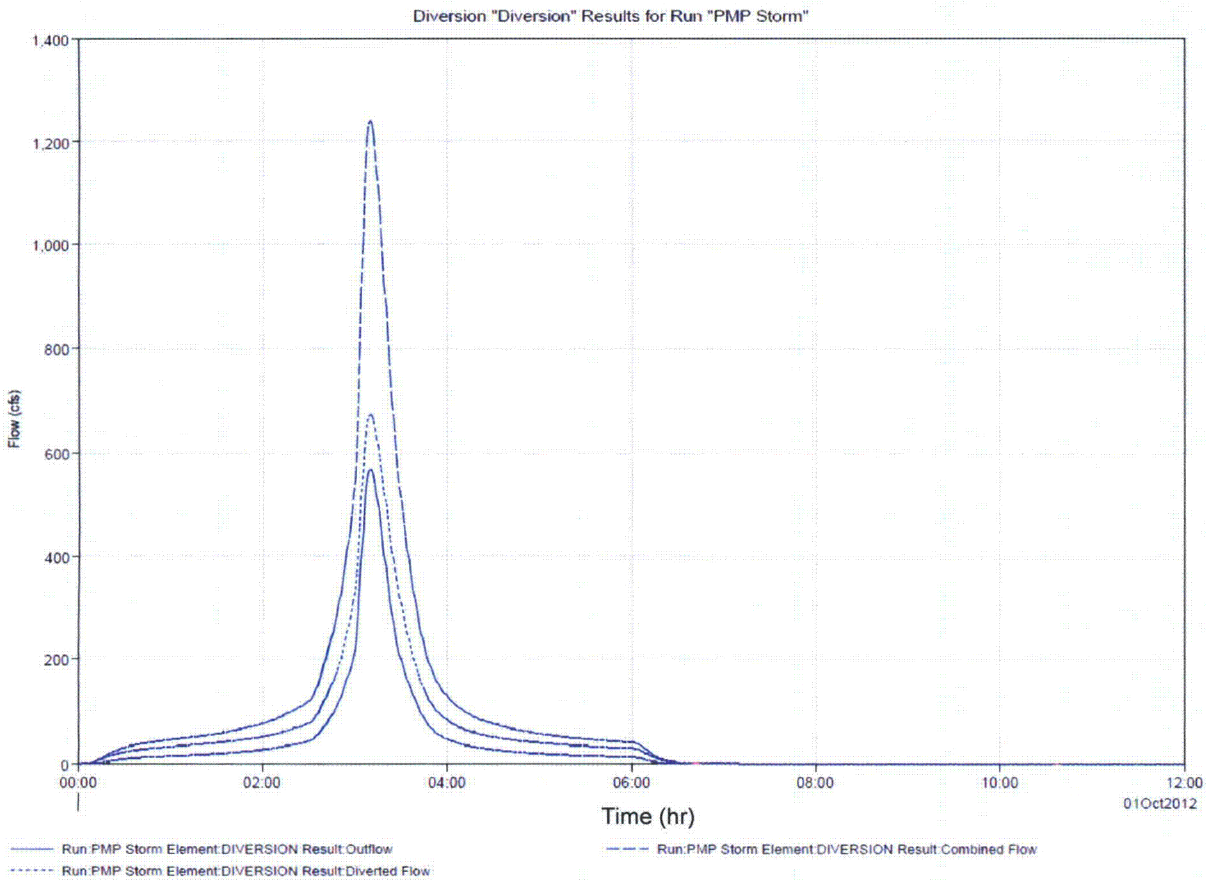


Figure 2.1-6j Hydrograph at Outlet-1

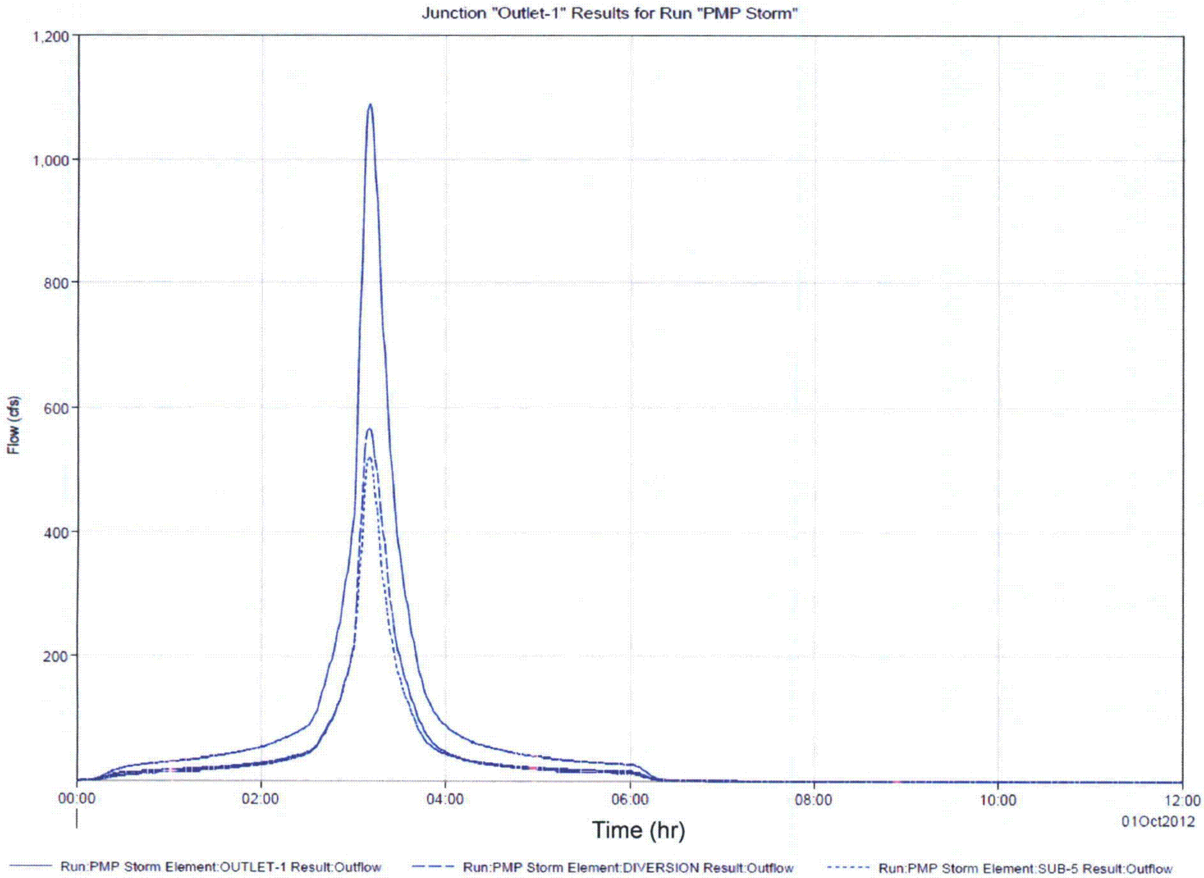


Figure 2.1-6k Hydrograph at Junction J-3

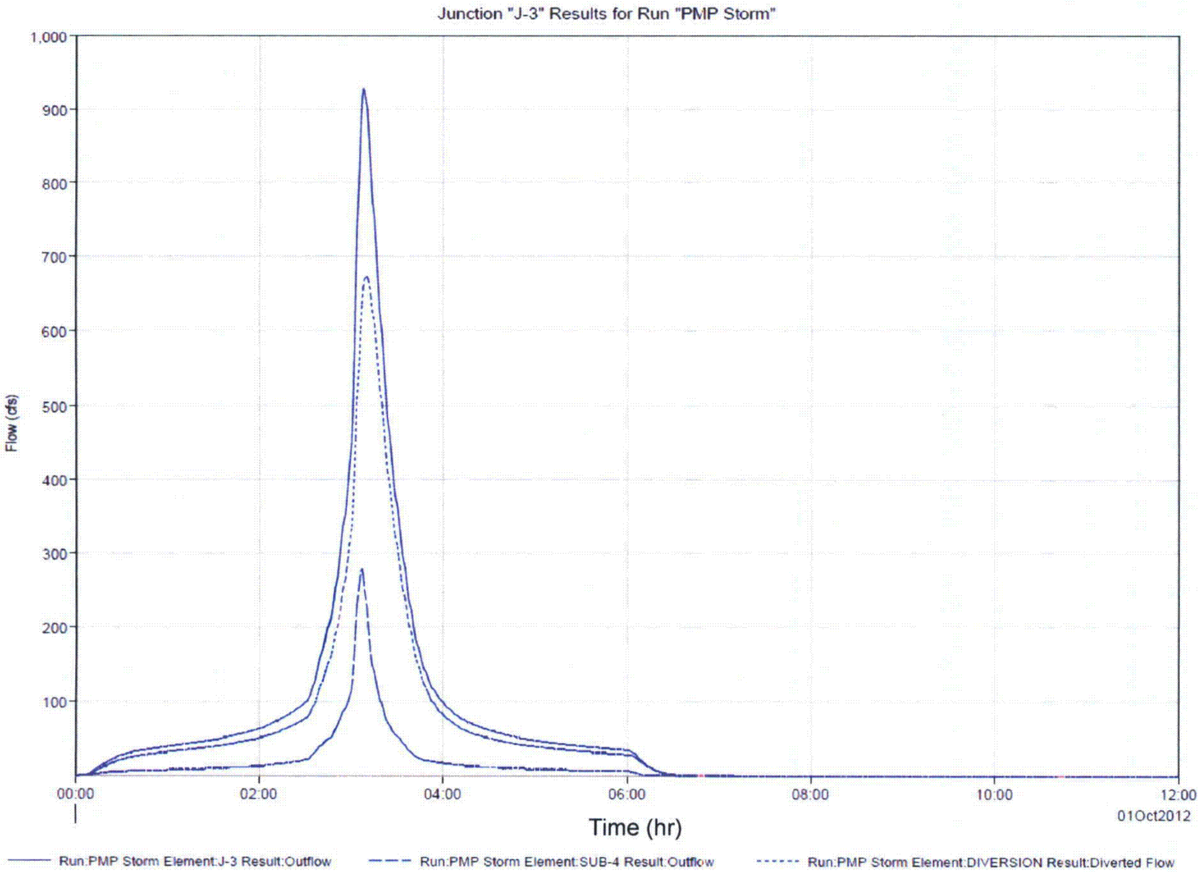


Figure 2.1-6l Hydrograph at Outlet-2

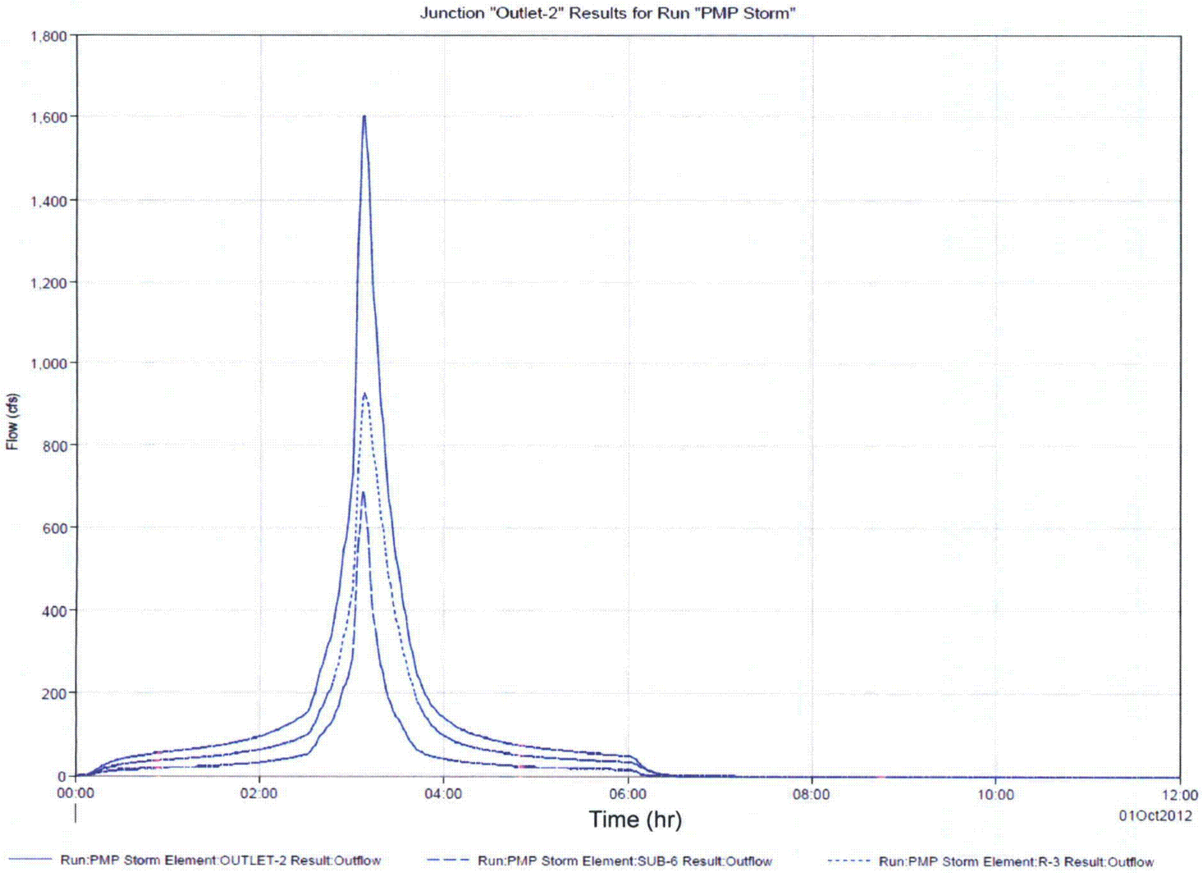


Figure 2.1-7a HEC-RAS Model Cross Section Plan

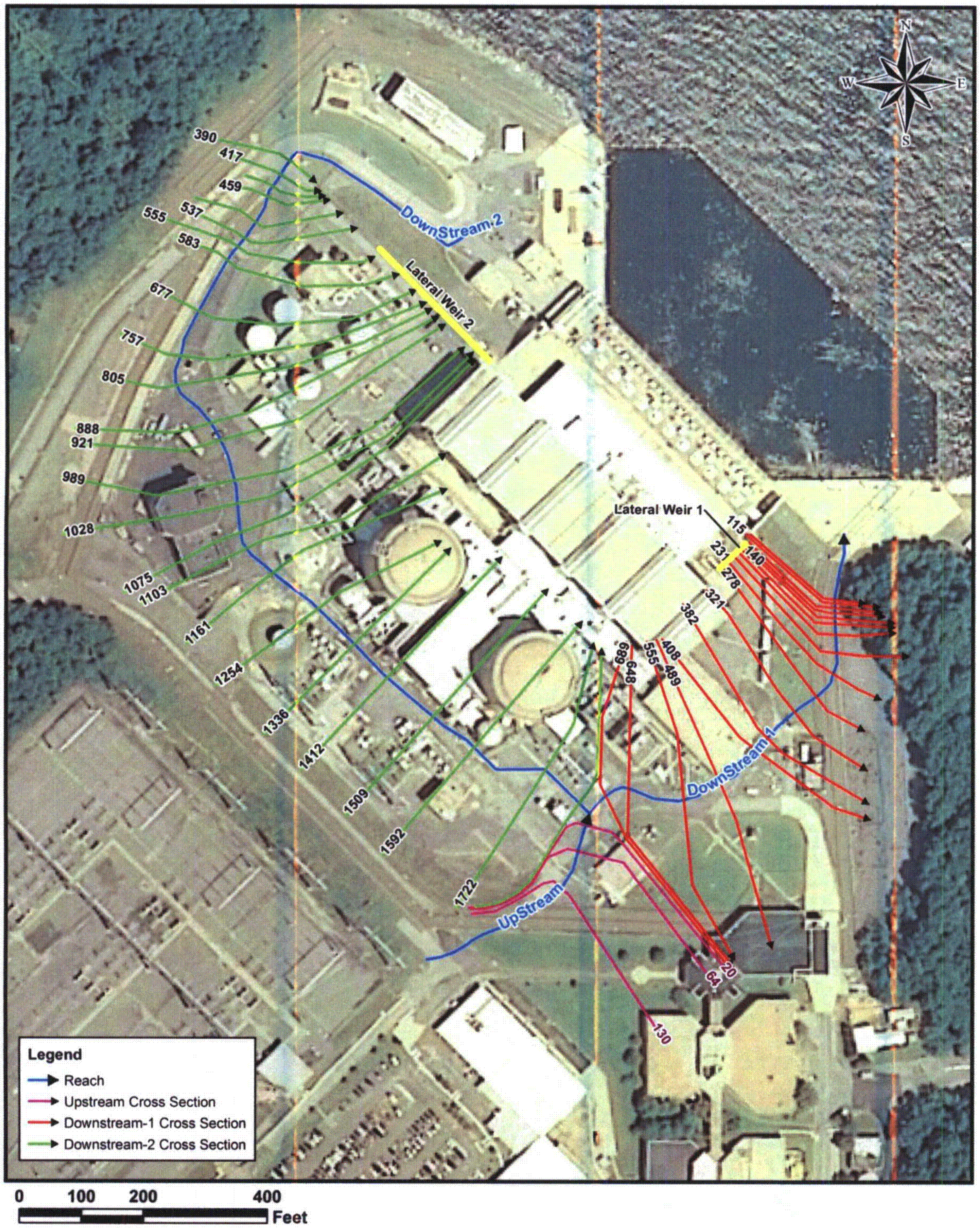


Figure 2.1-7b HEC-RAS Cross Section Plan

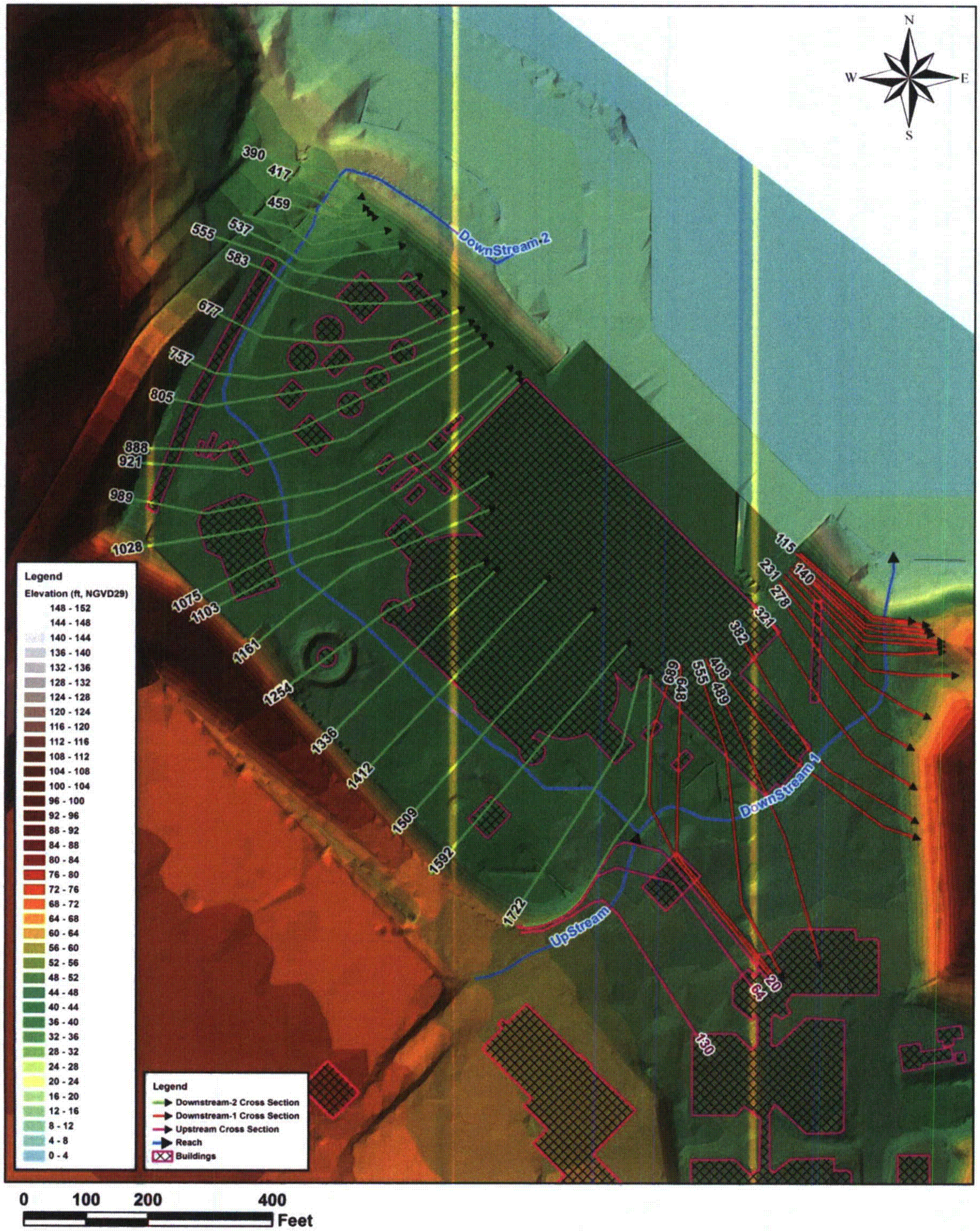


Figure 2.1-8 Schematic of HEC-RAS Model

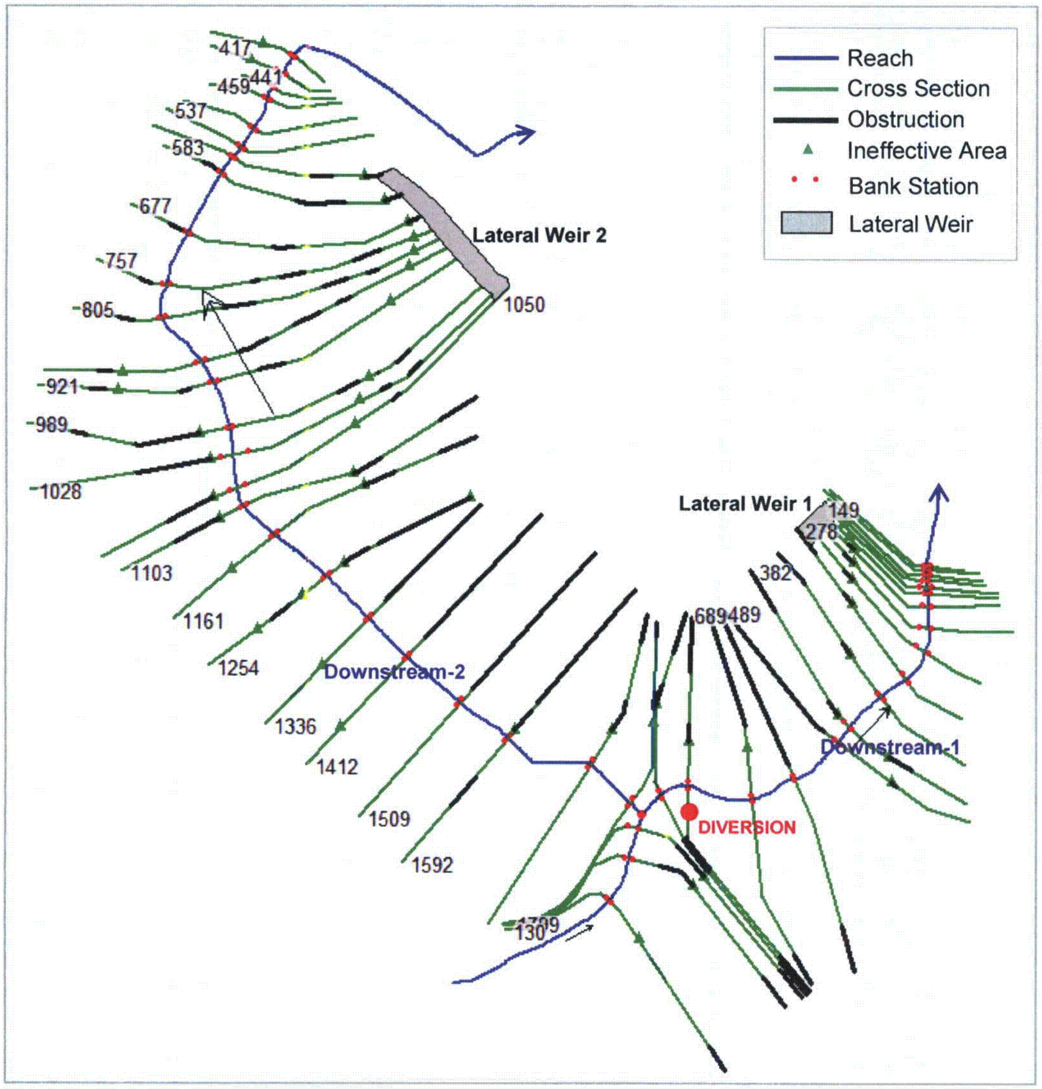


Figure 2.1-9 Local PMP Maximum Water Level Profiles

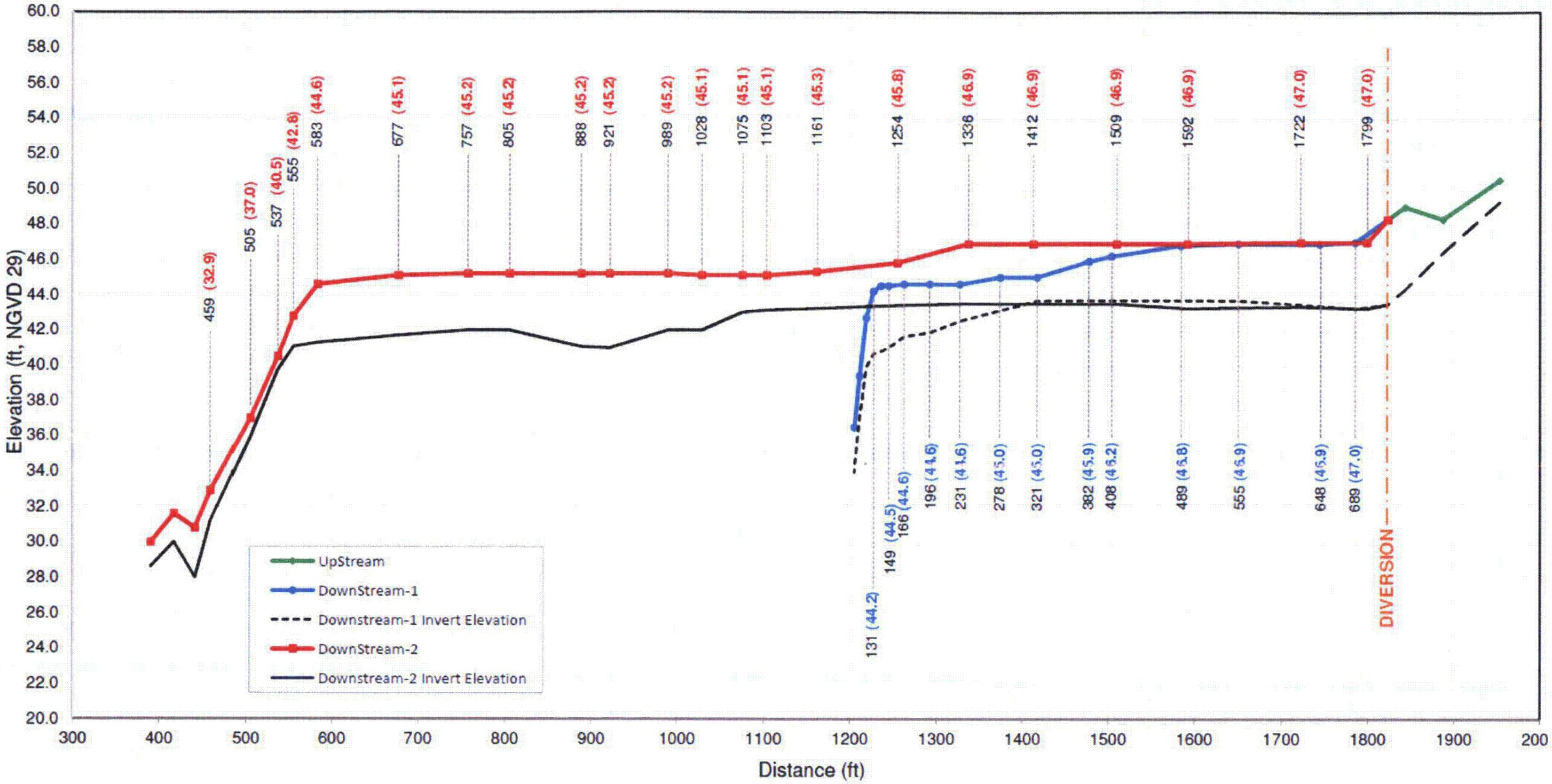
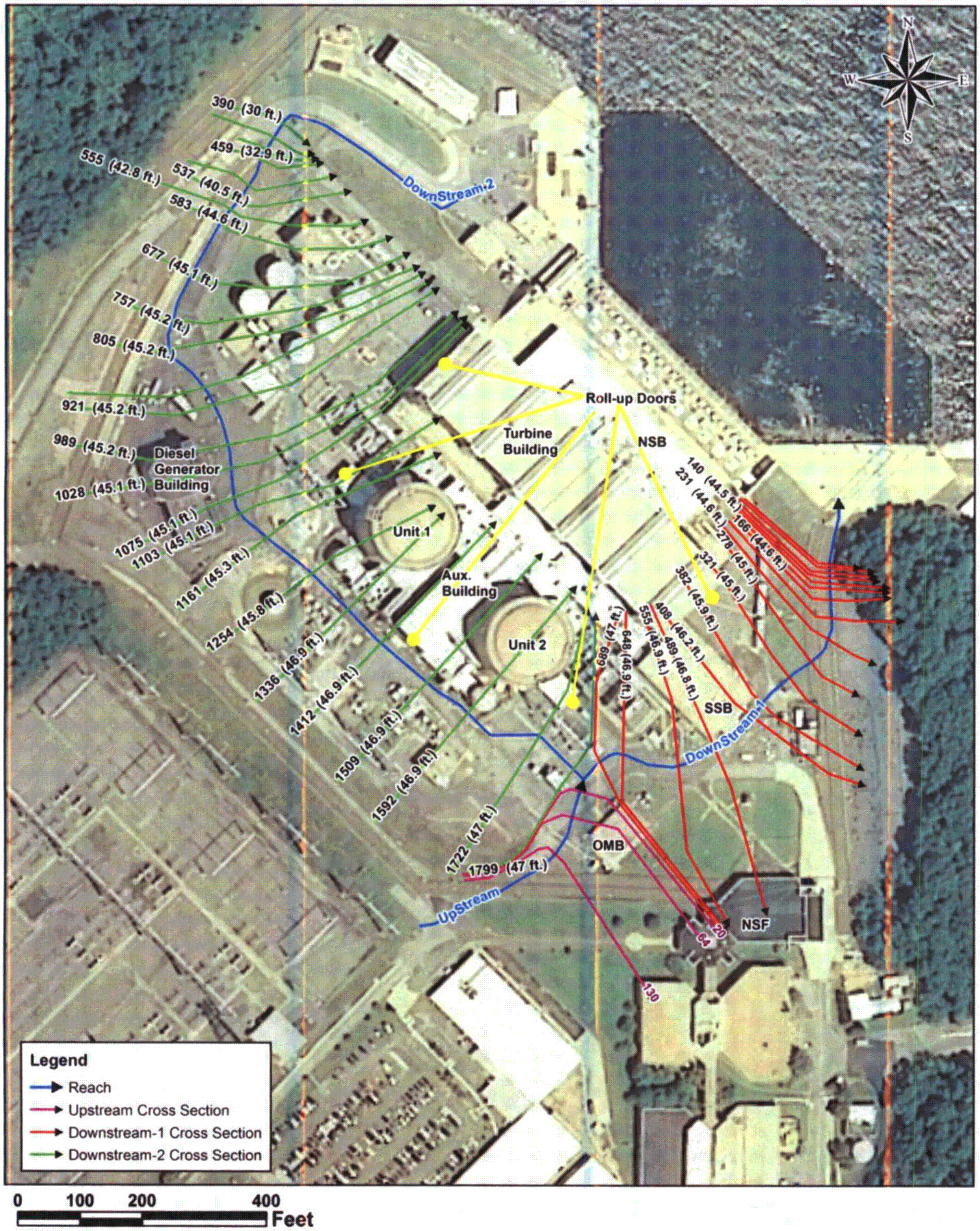
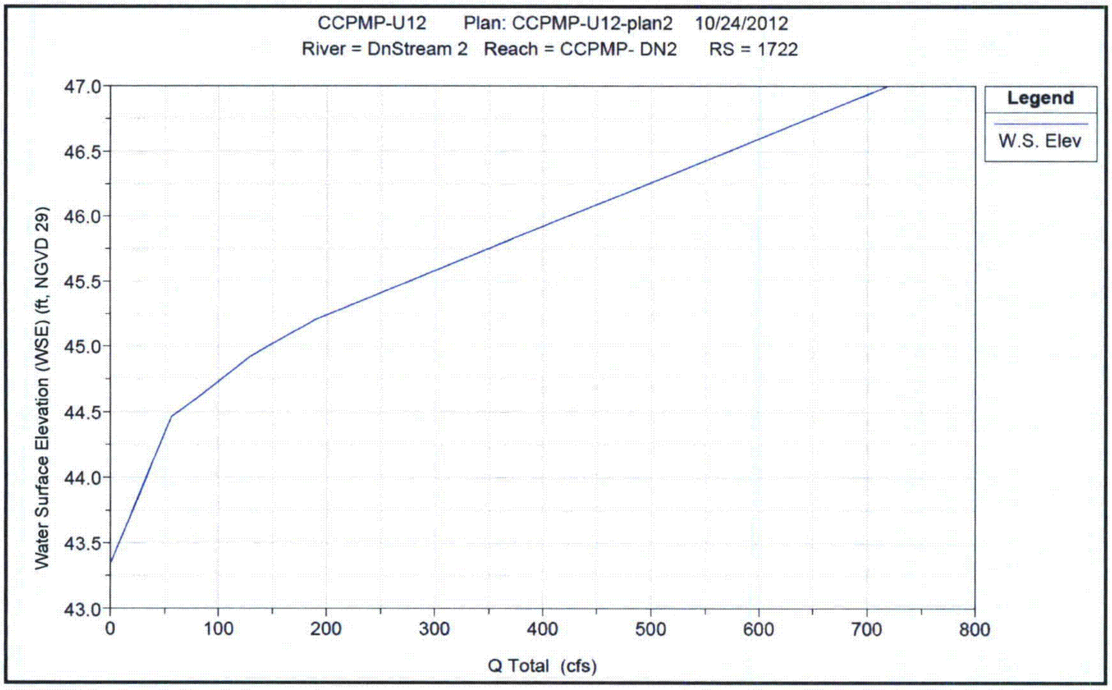


Figure 2.1-10 Maximum PMP Water Levels



Note: Unit 1, Unit 2: Containment Buildings, NSB, SSB: North and South Service Buildings, OMB: Outage Management Building, NSF: Nuclear Security Facility

Figure 2.1-11a Rating Curve and Cross Section Plot at Cross Section 1722 of Downstream-2



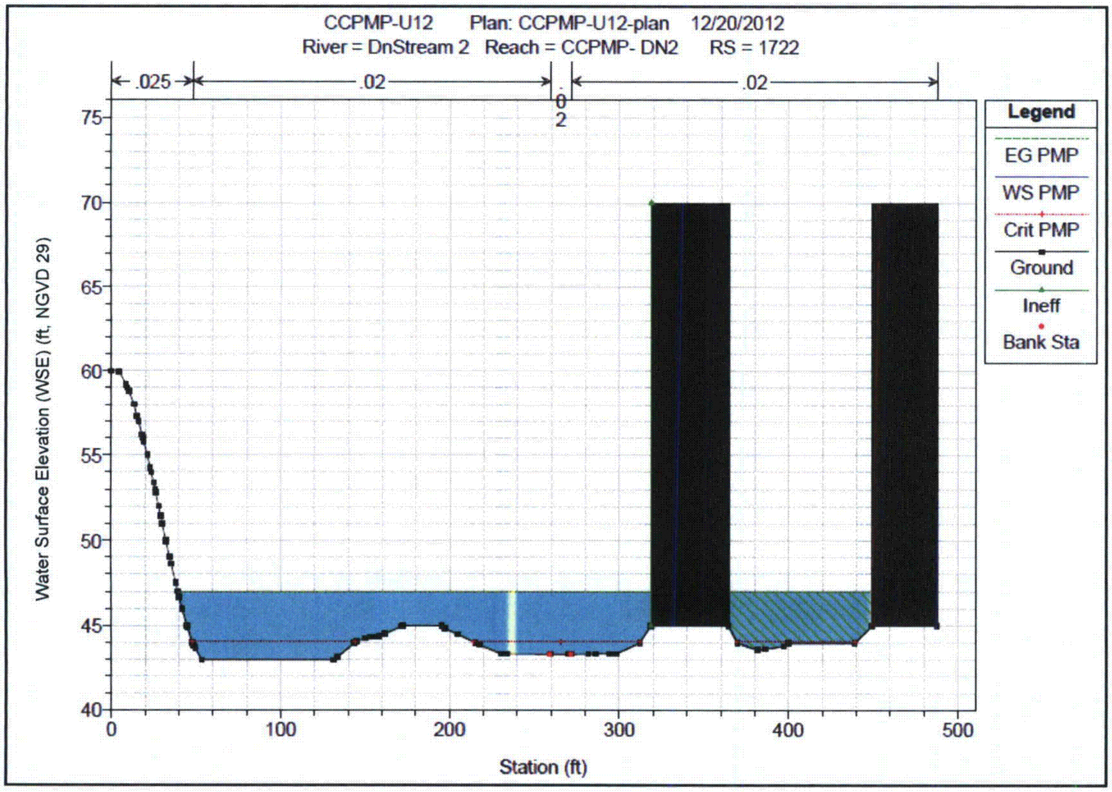
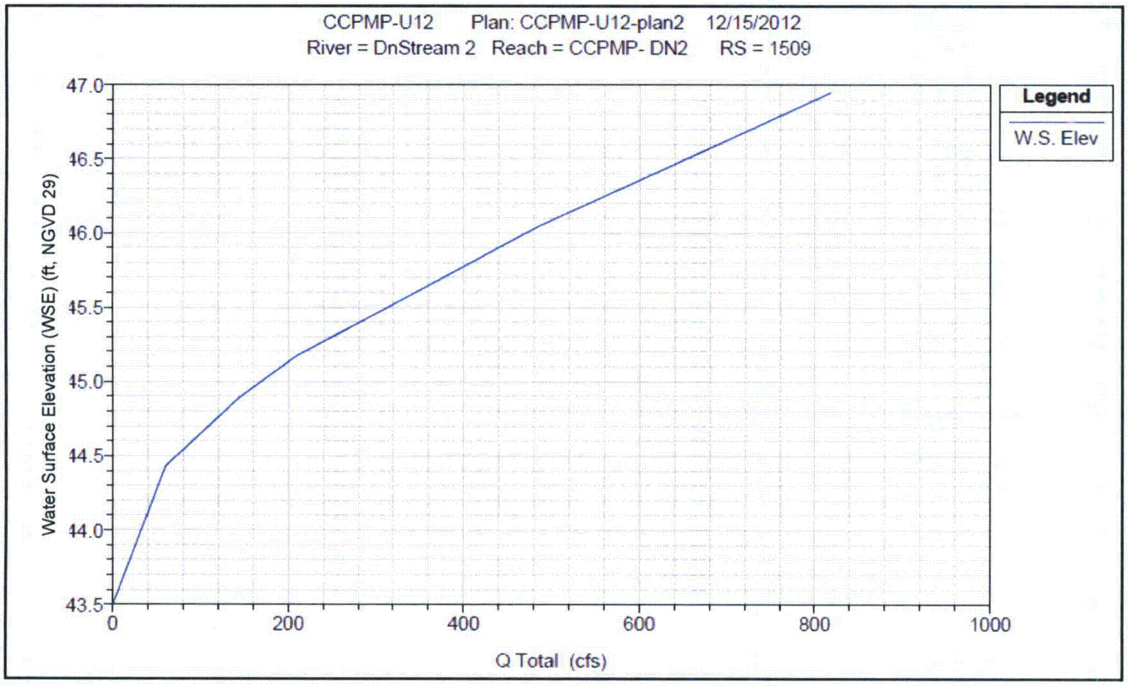


Figure 2.1-11b Rating Curve and Cross Section Plot at Cross Section 1509 of Downstream-2



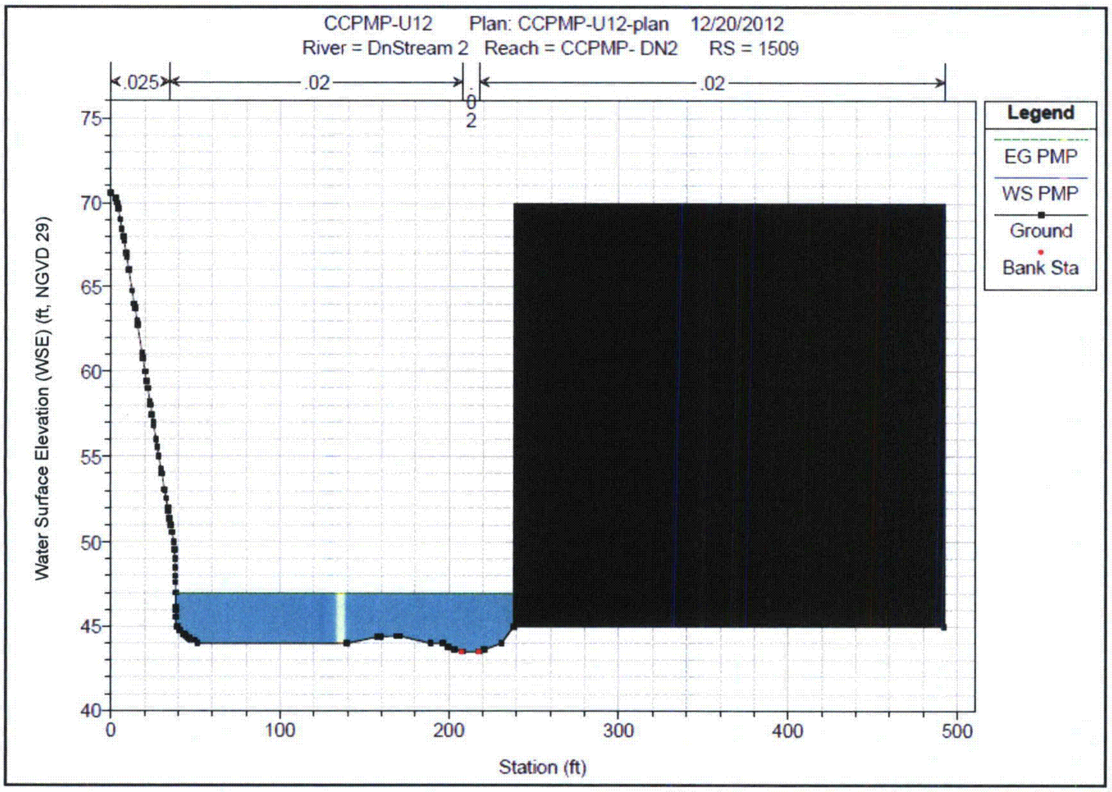
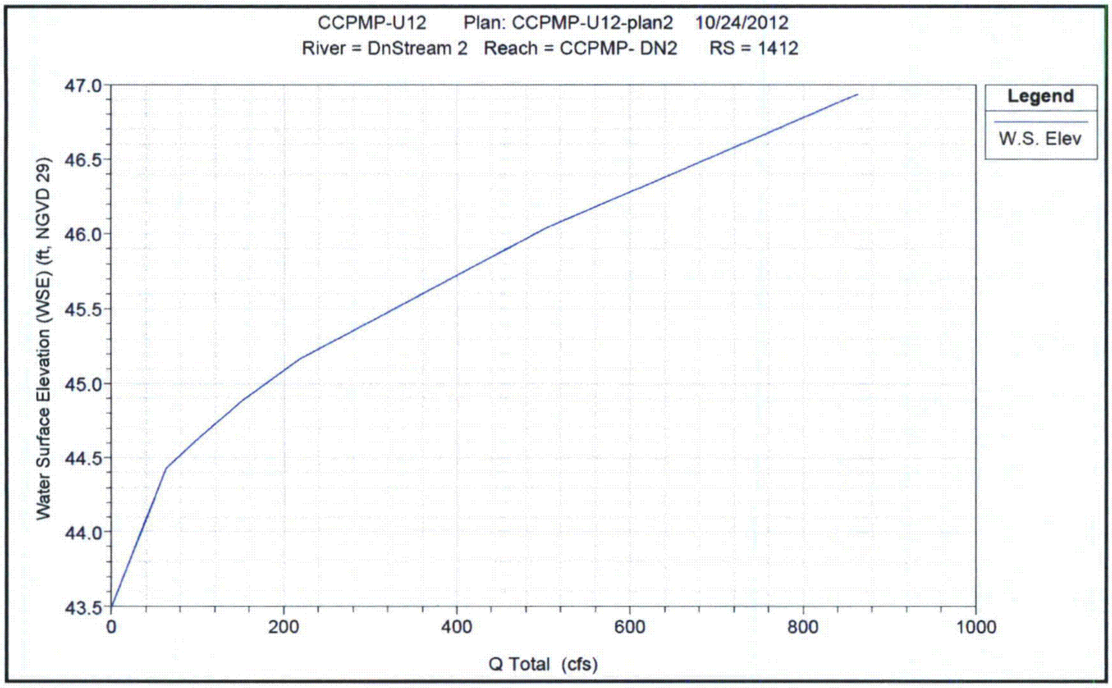


Figure 2.1-11c Rating Curve and Cross Section Plot at Cross Section 1412 of Downstream-2



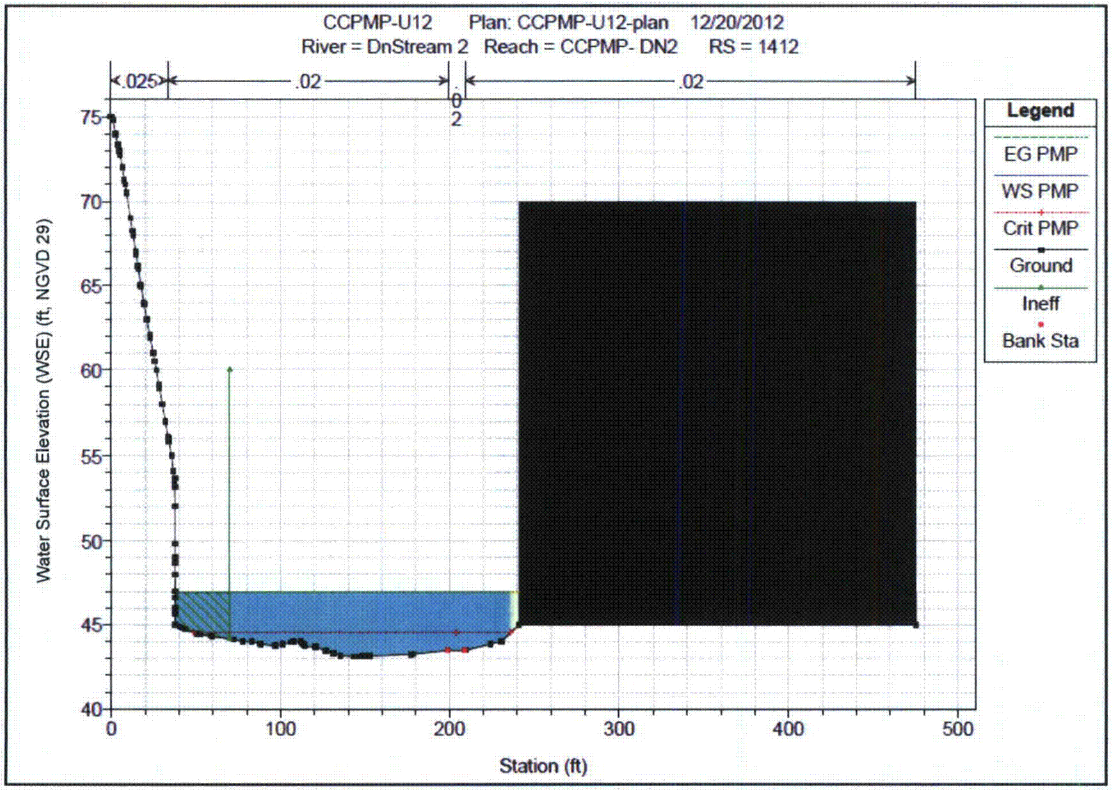
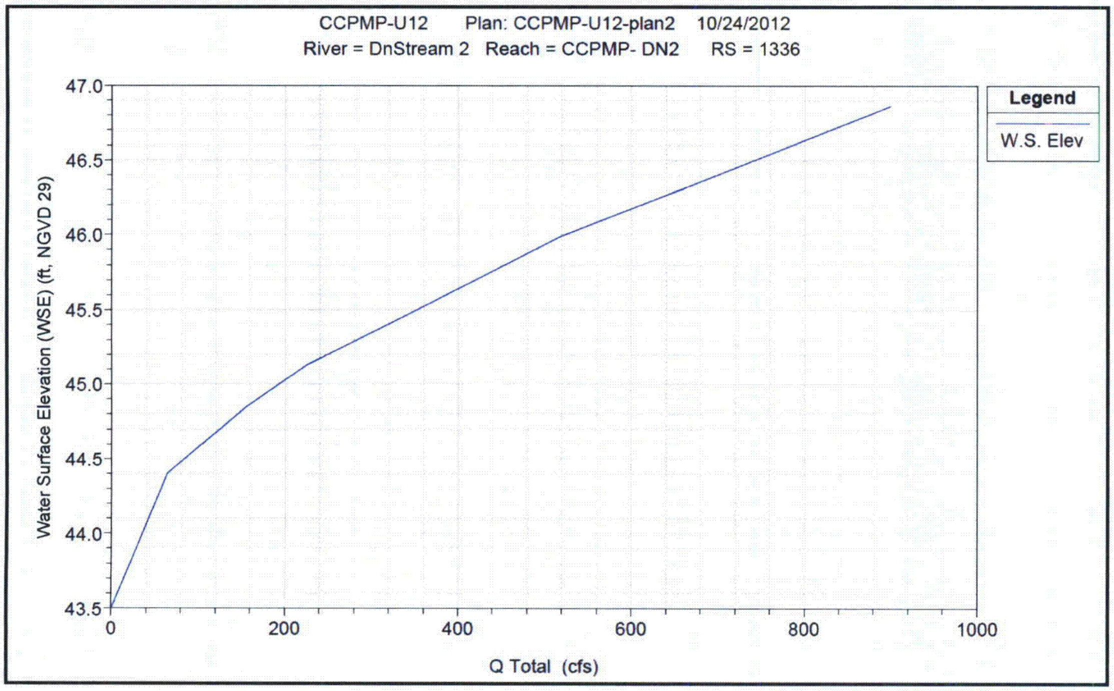


Figure 2.1-11d Rating Curve and Cross Section Plot at Cross Section 1336 of Downstream-2



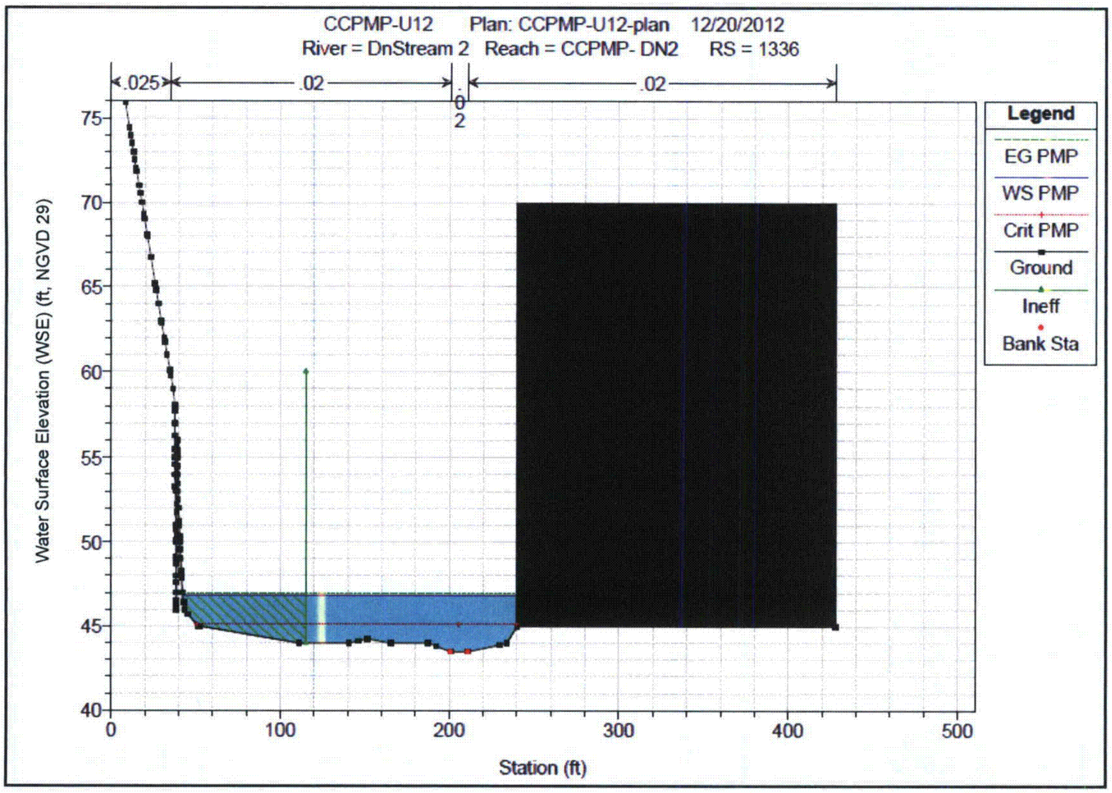
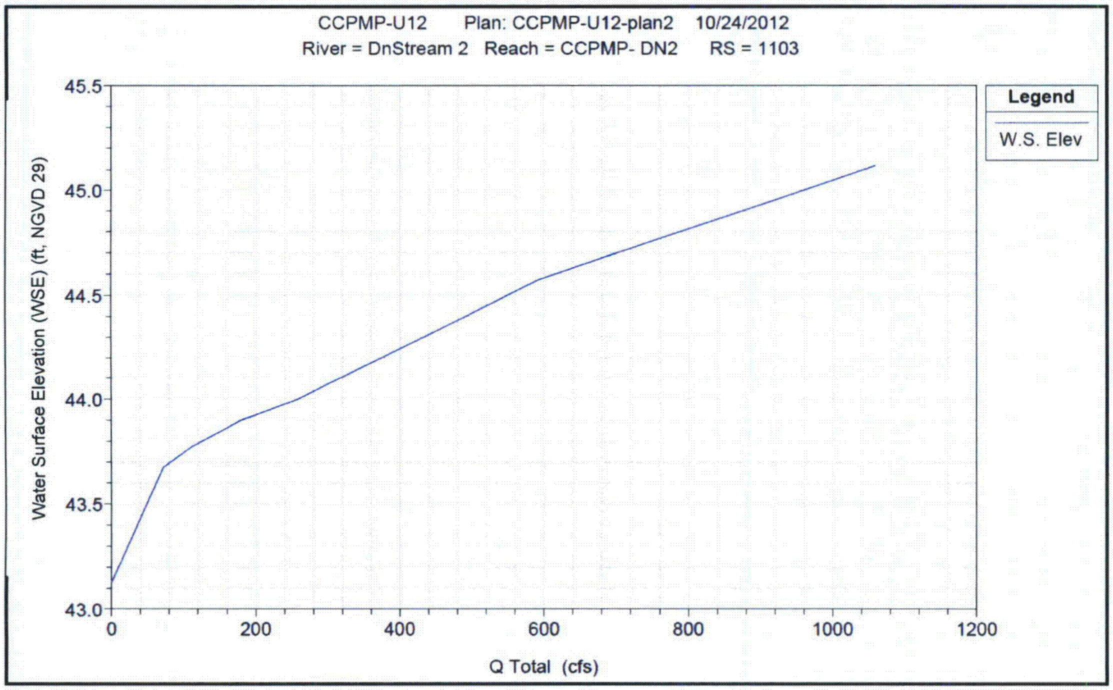


Figure 2.1-11e Rating Curve and Cross Section Plot at Cross Section 1103 of Downstream-2



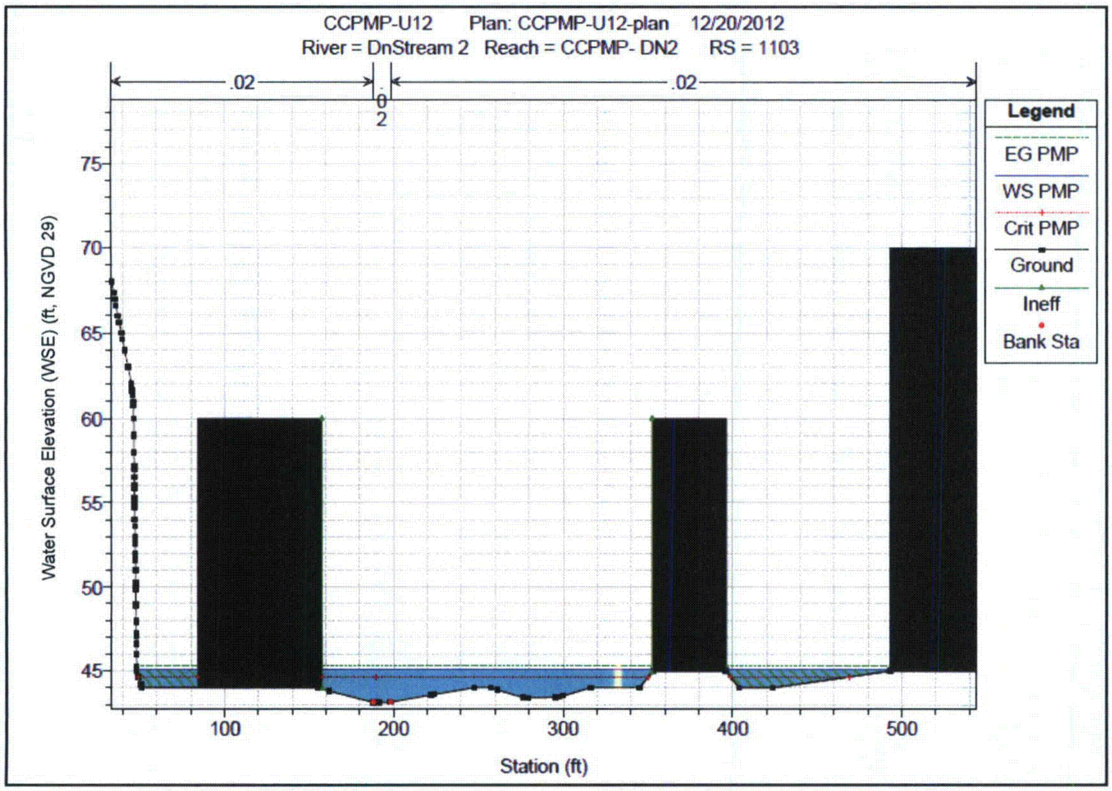
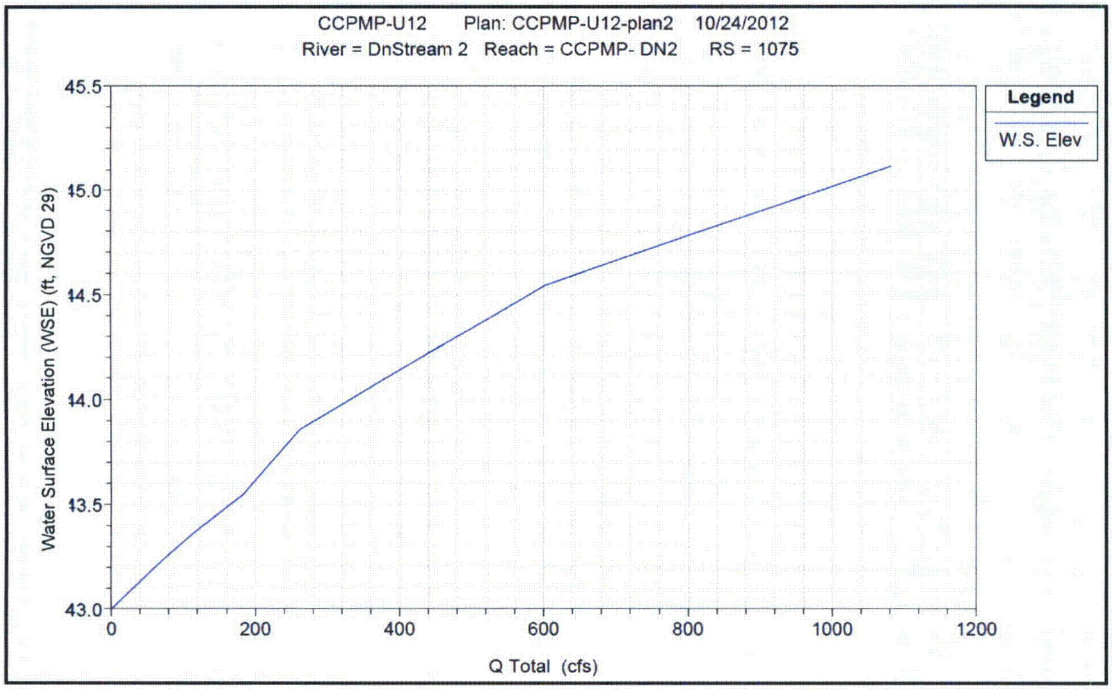


Figure 2.1-11f Rating Curve and Cross Section Plot at Cross Section 1075 of Downstream-2



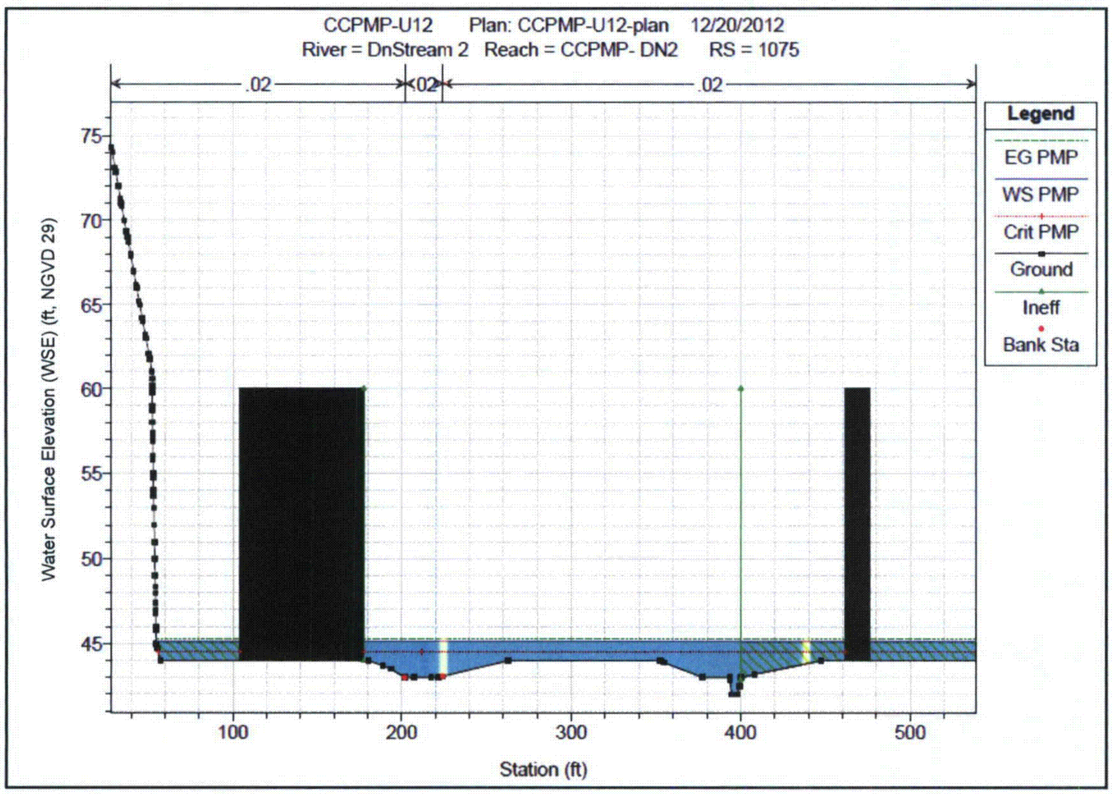
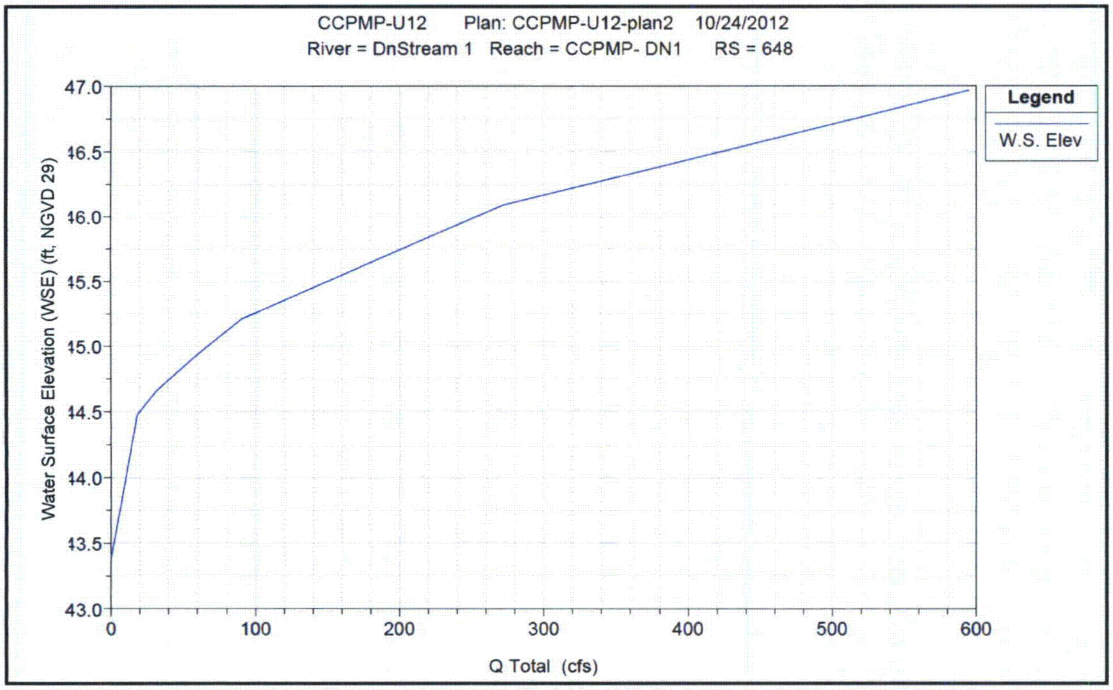


Figure 2.1-11g Rating Curve and Cross Section Plot at Cross Section 648 of Downstream-1



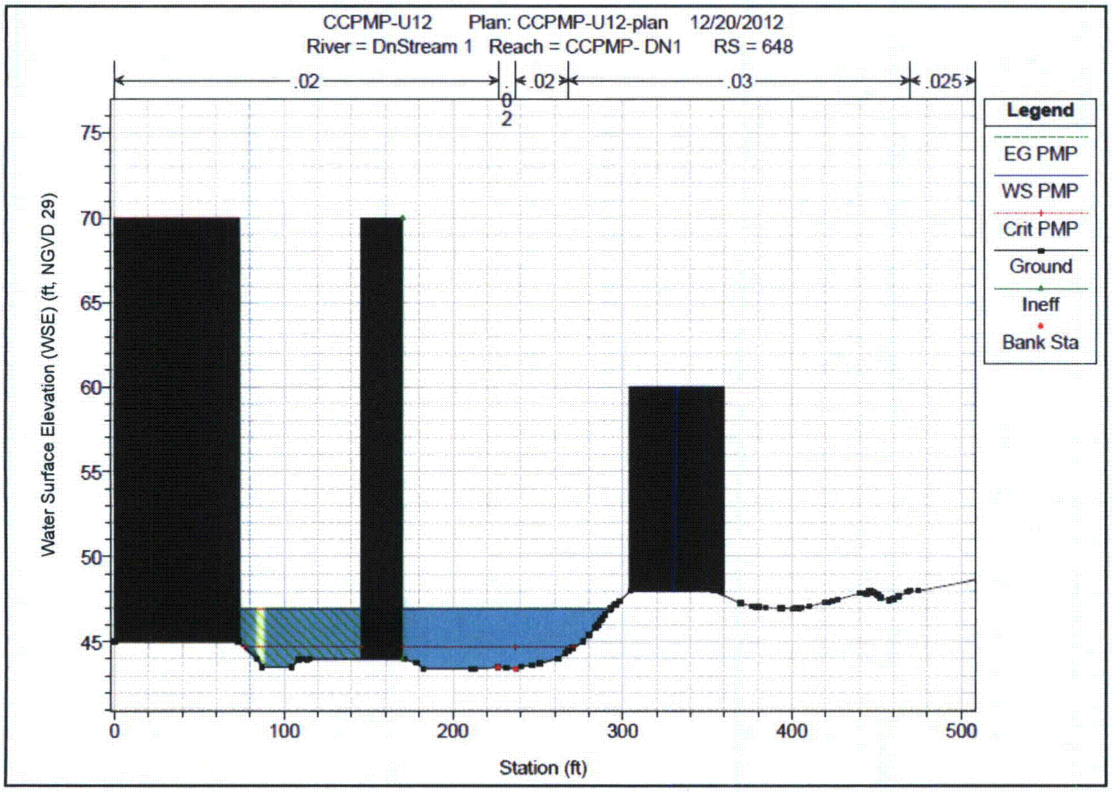
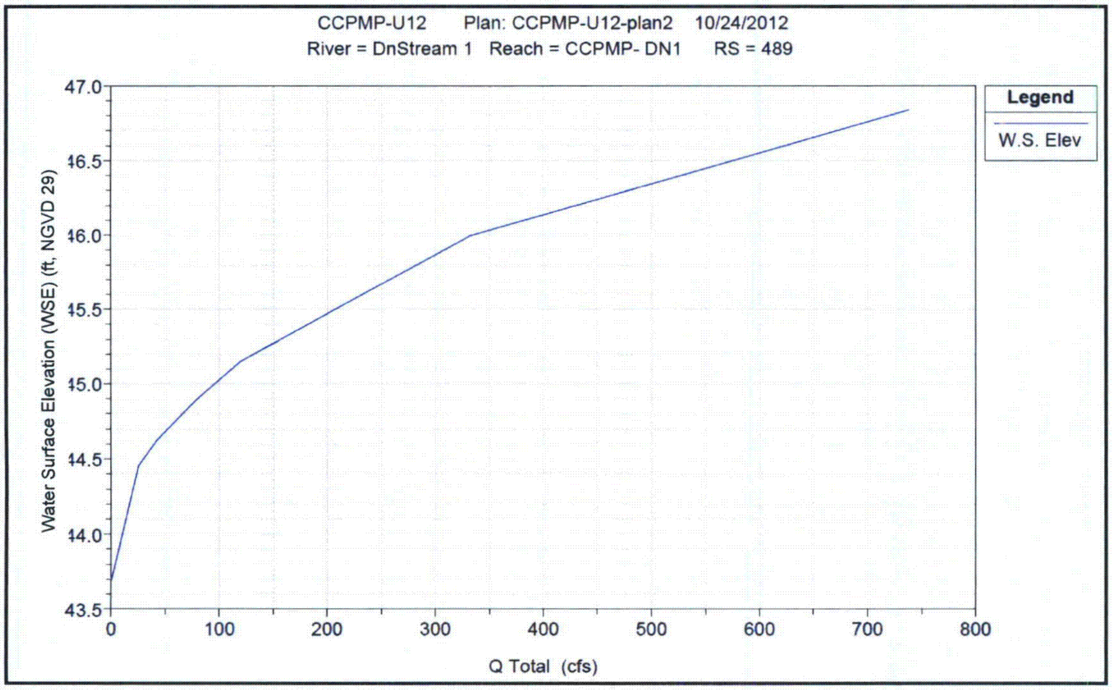


Figure 2.1-11h Rating Curve and Cross Section Plot at Cross Section 489 of Downstream-1



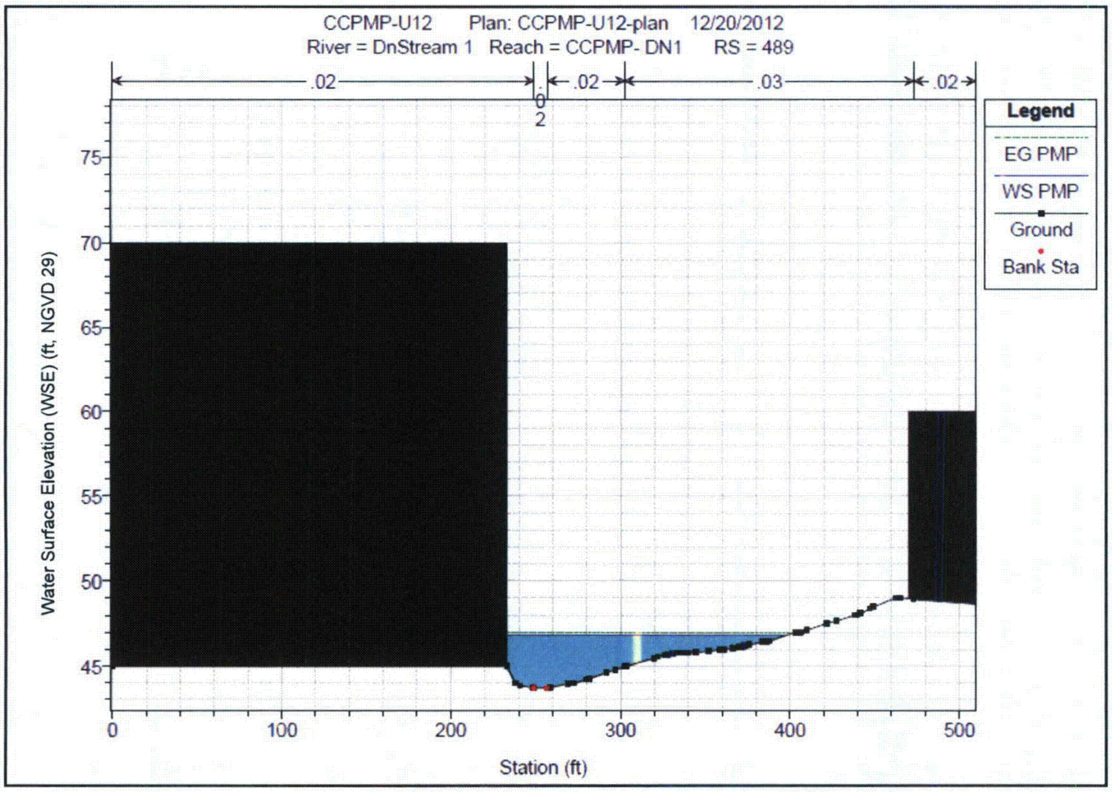
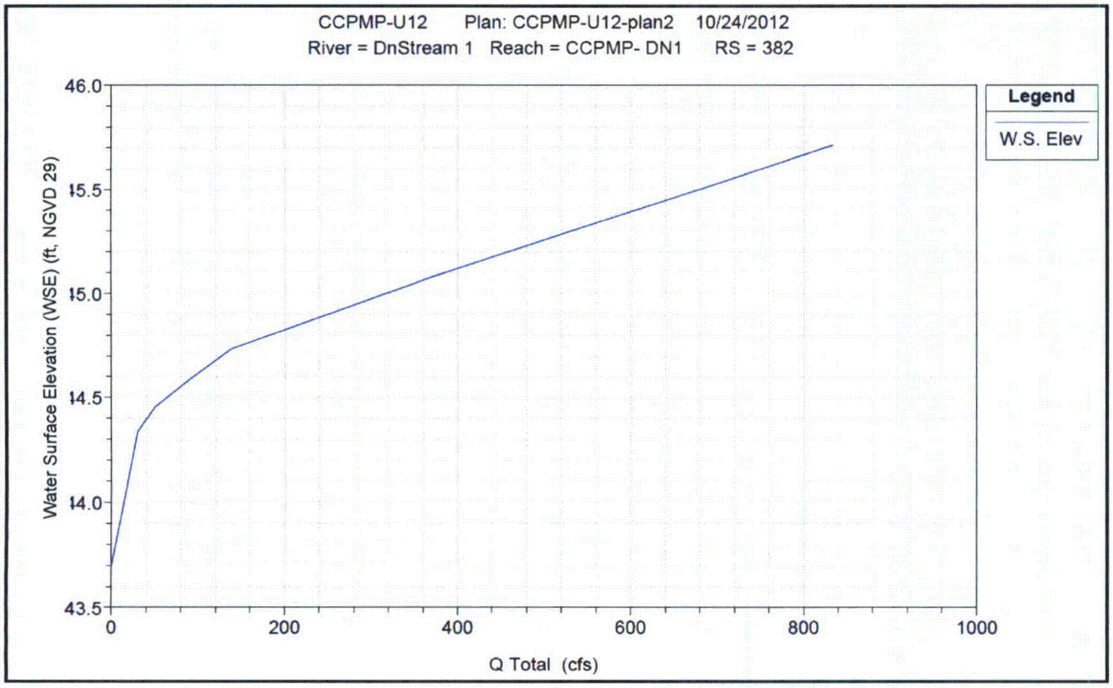


Figure 2.1-11i Rating Curve and Cross Section Plot at Cross Section 382 of Downstream-1



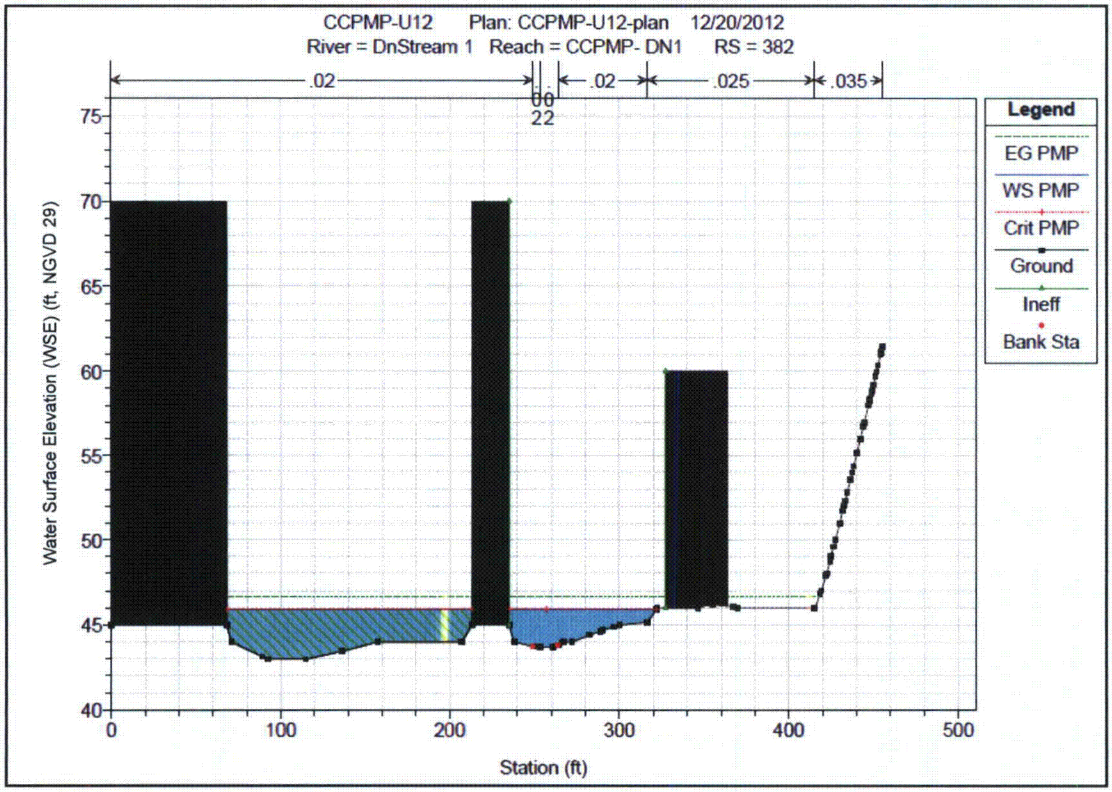
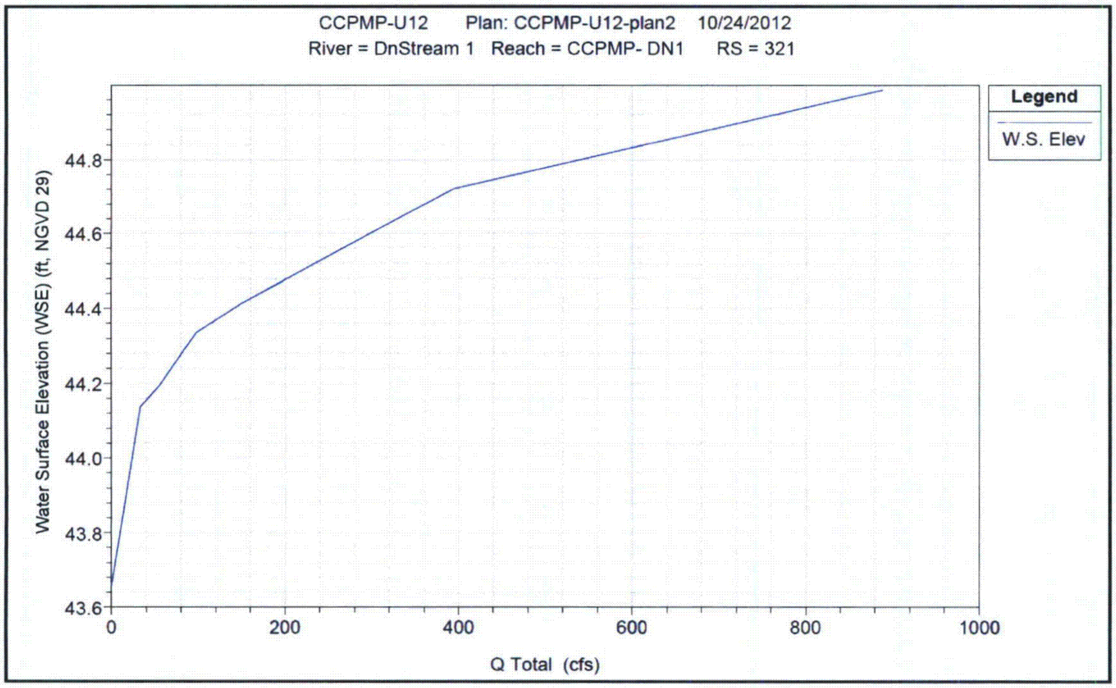
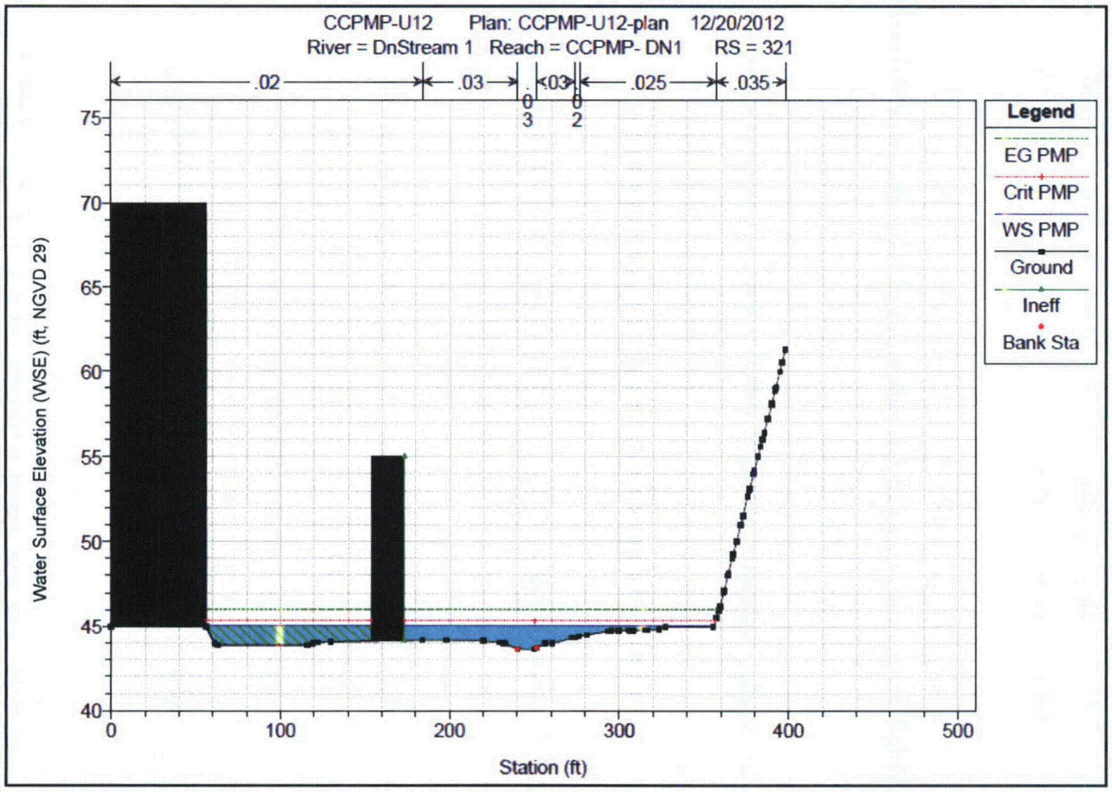


Figure 2.1-11j Rating Curve and Cross Section Plot at Cross Section 321 of Downstream-1





2.2 Flooding in Streams and Rivers

The CCNPP site is located on the western shore of the Chesapeake Bay as shown in Figure 1.1-2. Sources of potential flooding at the CCNPP site are the Chesapeake Bay to the east, Johns Creek and its tributaries to the west, and local intense precipitation directly over the site. This section discusses the probable maximum flood (PMF) on streams and rivers as a result of the PMP over the watershed.

The Chesapeake Bay is the largest estuary on the east coast of the United States. The surface area of the Chesapeake Bay and its tidal tributaries is approximately 4,480 sq mi (Reference 2.2-1). Many tributaries discharge into the Chesapeake Bay, including the Susquehanna, Patapsco, Patuxent, Potomac, Rappahannock, York, and James Rivers. The Chesapeake Bay empties into the Atlantic Ocean near Norfolk, Virginia, about 110 mi south of the CCNPP site.

Since the Chesapeake Bay is connected to the Atlantic Ocean, water levels in the bay are largely influenced by coastal processes, for example, tides, storm surges, seiches, and wind-generated waves. Although, river discharge into the Chesapeake Bay can have some effect on water levels, this effect is minimal compared to flood water levels generated by the events listed above. Thus, the water levels in the Chesapeake Bay due to the PMF in the Chesapeake Bay Basin are not assessed in this section. Flooding on the Chesapeake Bay and associated hazards to CCNPP Units 1 & 2 due to the events listed above are addressed in Section 2.4, and Section 2.5.

Three streams are identified to have potential impacts on the flood level within the CCNPP property boundary. The first is Johns Creek and its tributary (Branch 3) located southwest of the CCNPP site as shown in Figure 1.1-2. The other two are unnamed creeks, Branch 1 and Branch 2, located southeast of CCNPP Units 1 & 2 also shown in the figure.

Johns Creek is a tributary to St. Leonard Creek, which is a tributary to the Patuxent River as shown in Figure 2.2-1. St. Leonard Creek is tidally influenced at the mouth of Johns Creek and is an extension of the Chesapeake Bay, as is the Patuxent River. The CCNPP site is located far enough away from the limit of the tidally influenced areas that flood flows on these water courses have no effect on the water levels near the site. Thus, neither St. Leonard Creek nor the Patuxent River would have any impact on the PMF on streams or rivers around the CCNPP site.

Johns Creek and its tributaries are located west of the drainage divide between the Maryland Lower Western Shore Watershed and the Lower Patuxent River Watershed and contribute flow to the Patuxent River. Consequently, flooding in Johns Creek is not likely to affect the functioning of the CCNPP Units 1 & 2 plant, which is located in the Maryland Lower Western Shore Watershed. Branch 1 and Branch 2 drain areas south-southwest of the CCNPP Units 1 & 2 plant and discharge directly to Chesapeake Bay. Therefore, flooding in these streams will not affect the safe functioning of CCNPP Units 1 & 2.

Johns Creek crosses the Maryland State Highway 2/4 through a culvert. The minimum top of road elevation at the culvert location is approximately 45.5 ft NGVD 29. If the culvert at Maryland State Highway 2/4 is assumed blocked during a PMF event, flow from Johns Creek would overtop the road and flow towards the Patuxent River. Backwater from Johns Creek could only over flow into the Maryland Lower Western Shore watershed if the water level in Johns Creek exceeds the low point in the drainage divide boundary at about 100.0 ft NGVD 29, which passes through the proposed CCNPP Unit 3 switchyard. The drainage divide boundary is about 55.0 ft above the top of culvert elevation at Johns Creek. Because Johns Creek and its tributaries at the culvert location collect runoff from the headwaters mostly within the CCNPP property boundary with small contributing catchment areas, it is unlikely that flooding in Johns Creek would overtop the drainage divide. Any flooding impact from streams and rivers to CCNPP Units 1 & 2 therefore is precluded.

This evaluation is consistent with the PMF analysis on Johns Creek, as described in CCNPP Unit 3 COLA (Reference 2.2-2). In the CCNPP Unit 3 COLA, the PMP was developed according to procedures outlined in HMRs 51, 52, and 53 using the all-season point PMP depths. The distribution of the PMP storm and the runoff hydrograph was determined using the HEC-HMS computer model (Reference 2.2-3) for the antecedent storm condition as indicated in ANSI/ ANS-2.8-1992 (Reference 2.2-4). Assuming that the culvert at Maryland State Highway 2/4 is completely blocked and using the NRCS unit hydrograph method in the computer program HEC-RAS (Reference 2.2-5), the maximum flood elevation near the CCNPP site was obtained as 65.0 ft NGVD 29. This elevation is about 35.0 ft below the lowest drainage divide at Elevation 100.0 ft NGVD 29.

2.2.1 Conclusions

Because the CCNPP Units 1 & 2 plant is located within the Maryland Lower Western Shore Watershed, flooding in Johns Creek and its tributaries, which are located in the Lower Patuxent River Watershed, will not affect the CCNPP Units 1 & 2 plant. Branch 1 and Branch 2 drain areas south-southwest of the CCNPP Units 1 & 2 plant and discharge directly to Chesapeake Bay. Flooding in these small streams also will not affect the CCNPP Units 1 & 2 plant. This conclusion is consistent with the evaluation in the CCNPP Units 1 & 2 UFSAR.

2.2.2 References

2.2-1 Chesapeake Bay Program, Factoid Sheet, Annapolis, MD, October 2004.

2.2-2 Unistar Nuclear Services, LLC, *Calvert Cliffs Nuclear Power Plant Unit 3, Combined License Application*, Revision 8, March 2012.

2.2-3 U.S. Army Corps of Engineers, *HEC-HMS Hydrologic Modeling System*, Version 3.5, Hydrologic Engineering Center, August 2010.

2.2-4 American National Standards Institute/American Nuclear Society, *Determining Design Basis Flooding at Power Reactor Sites*, ANSI/ANS-2.8-1992, Nuclear Standard 2.8, 1992.

2.2-5 U.S. Army Corps of Engineers, *HEC-RAS River Analysis System*, Version 4.1.0, Hydrologic Engineering Center, March 2008.

Figure 2.2-1 Major Streams and Rivers near CCNPP Units 1 & 2

2.3 Dam Breaches and Failures

As mentioned in Section 1.1, the CCNPP site property is located on the western shore of the Chesapeake Bay in Lusby, Maryland. Flooding sources for the site include the Chesapeake Bay east of the site, Johns Creek west of the site, and local intense precipitation. Johns Creek is a tributary to St. Leonard Creek, which is a tributary to the Patuxent River, which is a tributary to the Chesapeake Bay. Both St. Leonard Creek and the Patuxent River are extensions of the Chesapeake Bay near the CCNPP site. Figure 2.2-1 shows the locations of these surface water features relative to the site. The water levels in these water bodies are controlled by tides, storm surges, waves, and tsunamis in the Chesapeake Bay.

There are no dams on Johns Creek or St. Leonard Creek. There are two dams on the Patuxent River: Rocky Gorge Dam and Brighton Dam. Rocky Gorge Dam is located about 65 mi upstream of the mouth of St. Leonard Creek. Brighton Dam is located about 78 mi upstream of the mouth of St. Leonard Creek. Figure 2.3-1 shows the location of both dams. The combined maximum storage capacity for both of these dams is approximately 49,000 acres-ft (Reference 2.3-1). The surface area of the tidal reach of the Patuxent River, as measured from U.S. Geological Survey topographic maps, is approximately 40.9 sq mi (Reference 2.3-2 through Reference 2.3-7). The tidal reach, shown in Figure 2.3-1, extends from the mouth of Patuxent River to a point about 32 mi upstream. If the total volume of these two reservoirs were to be instantly added to the tidal region of the Patuxent River and not allowed to escape into the Chesapeake Bay, the water level increase in the tidal river reach would be approximately 2 ft. This would create a backwater condition for St. Leonard Creek and possibly Johns Creek. As the drainage divide between the creeks and CCNPP Units 1 & 2 is nearly at 100 ft NGVD 29, there is no risk of the drainage divide being overtopped during a PMF event with a flood elevation of 65 ft NGVD 29 (Reference 2.3-8) near the CCNPP site, as demonstrated in Section 2.2. Therefore, a 2-ft increase in water level in the Patuxent River as a results of postulated failures of the two upstream dams even when combined with a PMF event will have no flooding consequences to the safe functioning of the plant.

Flood water levels from dam breaches for both Rocky Gorge Dam and Brighton Dam would be much less than the 2 ft increase estimated above due to flood attenuation over the 65 mi reach of the river. Additionally, the dam break flood flow would continue to discharge to the Chesapeake Bay without being retained within the Patuxent River as conservatively assumed in the above evaluation.

Several other dams are located on other tributaries to the Chesapeake Bay upstream of the CCNPP site. However, dam failures from these other dams would have negligible flooding effect to the CCNPP site as the flood waves would discharge directly into the Chesapeake Bay. Once the flood wave reaches the Chesapeake Bay, water levels would be attenuated by the size and storage volume available in the Chesapeake Bay.

2.3.1 Conclusions

There are no dams on Johns Creek or St. Leonard Creek, and as described above, simultaneous failure of the two dams on the Patuxent River will not affect safe functioning of the CCNPP Units 1 & 2 plant.

2.3.2 References

2.3-1 U.S. Army Corps of Engineers, *National Inventory of Dams*, Brighton Dam (MD00005) and Rocky Gorge Dam (MD00020), 2006.

2.3-2 U.S. Geological Survey, *7.5 Minute Series Topographic Maps*, Scale 1:24,000 Mechanicsville MD, 1974.

2.3-3 U.S. Geological Survey, *7.5 Minute Series Topographic Maps*, Scale 1:24,000 Benedict MD, 1983.

2.3-4 U.S. Geological Survey, *7.5 Minute Series Topographic Maps*, Scale 1:24,000 Hollywood MD, 1984.

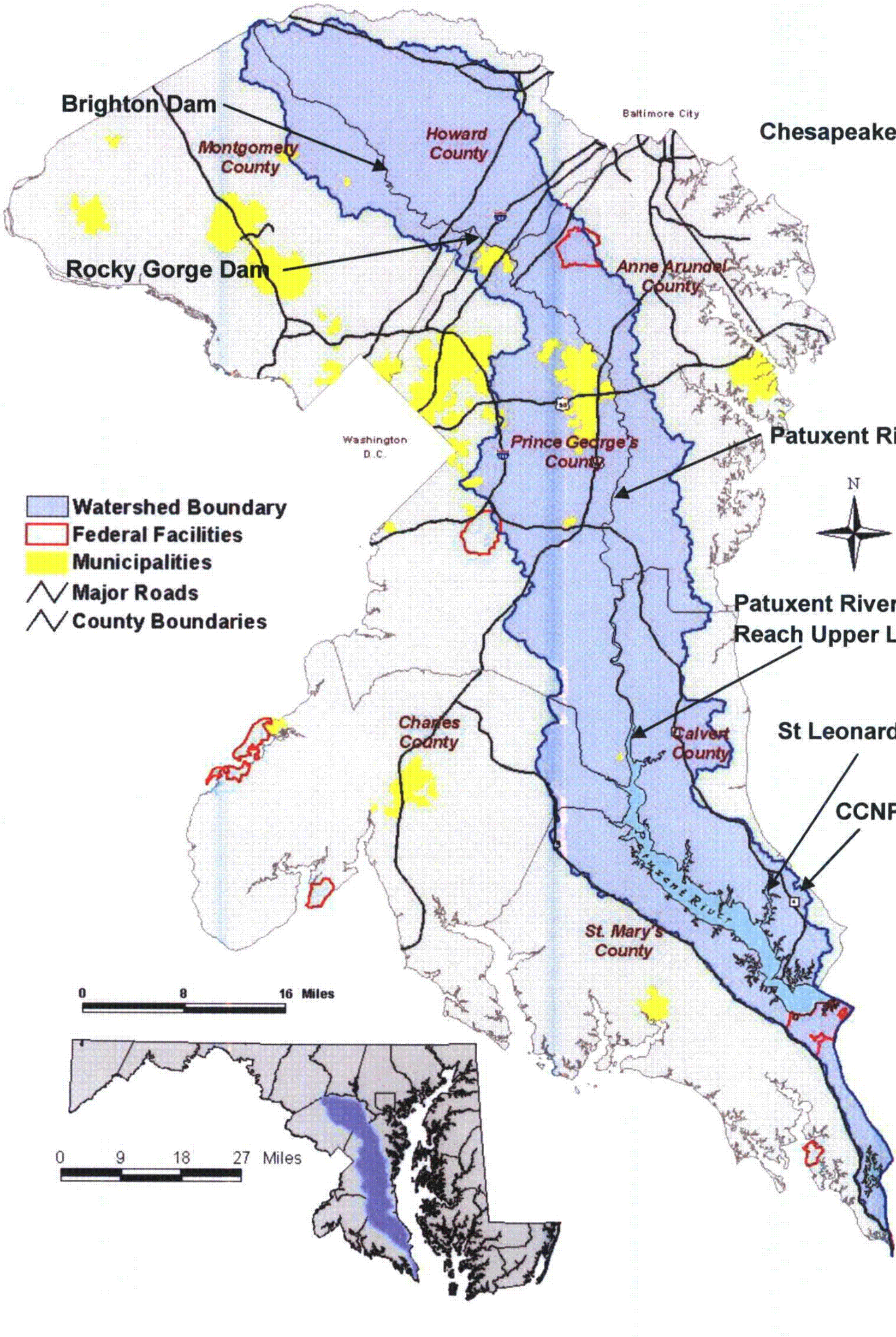
2.3-5 U.S. Geological Survey, *7.5 Minute Series Topographic Maps*, Scale 1:24,000 Broomes Island MD, 1986.

2.3-6 U.S. Geological Survey, *7.5 Minute Series Topographic Maps*, Scale 1:24,000 Cove Point MD, 1987.

2.3-7 U.S. Geological Survey, *7.5 Minute Series Topographic Maps*, Scale 1:24,000 Solomons Island MD, 1987.

2.3-8 Unistar Nuclear Services, LLC, *Calvert Cliffs Nuclear Power Plant Unit 3, Combined License Application*, Revision 8, March 2012.

Figure 2.3-1 Patuxent River Watershed And Dam Locations



2.4 Storm Surge

2.4.1 Introduction

This report summarizes the analyses performed to evaluate the probable maximum storm surge (PMSS) and wave runup for Calvert Cliffs Nuclear Power Plant Units 1 & 2 (CCNPP 1 & 2). These analyses are based on requirements stipulated in the most recent regulations and guidelines published by the United States Nuclear Regulatory Commission (USNRC).

The USNRC issued a letter on March 12, 2012, (Reference 2.4-16) pursuant to Title 10 of the Code of Federal Regulations (CFR), Section 50.54(f), related to implementation of Recommendations 2.1, 2.3, and 9.3 from the Near Term Task Force (NTTF), a section of which calls for reevaluating flood risks to existing Nuclear Power Plants (NPP) in the United States. Recommendation 2.1: Flooding calls for reassessing all applicable flooding and low water risks to existing licensed plants using present-day regulatory guidance and methodologies. This requires reevaluating all flood-causing mechanisms for the site and comparing the recalculated flood levels to the current licensing basis at the plant. If the recalculated flooding levels are higher than the current licensing basis, additional work will be required to assess those flooding risks to the plant and additional protection measures, if needed, shall be implemented. Analyses are also required to verify that an adequate water supply exists to shut down the plant in the event of natural events.

This reevaluation was performed in accordance with the USNRC's, Design Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America, Nuclear Regulatory Guide (NUREG)/CR-7046, November 2011. NUREG-7046 is the most recent USNRC guidance available for establishing design basis flood level. The guideline recommends the hierarchical hazard assessment (HHA) approach for evaluating the flood hazards for an NPP. The HHA is a progressively refined, stepwise estimation of site-specific hazards that evaluates the safety of NPP structures with the most conservative input parameters, methodology, and assumptions.

2.4.1.1 Vertical Datums Used

This sections uses the local tidal datums as the vertical datums and converts it to NGVD 29 using a site specific datum conversion relation. Consequently, the MSL in this section refers to the local mean sea level tidal datum.

According to the Calvert Cliffs Nuclear Power Plant Unit 3 (CCNPP 3) PSAR, the Mean Sea Level (MSL) datum is 0.64 feet (ft) higher than National Geodetic Vertical Datum of 1929 (NGVD 29), and 0.52 ft higher than the Mean Low Water (MLW) datum. The conversion from MLW to NGVD 29 can then be calculated as NGVD 29 = MLW + 0.12 ft.

2.4.1.2 Summary of Previous Evaluations

In the CCNPP 1 & 2 UFSAR (Reference 2.4-4), the Probable Maximum Hurricane (PMH) parameters were based on the Technical Memorandum Numbers 120 (Reference 2.4-3) and HUR 7-97 (Reference 2.4-18). In the present analysis, updated PMH parameters from National Weather Service (NWS) 23 (Reference 2.4-7) are used. The PMH track in the UFSAR for CCNPP 1 & 2 was developed assuming a central pressure of 912 millibars (mb), peripheral pressure of 1047 mb, radius of maximum wind of 30 nautical miles and forward speed of 23 miles per hour (mph). The storm surge level of the CCNPP 1 & 2 UFSAR (Reference 2.4-4) was determined by using a computer program developed by the Jacksonville District Corps of Engineers. In the current evaluation, the Sea, Lake, and Overland Surge from Hurricanes (SLOSH) program developed by the National Oceanic and Atmospheric Administration (NOAA) (Reference 2.4-9) is utilized. The SLOSH program is the state-of-the-art computer model for determining storm surge. The surge level reported in CCNPP 1 & 2 UFSAR is 16.24 ft NGVD, with a total peak wave runup elevation of 28.14 NGVD 29.

The CCNPP 3 FSAR (Reference 2.4-12) utilized the NWS 23 (Reference 2.4-7) for obtaining the PMH parameters and the SLOSH program (Reference 2.4-9) for determining the storm surge level. The PMH track in the FSAR for CCNPP3 was developed using a central pressure ranging from 897 mb to 982 mb, peripheral pressure of 1,020 mb, radius of maximum wind of 26 nautical miles and forward speed of 19.6 mph. The surge level reported in CCNPP 3 FSAR is 17.6 ft NGVD 29. The wave runup on the Makeup Water Intake Structure (MWIS) in the FSAR for CCNPP3 was calculated based on procedures described in the United States Army Corps of Engineers (USACE) Coastal Engineering Manual (Reference 2.4-13). The wave runup on the intake structure was computed to be 15.6 ft, providing a total peak wave runup elevation of 33.2 ft NGVD 29.

2.4.2 Probable Maximum Storm Surge Evaluations

The evaluation of the PMSS for CCNPP 1 & 2 was performed in two phases. The Phase I analysis is based on the results of the PMSS analysis provided in Section 2.4.5 of the Preliminary Safety Analysis Report (PSAR) for Calvert Cliffs Nuclear Power Plant Unit 3 (CCNPP 3). Specifically, the surge level and wind speed from the CCNPP 3 PSAR analyses were applied to CCNPP 1 & 2 to compute the wind-wave runup at the CCNPP 1 & 2 Intake Structure. The Phase I evaluation is described in detail in Section 2.1. The results of the Phase I evaluation yielded water levels which were higher than both the design basis water levels and the elevation of safety related structures at CCNPP 1 & 2. This initiated the Phase II analysis, in which the storm surge and subsequent wave runup levels were calculated based on site specific hurricane tracks and parameters. The Phase II analysis is described in detail in Section 2.2.

The peak PMSS level with wave runup is compared against the lowest vulnerable or critical elevation of safety related structures and the design basis flood level to determine, if the PMSS poses any danger to the site. The most critical elevation of safety related structures of the CCNPP 1 & 2 is the Intake Structure, which is at elevation (El.) 28.5 NGVD 29. A plan view and cross-section of the Intake Structure is shown on Figure 2.4-1 and Figure 2.4-2, respectively. The reported design basis flood elevation for the CCNPP 1 & 2 is El. 28.14 ft. NGVD 29.

2.4.2.1 Phase I Evaluations – Initial Runup and Overtopping Analysis

The application of the results of the storm surge analyses performed for CCNPP 3 is valid for CCNPP 1 & 2 because CCNPP 1 & 2 are located adjacent to CCNPP 3. The distance separating the two has no effect on calculated wind speed or water level values in the Chesapeake Bay. The grid cell which contains CCNPP 3 in the SLOSH computer software used for the surge analysis at CCNPP 3 also contains CCNPP 1 & 2. However, NUREG/CR-7046 was published in November 2011 after the completion of the CCNPP 3 analyses. Section 3.5.1 of this NUREG provides guidance for the application of the HHA process to the storm surge analysis. This guidance suggests estimating the PMSS based on the Maximum of MEOWS (MOM), which is essentially the most critical combination of PMH parameters and storm track. While the sources of PMH parameters are the same as CCNPP 3 (NWS 23), the exact values chosen and track direction are somewhat different in order to create the MOM suggested by the new NUREG. Also the calculation of the wave runup changes between CCNPP 3 and CCNPP 1 & 2, because of differing site geometry.

In the Phase I Evaluation, wave runup was calculated for the Site using the equivalent slope method described in the USACE Coastal Engineering Manual (CEM) (Reference 2.4-13). The geometry of the Intake Structure represents a series of two large steps (Figure 2.4-2). The equivalent slope method is a conservative approach to this type of geometry, compared to calculating runup for vertical wall. According to Van der Meer (Reference 2.4-17), the maximum theoretical limit for runup on a vertical structure is 1.4 times the significant wave weight. For sloping structures, maximum wave runup can reach up to 3.0 times the significant wave weight. The deep-water wave parameters and surge water level calculated for CCNPP 3 were used as inputs into this analysis. Specifically, a significant wave height (H_s; the average height of the highest of one-third waves in a given wave group) of 10.9 ft, a wave period (T; the time for a wave crest to traverse a distance equal to one wavelength) of 5.6 seconds, and a surge level of 18.1 ft (PMSS; includes addition of 20 percent to compensate for SLOSH software error margin) were used for the wave runup analysis. The geometry used in the wave runup analysis (Figure 2.4-2) was taken from as-built drawings provided by Constellation Energy Nuclear Group (CENG). The equivalent slope (α_{eq}) is calculated based on these wave parameters and geometry as shown on Figure 2.4-3. The wave runup is sensitive to the geometry of the lower deck because it affects both the breaking wave height and the equivalent slope used in the wave runup equations. The equivalent slope is dependent on the significant wave height.

The Phase I Evaluation was completed as a series of iterations, starting with the most conservative set of assumptions and continued refining the analysis to include more precise site-specific data for subsequent iterations. The site-specific data modeled in subsequent iterations includes geometry of the lower deck of the Intake Structure and its effect on breaking wave height and equivalent slope used in the analysis. This is consistent with the HHA approach outlined in NUREG/CR-7046 (Reference 2.4-15). All iterations are described in the following subsections.

2.4.2.1.1 Phase I HHA Iteration No. 1

The initial iteration was performed using the wave parameters described above as input into the wave runup using the equivalent slope method (Reference 2.4-13). A smooth surface, Rayleigh distributed wave height for shallow water effect, and a zero degree attack angle for incident waves were all conservatively assumed. The top elevation of the lower deck for the Intake Structure is +10 ft referenced to the NGVD 29. This concrete deck extends from waterfront to the vertical wall of the Pump Room. According to the storm surge calculation result for CCNPP Unit 3, the storm surge level is +18.1 ft

NGVD 29 for the PMH. Therefore, the lower deck is completely submerged under the PMH condition. For conservativeness during this initial iteration of the Phase I Evaluation, the effect of the lower deck of the Intake Structure is neglected i.e., in this iteration, the wave runup is calculated assuming calculation for the deep water wave condition. Therefore, the wave height of incoming waves is not affected by the depth of surf zone above the deck.

Using a significant wave height of 10.9 ft, the equivalent slope was calculated to be 29.2 degrees based on Figure 2.4-3. Using wave runup equations by de Waal and van der Meer (Reference 2.4-5), as described in the USACE CEM (Reference 2.4-13), the runup elevation that 2 percent of the waves will exceed ($R_{u2\%}$) was calculated as 32.7 ft NGVD 29. This was added to the PMSS level of 18.1 ft NGVD 29 to obtain a total wave runup elevation and peak water surface elevation of 50.8 ft NGVD 29. This elevation exceeds the roof elevation of the Intake Structure, which is at El. 28.5 ft NGVD 29.

2.4.2.1.2 Phase I HHA Iteration No. 2

The second and final iteration of the HHA for the Phase I evaluation was performed considering the effect of the lower deck on the incoming wave height and equivalent slope of the Intake Structure. All other input parameters remained the same as used in Iteration No. 1. Since the lower deck is at El. +10 ft NGVD, the incident wave heights is limited by the breaking wave height on the deck. Using breaking wave criteria in the CEM (Reference 2.4-13) the limiting (significant) wave height was calculated to be 6.3 ft, a reduction of 4.6 ft from the significant wave height of 10.9 ft used in Iteration No. 1. Using wave runup equations by de Waal and van der Meer (Reference 2.4-5), the reduced wave height of 6.3 ft changed the equivalent slope to 18.0 degrees and caused the breaking wave surf- similarity parameter (ξ) to fall between 0.5 and 2 which in turn resulted in a $R_{u2\%}$ of 15.5 ft. Thus, the use of lower deck geometry significantly reduced the limiting wave height and the wave runup height. This was added to the surge level of 18.1 ft NGVD 29 to obtain a runup level of El. 33.6 ft NGVD 29. This elevation also exceeds the roof elevation of the Intake Structure (i.e., upper deck of the Intake Structure) which is at El. 28.5 ft NGVD 29.

Further site-specific refinement of the wave parameters were considered, but could not be carried out because there is not adequate data to determine the Rayleigh distributed wave during a PMH event at the Site, and there is no practical method to estimate surface reduction factors. It is also concluded that changing the wind direction from perpendicular-to-coastline to other angles will compromise the criteria used to attain the maximum storm surge for the PMH.

2.4.2.2 <u>Phase II Evaluations – Surge Analysis and Revised Runup Calculations</u>

The Phase I wave runup results for CCNPP 1 & 2, obtained by applying results of the previous CCNPP 3 storm surge calculations, yielded water levels which were higher than the elevations of safety related structures at CCNPP 1 & 2. Therefore, Phase II of the evaluation was initiated in order to recalculate the surge level based on revised storm track and hurricane parameters for the PMH. This analysis consists of two parts – the storm surge calculation using the SLOSH model and the wave runup calculation.

2.4.2.2.1 SLOSH Model

The SLOSH computer model (Reference 2.4-9) was developed to forecast real-time hurricane storm surge levels on continental shelves, across inland water bodies and along coastlines, including inland routing of water levels. SLOSH is a depth-averaged two dimensional finite difference model on curvilinear polar, elliptical, or hyperbolic grid schemes. Modification of storm surges due to the overtopping of barriers (including levees, dunes, and spoil banks), the flow through channels and floodplains, and barrier cuts/breaches are included in the model. The effects of local bathymetry and hydrography are also included in the SLOSH simulation (Reference 2.4-8). The accuracy of the SLOSH model results is within ±20 percent (Reference 2.4-8).

The SLOSH model requires the following parameters to perform a storm surge calculation: hurricane pressure difference; hurricane track description, including landfall location; forward speed; and size, given as the radius of maximum wind (Reference 2.4-8). A polar coordinate grid for Chesapeake Bay was generated by NOAA as part of the built-in package for the SLOSH model (Reference 2.4-9). Version 2 of the Chesapeake Bay grid (cp2) is selected for this calculation. The SLOSH computational grids are maintained by NOAA. A new version of the grid is under development for the study area. However, it was advised by NOAA that cp2 is the most current operational grid for the Chesapeake Bay. Figure 2.4-6 shows the Chesapeake Bay grid for cp2 in SLOSH.

The time sequence of the hurricane track is a required input to the SLOSH model represented by a series of successive locations of the center of hurricane. The hurricane track is derived as a function of the hurricane direction (angle), forward speed, and landfall location (defined as the location where the hurricane crosses the shoreline). Model simulations are performed using SLOSH program executable MS-DOS batch file (sloshdos.bat) for different combinations of the PMH parameters to obtain the maximum surge water level at the site. The model results are processed using the NOAA SLOSH Display Program (Reference 2.4-9). The CCNPP 1 & 2 are located in the SLOSH model grid cell (31, 59) and the simulated time histories of water levels are extracted from this grid cell for the PMSS evaluation.

2.4.2.2.2 Phase II Storm Surge Methodology

In this evaluation, the storm surge level at CCNPP 1 & 2 due to the PMH was determined using guidelines in the National Oceanic and Atmospheric Administration (NOAA) NWS Technical Report 23 (Reference 2.4-7) to estimate the characteristics of the PMH. The SLOSH software was used to calculate the resulting water level.

The storm track for the PMH is based on historical hurricanes that have occurred in the Chesapeake Bay Region. In order to simulate the maximum storm surge at the Site, a hurricane track is selected such that the maximum storm surge will occur at the CCNPP Site:

- The forward direction of the storm just before impacting the Site is perpendicular to the coastline of the Site.

- The storm track stays on the left side (looking toward the Site from the water) of the Site, due to the counterclockwise rotation of hurricanes in the northern hemisphere.
- The storm track stays on the west edge of Chesapeake Bay as much as possible before reaching the CCNPP site. This type of storm track will continue to push the surge toward the upper bay, resulting in a higher storm surge level at the Site.
- The storm makes landfall to the south of bay mouth in order to push storm surge from the Atlantic Ocean into the bay as much as possible.

The selected track is referenced to Hurricane Connie's track in 1955 (Track No. 2 on Figure 2.4-4) (Reference 2.4-6). The following modifications were made to this track to increase the modeled storm surge at the Site (Figure 2.4-5):

1. As the track approached the Site within the Chesapeake Bay, the angle of approach towards the Site was modified slightly to make the approach perpendicular to the Site.
2. The site is on the left side of the track with a distance about 28 nautical miles (radius of maximum wind) from the center of the storm such that the wind speed will be maximized at the Site.

After the storm parameters and track are determined for the PMH, the SLOSH model is applied to calculate the storm surge level. The SLOSH model was developed by NOAA (Reference 2.4-8). It applies the finite-difference scheme to solve the shallow water momentum equations in polar coordinate system.

The following storm parameters were then determined from NWS 23 (Reference 2.4-7):

- **Central Pressure Deficit:** This central pressure deficit is the difference in pressure between the center of the storm and the periphery of the storm. It is dependent upon the latitude/longitude of the center of the storm and is calculated for each point in the PMH track. The central pressure deficit is taken from the largest possible value for the coastal segment between 2,150 and 2,400 nautical miles (south of Wilmington, NC to Ocean City, MD, respectively).
- **Radius of Maximum Winds:** The radius of maximum winds is the radius from the center of the PMH to the point of maximum wind speed. This is taken as the largest possible value for the coastal segment between 2,150 and 2,400 nautical miles.
- **Forward Speed:** This is the speed at which the center of the PMH moves. This is adjusted in the SLOSH model to generate maximum storm surge.

The initial water level includes the 10 percent exceedance high spring tide (1.53 ft MSL), initial rise (1.1 ft), long-term sea level rise (1.07 ft), and mean sea level to NGVD 29 conversion height 0.64 ft. This provided the initial Still Water Level (SWL) of 4.34 ft, which was used as the antecedent water level (antecedent water level is the sum of 10 percent exceedance high tide, initial rise and long term sea level rise) in the SLOSH model and is also identical to that used for the CCNPP3 PMSS analysis. The storm parameters and track are then input into the SLOSH model which contains the polar coordinate grid for the Chesapeake Bay (cp2) (Figure 2.4-6).

The storm track contains 13 points that indicate the track of the PMH. The first eight points indicate the track prior to landfall; the ninth point is the point of landfall or nearest approach; the last four points are post-landfall. The point at the center of the mouth of the Chesapeake Bay mouth was picked as the reference point. Using a radius of maximum wind of 28 miles, a point at 28 miles south of the mouth of the bay was selected in order to ensure that the maximum wind in the PMH was centered over the mouth of the bay; thus, forcing as much water from the Atlantic Ocean into the Chesapeake Bay. This point 28 miles south of the bay is the first point of track after landfall (i.e., point no. 10 in the SLOSH track file).

The points preceding landfall (i.e., points 1-9 in the SLOSH track file) were generated by assuming the PMH travels at the minimum possible forward speed between the first nine points. This is approximately 8 mph for the South Carolina and North Carolina latitudes according to NWS 23. This allows the storm to create a large storm surge wave in the Atlantic Ocean. The PMH then accelerates toward the point of landfall at approximately 31 mph (representative of North Carolina and Virginia latitudes) in order to push the surge through the mouth of the bay.

The next point after landfall (i.e., point no. 11 in the SLOSH track file) was chosen such that it was at the corner of the near 90 degree bend that the track takes in order to directly approach the site. The point of the bend and also the next point (point no. 12) is located such that the storm passes by the site at the radius of maximum winds (i.e., 28 miles) south of the site. Locating the radius of maximum winds directly over the site maximizes the surge at the site and is, therefore, conservative. The PMH travels at a velocity of approximately 18 mph, which is consistent with PMH forward speeds for this latitude from NWS 23, and the calculated wave celerity in the Chesapeake Bay. The coordinates of these 13 points are shown in Table 2.4-1.

2.4.2.2.3 Phase II - Wave Runup Methodology

Wave runup for the Phase II Evaluation was calculated for the Site using the equivalent slope method described in the USACE CEM (Reference 2.4-13), which was also used for the Phase I wave runup analysis. The deep water wave parameters and surge water level calculated for CCNPP 3 were used as inputs into this analysis. Specifically, a significant wave height of 10.9 ft and a wave period of 5.6 seconds were used for the wave runup analysis. The geometry used in the runup analysis was taken from as-built drawings provided by CENG. The equivalent slope based on these wave parameters and geometry is given on Figure 2.4-3. This methodology and set of inputs is identical to those used for the Phase I Evaluation (Section 2.4.2.1), with the exception of the surge level, which varied, based on the iteration of the HHA for the recomputed storm surge analysis.

2.4.2.2.4 Phase II Evaluations - HHA Methodology

The HHA methodology as described by the USNRC (Reference 2.4-15) was applied as follows to perform the evaluations for Phase II:

1. Determine most conservative PMH track which will have potential to cause the greatest degree of flooding at the CCNPP 1 & 2.

2. Develop PMH parameters for these tracks based on the most conservative values available.

3. Use the antecedent water level determined in previous analyses for CCNPP3.

4. Calculate the storm surge level in SLOSH.

5. Calculate the wave runup and add it to the PMSS water level to obtain the peak flood level.

6. Evaluate the peak water level against critical elevations for safety related structures at the Site and the design basis flood elevation.

7. If it is determined during Step 6 that peak water levels are higher than the critical elevations of the safety related structures, return to Steps 2 through 5 to refine the parameters with more site specific data (i.e., less conservative PMH parameters and wave runup assumptions) to reduce to total surge and runup level at the Site. Repeat these steps by progressively refining the PMH tracks and parameters until the inclusion of all available site-specific data or Step 6 yields a water level which is less than the critical elevations of the safety related structures.

If the iterative process exhausts all the available site-specific data and the resulting water level is higher than the original design basis for safety-related structures, then flooding protection measures acceptable to the USNRC should be considered (Reference 2.4-15). Sections 2.4.2.2.4.1 through 2.4.2.2.4.7 provide further details of the analysis performed in Phase II.

2.4.2.2.4.1 Phase II Storm Surge HHA Iteration No. 1

Assuming the most conservative parameters for PMH based on the range of values for varying latitudes given in NWS 23 (Reference 2.4-7), the storm surge level including the antecedent water level of 4.34 ft output by SLOSH is El. 16.6 ft (Figure 2.4-7). A summary of the PMH parameters used for Iteration No. 1 is given for each point in the track in Table 2.4-1.

The accuracy of SLOSH is ±20 percent as indicated by NOAA. Therefore, the surge elevation from SLOSH is multiplied by a factor of 1.2 in order to account for this possible error. The resulting storm surge elevation after accounting for the 20 percent accuracy of SLOSH is El. 19.9 ft NGVD 29.

2.4.2.2.4.2 Phase II Storm Surge HHA Iteration No. 2

A revised storm surge analysis was performed in SLOSH which allowed for the decay of the storm pressure deficit as the storm traveled northward in its track. A summary of the PMH parameters used for Iteration No. 2 is given for each point in the track in Table 2.4-1.

The revised PMH with central pressure deficit of 118 mb near the Site gives the storm surge level output by SLOSH of El. 15.3 ft NGVD 29 (Figure 2.4-8), which includes the antecedent water level of El. 4.34 ft NGVD 29. Applying the 20 percent possible error directly to the SLOSH output, the PMSS becomes El. 18.4 ft NGVD 29.

2.4.2.2.4.3 Phase II Storm Surge HHA Iteration No. 3

The third and final iteration of the surge analysis utilized the results of Iteration No. 2, but applied the 20 percent possible SLOSH error only to the storm surge rise calculated by SLOSH. The storm surge rise is the difference between the SLOSH output water surface elevation (El. 15.3 ft NGVD) and the antecedent water level (El. 4.34 ft NGVD). Applying the 20 percent factor only to this portion of the total surge elevation and adding back in the antecedent water level yields a total surge level of El. 17.5 ft NGVD 29.

The central pressure rises due to low water temperature at high latitude regions, surface friction, and dry/cool inland air. Generally, tropical storms lose their strength at landfall. The land friction causes weakening of the storm after landfall and changes its wind field structure to an unsymmetrical shape. When a tropical storm collides with the inland air, a vertical friction between cool and dry inland air and warm air in the ocean weakens the storm rapidly. Meanwhile, the storm size grows with weaker wind (Reference 2.4-11) at landfall. After landfall, the storm size may grow larger, but the net destructive energy is reduced. In order to model this phenomenon, the wind force reduction due to land friction needs to be implemented in the SLOSH model by varying the land friction at each grid cell. However, the SLOSH model does not allow users to alter the friction at each grid. Additionally, after landfall, the growing storm size and weakening wind field can be simulated by reducing the Holland B coefficient in wind model. Again, the SLOSH model does not allow users to apply this methodology either. Therefore, the HHA process with respect to calculation of the storm surge level was terminated with this third iteration, as further refinement within the parameters and capabilities of the software being applied is not possible.

For comparison with other published guidance in computing surge levels, the results of SLOSH were compared with USNRC Regulatory Guide (RG) 1.59 (Reference 2.4-14). RG 1.59 suggests a total surge level (not including wave runup effects) of El. 22.20 ft MLW (22.32 ft NGVD) at the mouth of the Chesapeake Bay. The maximum surge calculated by SLOSH for Storm Surge Iteration No. 3 without the 20 percent factor applied at the mouth of the Chesapeake Bay is 23.1 ft NGVD 29. Applying the 20 percent factor to the storm surge rise calculated by SLOSH and adding in the antecedent water levels yields a total storm surge level of 26.85 ft NGVD 29 at the mouth of the Chesapeake Bay, which bounds the surge level reported by RG 1.59.

2.4.2.2.4.4 Phase II Wave Runup HHA Iteration No. 1

The initial iteration of the wave runup analysis used the storm surge elevation of 19.9 ft NGVD 29 calculated from the most conservative storm parameters described in Section 2.4.2.2.3.1. Using a significant wave height of 10.9 ft, the equivalent slope was calculated to be 29.2 degrees based on Figure 2.4-3.

The corresponding wavelength, L, was calculated to be 160.5 ft, according to the USACE CEM (Reference 2.4-13). Using wave runup equations by de Waal and van der Meer (Reference 2.4-5), as described in the USACE CEM (Reference 2.4-13), the $R_{u2\%}$ for these conditions was calculated to be 32.7 ft without considering wave reduction due to the influence of the lower deck of the Intake Structure (located at El. 10 ft NGVD). Adding the wave runup to the surge level of El. 19.9 ft NGVD, the resulting peak water surface elevation at CCNPP 1 & 2 is El. 52.6 ft NGVD, which exceeds the roof elevation of Intake Structure at 28.5 ft NGVD 29.

2.4.2.2.4.5 Phase II Wave Runup HHA Iteration No. 2

The second iteration of the wave runup analysis used the storm surge elevation of 18.4 ft NGVD 29 calculated from the second iteration of storm surge analysis described in Section 2.4.2.2.4.2. The wave runup input parameters and results remain the same as the initial iteration of the Phase II wave runup (Section 2.4.2.2.4.4). Adding the wave runup to the surge level of El. 18.4 ft NGVD, the resulting peak water surface elevation at CCNPP 1 & 2 is 51.1 ft, which exceeds the roof elevation of Intake Structure at 28.5 ft NGVD 29.

2.4.2.2.4.6 Phase II Wave Runup HHA Iteration No. 3

The third iteration of the wave runup analysis used the storm surge elevation of 18.4 ft NGVD 29 calculated from the second iteration of storm surge analysis described in Section 2.4.2.2.4.2. The wave runup was calculated using the same inputs for significant wave height (10.9 ft) and wave period (5.6 seconds) as the previous wave runup iterations in Sections 2.4.2.2.4.4 and 2.4.2.2.4.5. The runup was calculated using wave runup equations by de Waal and van der Meer (Reference 2.4-5), as described in the USACE CEM (Reference 2.4-13), as was also done with the previous wave runup iterations. However, on this third iteration, the influence of the lower deck of the Intake Structure at El. 10 ft NGVD 29 was accounted for. Since the lower deck is at El. 10 ft NGVD, the incident wave heights will be limited by the breaking wave height on the deck. Using breaking wave criteria in the CEM (Reference 2.4-13), the limiting (significant) wave height was calculated to be 6.3 ft, a reduction of 4.6 ft from the significant wave height of 10.9 ft used in Iteration No. 1 above. This also reduced the equivalent slope to 18.5 degrees and produced a $R_{u2\%}$ of 15.5 ft. This was added to the surge level of 18.1 ft NGVD 29 to obtain a runup level of El. 33.6 ft NGVD 29. This elevation also exceeds the roof elevation of the Intake Structure at El. 28.5 ft NGVD 29.

2.4.2.2.4.7 Phase II Wave Runup HHA Iteration No. 4

The fourth and final iteration of the wave runup analysis used the storm surge elevation of 17.5 ft NGVD 29 calculated from the third iteration of storm surge analysis described in Section 2.4.2.2.4.3. The wave runup was calculated using the same inputs for significant wave height (10.9 ft) and wave period (5.6 seconds) as the previous wave runup iterations in Sections 2.4.2.2.4.4 through 2.4.2.2.4.6. The significant wave height was reduced to the breaking wave height, as was done in the previous iteration. The breaking wave was reduced to 5.8 ft, which reduced the equivalent slope to 16.7 degrees. The resulting wave runup was calculated to be 13.8 ft, which was added to the

PMSS level of 17.5 ft to produce a peak water surface elevation of 31.3 ft NGVD 29. This peak water surface elevation (PMSS plus wave runup)for this final iteration also exceeds the roof elevation of the Intake Structure at El. 28.5 ft NGVD 29.

As explained in Section 2.4.2.1.2, further possible site-specific data for refining the conservatism of the wave parameters were considered. Since there is not adequate data to determine the Rayleigh distributed wave during a PMH event at the Site, and there is not a practical method to estimate surface reduction factors, no further refinement could be performed to these wave parameters. It is also concluded that changing wind direction from perpendicular-to-coastline to other angles will compromise the criteria used to attain the maximum storm surge for the PMH, which assumed a wind fetch direction perpendicular to the coastline of the site.

2.4.3 Summary of Results

2.4.3.1 Results from Computed PMSS and Runup Analyses

A summary of the results from all wave runup iterations of the Phase I and Phase II Evaluations are provided below in Table 2.4-2.

2.4.3.2 Comparison of Computed Versus Design PMSS Levels

Table 2.4-3 summarizes and compares the results of the current storm surge evaluation at CCNPP 1 & 2 to the design basis storm surge and wave runup parameters and results listed in Section 2.8.3 of the CCNPP 1 & 2 UFSAR (Reference 2.4-4) and in Section 2.4.5 of the CCNPP 3 FSAR (Reference 2.4-12).

In the CCNPP 1 & 2 UFSAR (Reference 2.4-4), the PMH parameter central pressure of the hurricane was estimated from the Technical Memorandum No. 120, published in 1960 (Reference 2.4-3). In this analysis, more recent figures from NWS 23 published in 1979 (Reference 2.4-7) were used. Hence, differences in central pressure for the storms are expected.

The radius of maximum winds used in the CCNPP 1 & 2 UFSAR (Reference 2.4-4) was taken from Memorandum HUR 7-97 (Reference 2.4-18), published in 1968. For CCNPP 3, and also in this current analysis, more recent data from NWS 23 published in 1979 (Reference 2.4-7) were used.

For the CCNPP 1 & 2 UFSAR (Reference 2.4-4) analyses, all PMH parameters were input into a computer program developed by Jacksonville District Corps of Engineers and run on a GE 415 Computer. The SLOSH computer model, developed by the Federal Emergency Management Agency (FEMA), USACE, and NWS in 1992 is a more state-of-the-art model for determination of storm surge levels in the United States. The SLOSH software has been verified against historical events and shown to be within 20 percent of observed water levels (Reference 2.4-10). The SLOSH model was used for both the CCNPP 3 FSAR analyses and the current flood hazard analyses.

For the CCNPP 1 & 2 analyses, the wave height was calculated according to USACE Shore Protection Planning and Design Technical Report No. 4. For the CCNPP 3 FSAR (Reference 2.4-12) and for the current analyses, the wave runup heights were calculated based on procedures described in the USACE Coastal Engineering Manual (Reference 2.4-13).

2.4.4 Conclusions

As shown in Table 2.4-3, the resulting flood levels from all iterations of the HHA for the storm surge flood hazard, including wave runup effects, are above the roof elevation of Intake Structure at 28.5 ft NGVD 29. The results of the final storm surge and wave runup HHA iterations are recommended to be used for evaluation of the impact on the site that the PMSSS and wave runup produce. The final iterations utilize the important site-specific data, such as decay of the hurricane after landfall and as-built geometry of the intake structure, but still maintain conservative assumptions. The most site-specific iteration of the analysis yields a total peak water surface elevation (PMSS plus wave runup) of El. 31.3 ft NGVD, which takes into account the 20 percent accuracy of the SLOSH model for the storm surge rise and the wave breaking effect of the lower deck of the Intake Structure. The estimated peak water surface elevation is approximately 2.8 ft higher than the existing roof (i.e., upper deck) of the Intake Structure.

Impacts of this reevaluated maximum storm surge elevation, which exceed the CLB elevation. Further evaluations and actions planned are described in Section 3.4 and Chapter 4.

2.4.5 References

2.4-1 BG&E, 1972, Baltimore Gas and Electric Co. (BG&E), "Equipment Location, Service Building, Water Treatment Area & Intake Structure, Section J-J," BG&E Drawing No. 60-220-E, Rev. 5, November 1972.

2.4-2 BG&E, 1988, Baltimore Gas and Electric Co. (BG&E), "Plant Property and Buildings," BG&E Drawing No. 61-502-E, Rev. 20, March 1988.

2.4-3 BEB, 1960, Beach Erosion Board (BEB) Technical Memorandum No. 120, "The Prediction of Hurricane Storm-Tides in New York Bay," August 1960.

2.4-4 Constellation Energy, Updated Final Safety Analysis Report (UFSAR) for Calvert Cliff Units 1 & 2, Section 2.8.3, Revision 39.

2.4-5 de Waal, J. P., and van der Meer, J. W. (1992), "Wave Run-up and Overtopping on Coastal Structures," Proceedings of the 23rd International Coastal Engineering Conference, American Society of Civil Engineers, 2, pp. 1758-1771, 1992.

2.4-6 NHC, 2012, National Hurricane Center (NHC), Past Track Seasonal Map – 1955, Website: http://www.nhc.noaa.gov/pastall.shtml#tracks_all , Date Accessed October 2012.

2.4-7 NOAA, 1979, National Oceanic and Atmospheric Administration (NOAA) Technical Report NWS 23, "Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Wind fields, Gulf and West Coasts of the United States," Washington DC, September 1979.

2.4-8 NOAA, 1992, National Oceanic and Atmospheric Administration (NOAA) Technical Report NWS 48, "SLOSH: Sea, Lake, and Overland Surges from Hurricanes," April, 1992.

2.4-9 NOAA, 2009, National Oceanic and Atmospheric Administration (NOAA), "Sea, Lake and Overland Surge from Hurricanes (SLOSH)," Version 3.94, 2009.

2.4-10 NOAA, 2012, National Oceanic and Atmospheric Administration (NOAA), Website: http://slosh.nws.noaa.gov/sloshPub/#intro , Date Accessed October 2012.

2.4-11 Powell, 2010, Powell, M. D., Murillo. Dodge, S. P., E. Uhlhorn J. Gamache, Cardone, V., Cox, A., Otero, S., Carrasco, N., Annane, B., and St. Fleur, R., "Reconstruction of Hurricane Katrina's Wind Fields for Storm Surge and Wave Hindcasting," Ocean Engineering, 37, pp. 26-36, 2010.

2.4-12 Unistar, 2012, Unistar Nuclear Services, Final Safety Analysis Report (FSAR) for Calvert Cliff Unit 3, Section 2.4.5, Revision 8.

2.4-13 USACE, 2008, United States Army Corps of Engineers (USACE), Coastal Engineering Manual EM-1110-2-1110, 2008.

2.4-14 USNRC, 1977, United States Nuclear Regulatory Commission (USNRC), "Design-Basis Floods for Nuclear Power Plants," Regulatory Guide 1.59, Revision 2, 1977.

2.4-15 USNRC, 2011, United States Nuclear Regulatory Commission (USNRC), "Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America," NUREG/CR-7046, PNNL-20091, 2011.

2.4-16 USNRC, 2012, United States Nuclear Regulatory Commission (USNRC), "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 (NTTF) Review of Insights from the Fukushima Dai-Ichi Accident," March 2012.

2.4-17 Van der Meer, 1998, "Dikes and Revetments: Design, Maintenance and Safety Assessment," Editor: Krystian W. Pilarczyk, 1998.

2.4-18 Weather Bureau, 1968, U.S. Weather Bureau Memorandum, "Interim Report - Meteorological Characteristics of the Probable Maximum Hurricane, Atlantic and Gulf Coasts of the United States," HUR 7-97, 1968.

Table 2.4-1 PMH Parameters for Storm Surge HHA Iteration Nos. 1 and 2

HHA Iteration No. 1 and No. 2				HHA Iteration No. 1	HHA Iteration No. 2
Time (hours)	**Latitude (degrees)**	**Longitude (degrees)**	**Radius of Maximum Wind (Nautical Miles)**	**Pressure Deficit (mb)**	**Pressure Deficit (mb)**
0	33.2826	66.9846	28	124	124
6	33.5906	67.7726	28	124	124
12	33.8975	68.5774	28	124	124
18	34.1755	69.3822	28	124	124
24	34.4665	70.1870	28	124	124
30	34.7296	71.0066	28	124	124
36	34.9973	71.8263	28	124	124
42	35.2717	72.6460	28	124	124
48	35.5528	73.4657	28	124	124
54	36.5880	76.4940	28	124	123
60	38.1455	76.0599	28	124	118
66	38.1844	77.9466	28	124	80.2
72	39.3241	80.7021	40	80	55

Table 2.4-2 Summary of HHA Results

Phase	Wave Runup Iteration	Key Storm Surge Assumptions	PMSS Water Level (ft NGVD)	Key Wave Runup Assumptions	Calculated Equivalent Slope	Wave Runup Height (ft)	PMSS + Wave Runup Water Level (ft NGVD)
I	1	Use CCNPP 3 results	18.1	Does not take into account effect of Intake Structure lower deck	29.2°	32.7	50.8
	2	Use CCNPP 3 results	18.1	Reduced breaking wave height and equivalent slope due to effect of Intake Structure lower deck	18.0 °	15.5	33.6
II	1	No decay of storm; 20 percent SLOSH accuracy applied directly to SLOSH results	19.9	Does not take into account effect of Intake Structure lower deck	29.2 °	32.7	52.6
	2	Storm decays with higher latitudes; 20 percent SLOSH accuracy applied directly to SLOSH results	18.4	Does not take into account effect of Intake Structure lower deck	29.2 °	32.7	51.1
	3	Storm decays with higher latitudes; 20 percent SLOSH accuracy applied directly to SLOSH results	18.4	Reduced breaking wave height and equivalent slope due to effect of Intake Structure lower deck	18.5 °	16.2	34.6
	4	Storm decays with higher latitudes; 20 percent SLOSH accuracy applied only to storm surge rise	17.5	Reduced breaking wave height and equivalent slope due to effect of Intake Structure lower deck	16.7 °	13.8	31.3

Table 2.4-3 Comparison of Parameters and Results for Storm Surge and Wave Runup Analyses

Parameter/Results	UFSAR for CCNPP 1 & 2 (Reference 2.4-4)	FSAR for CCNPP 3 (Reference 2.4-12)	CCNPP 1 & 2 Values Used/Computed in this Report[3]
PMH Parameters and Results			
Central Pressure Deficit	135 mb	123 mb	124 – 55 mb
Radius of Maximum Winds	26 nautical miles	10 – 26 nautical miles	28 – 40 nautical miles
Wave Runup Parameters and Results			
Antecedent Water Level	2.82 ft NGVD	4.4 ft NGVD	4.4 ft NGVD[1]
PMSS Level	16.24 ft NGVD	17.6 ft NGVD	17.5 ft NGVD
Significant Wave Height	11.4 ft	10.8 ft	10.9 ft[1]
Breaking Wave Height	N/A	7.6 ft	5.84 ft
Wave Runup	11.9 ft	15.6 ft[2]	13.8 ft
PMSS + Wave Runup	28.14 ft NGVD	33.2 ft NGVD	31.3 ft NGVD

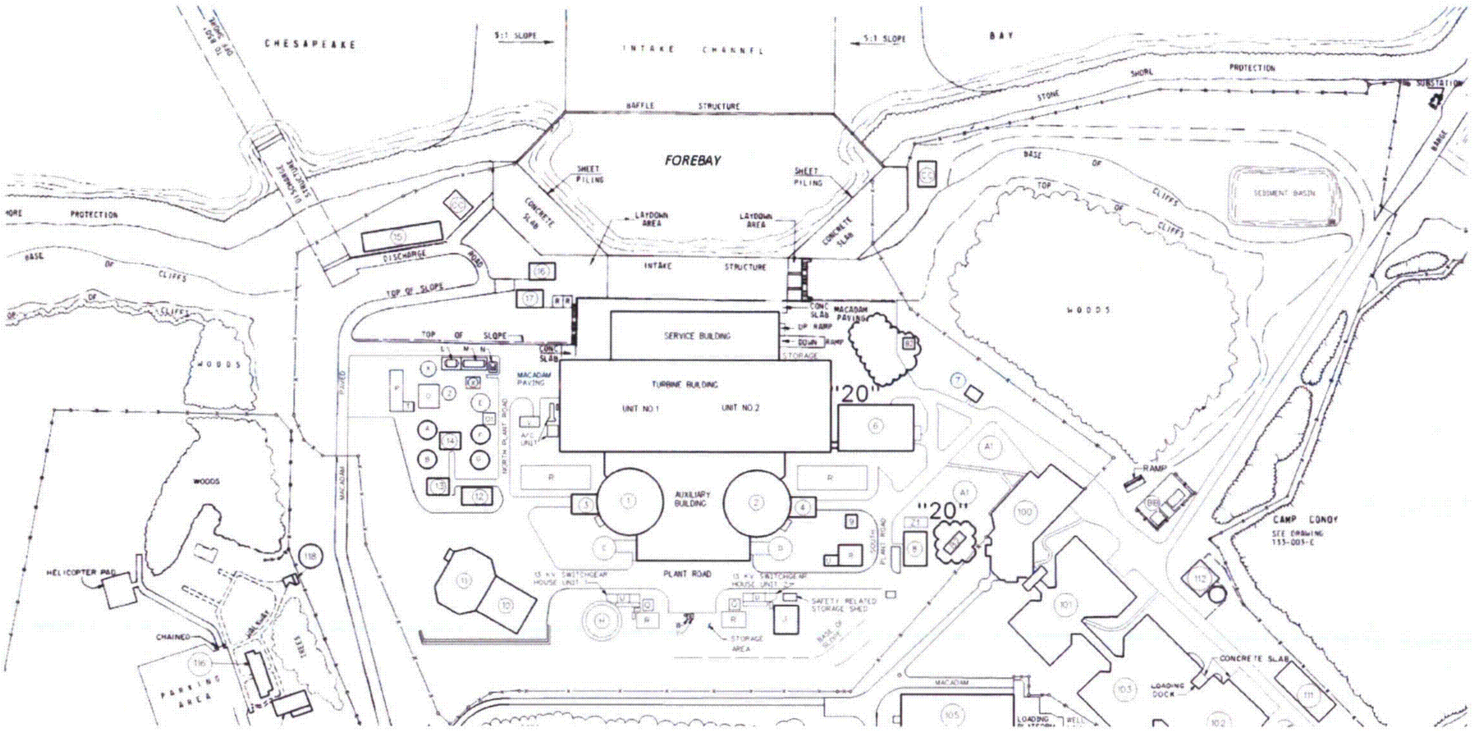
Note:

(1) Calculated for CCNPP 3.

(2) Wave runup value is calculated for the Makeup Water Intake Structure (MWIS).

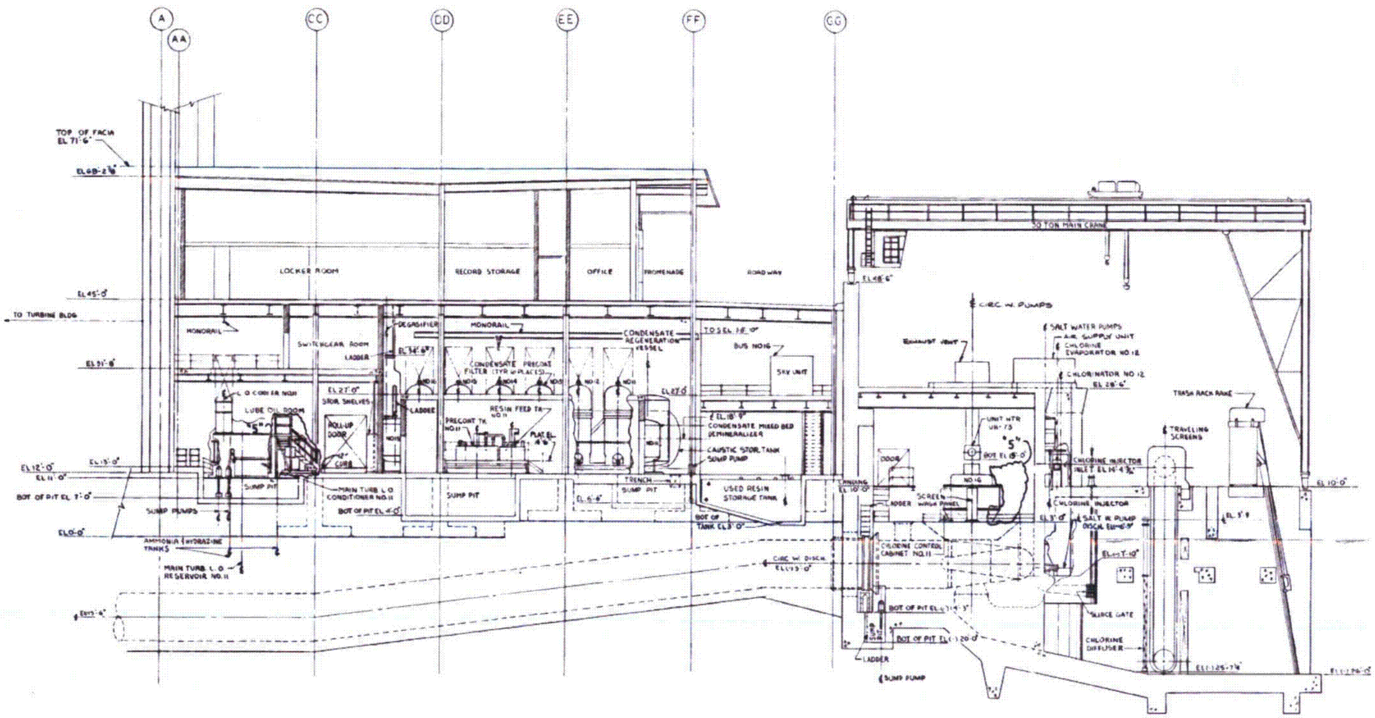
(3) Values presented here are for Phase II Iteration No. 4.

Figure 2.4-1 Plan View of Intake Structure and Forebay



Drawing made by CNPP Engineering Services Department, drawing number 61502SH0002 (Reference 2.4-2)

Figure 2.4-2 Cross Section of Intake Structure



Drawing made by Baltimore Gas & Electric Co.; Drawing Number 60-220-E (Reference 2.4-1)

Figure 2.4-3 Representative Equivalent Slope for Intake Structure of CCNPP 1 & 2

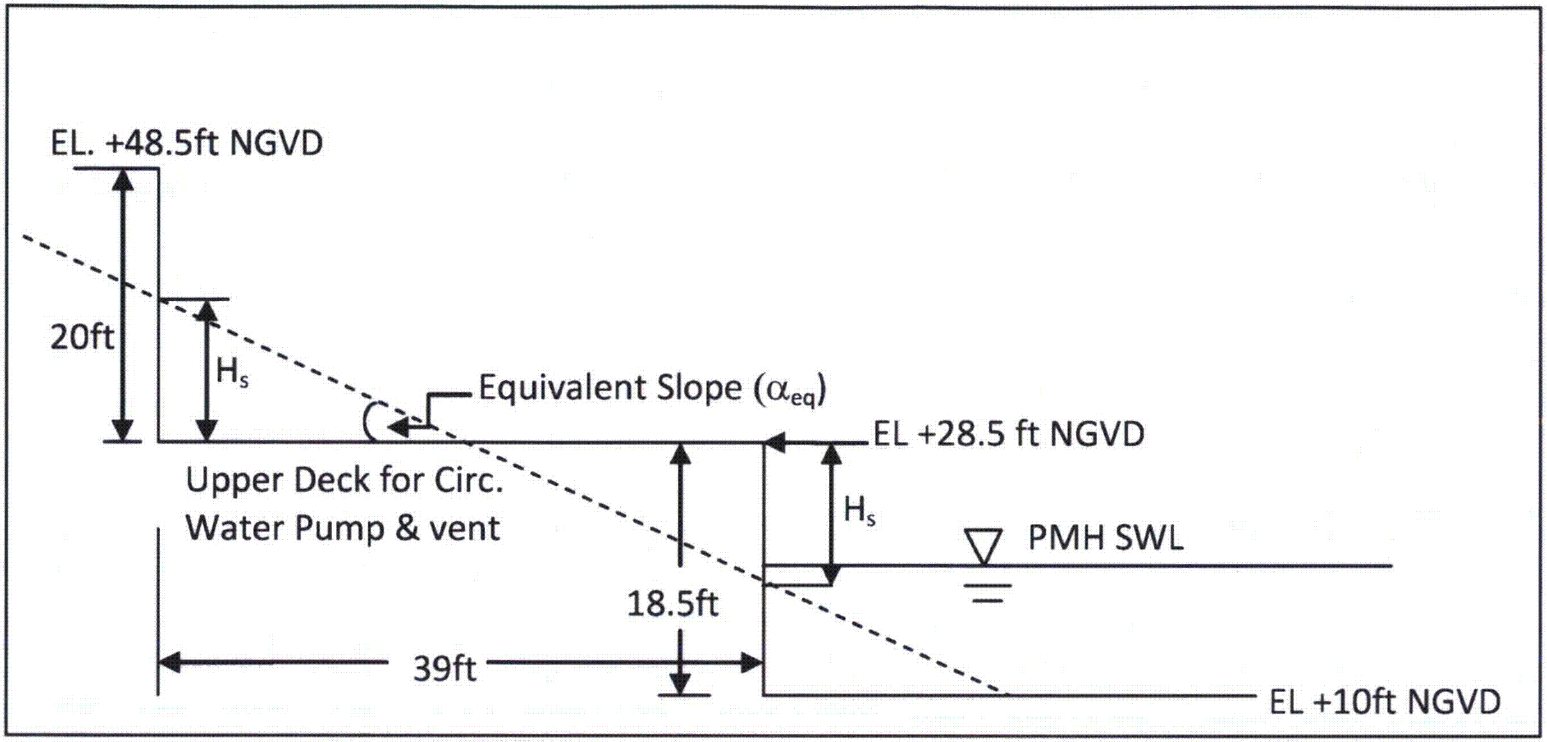


Figure 2.4-4 Storm Tracks for 1955 Hurricanes (Reference 2.4-6)

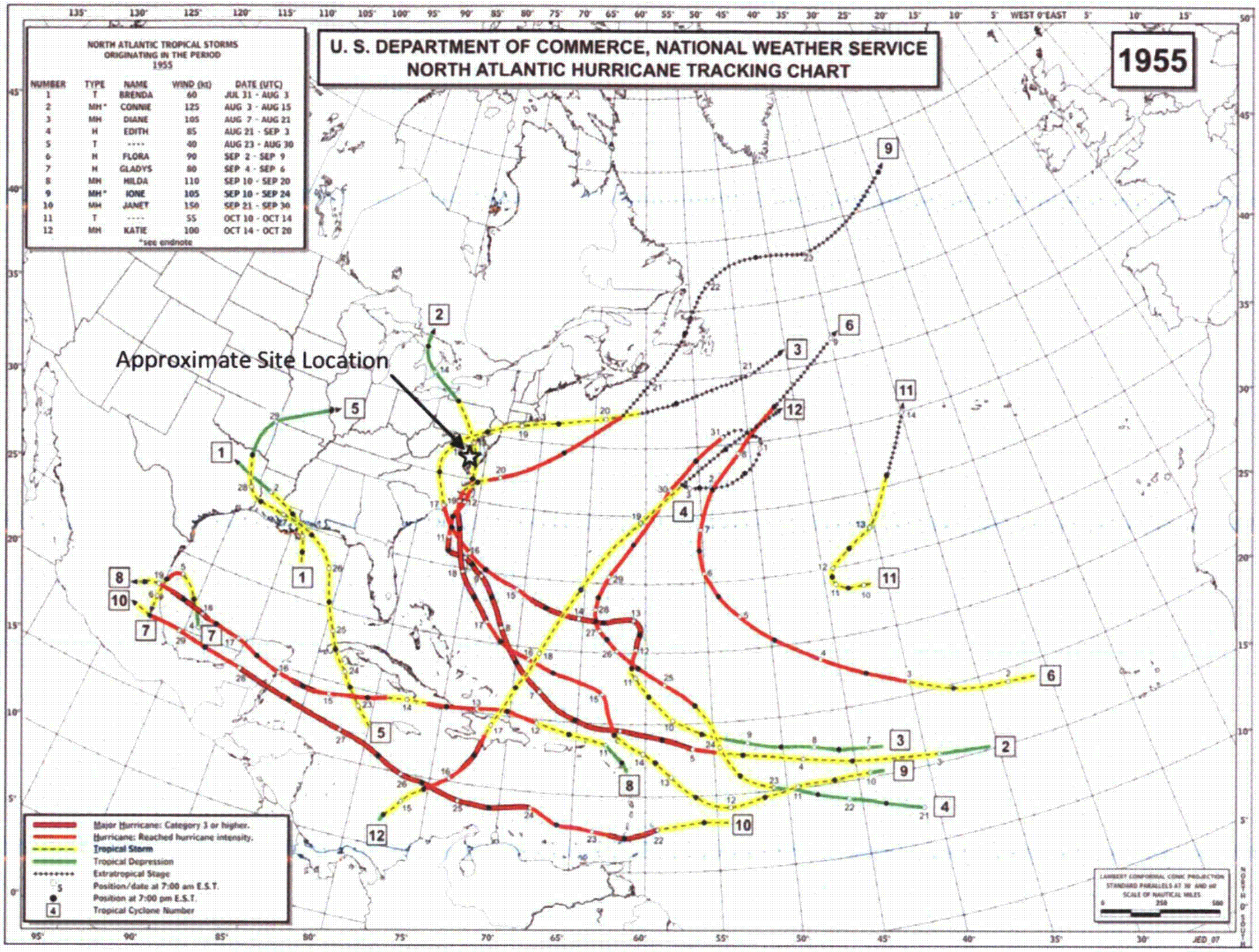


Figure 2.4-5 SLOSH Model Storm Track for PMH at CCNPP Site

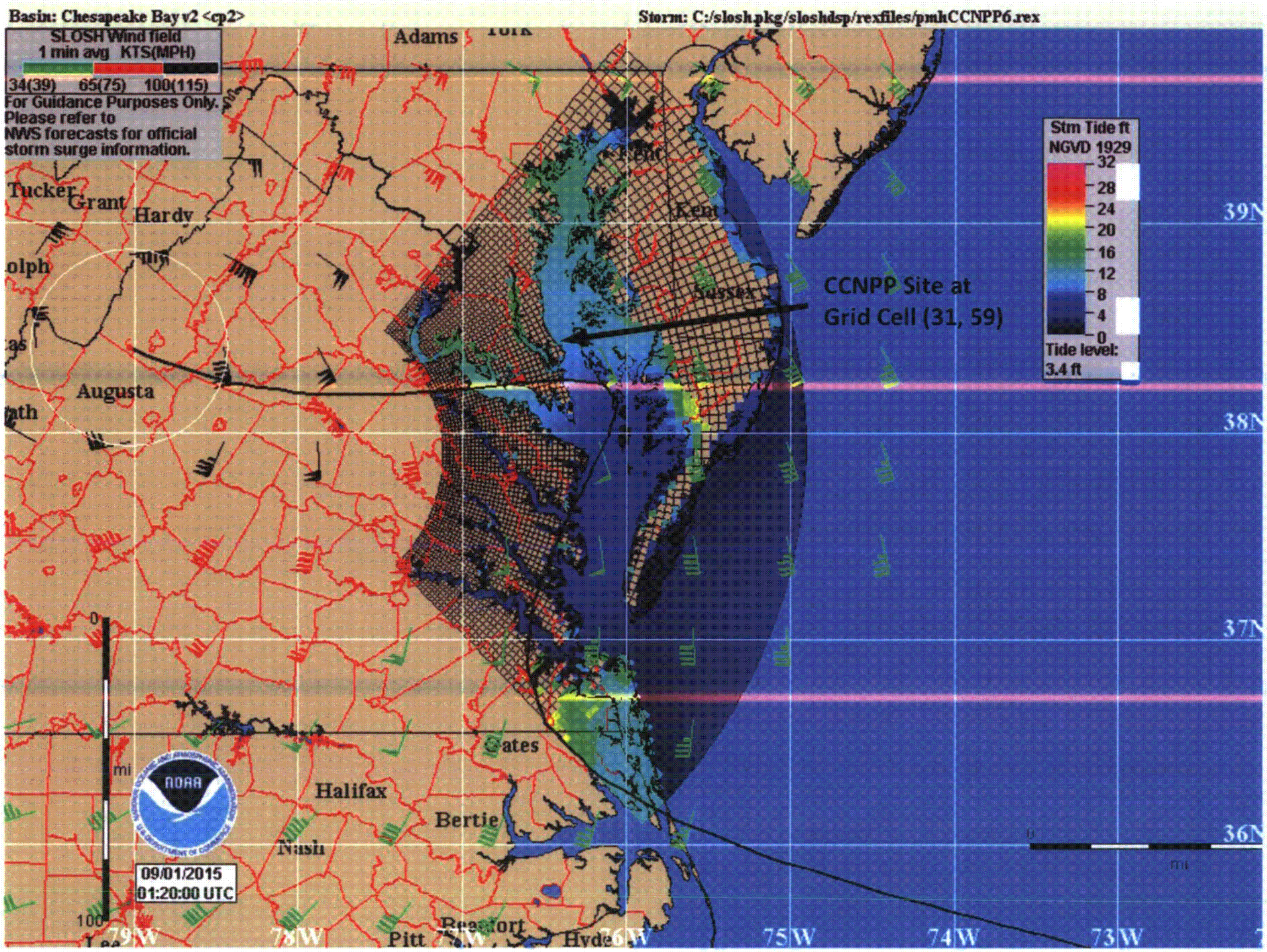


Figure 2.4-6 SLOSH Model Computational Grid for the Chesapeake Bay

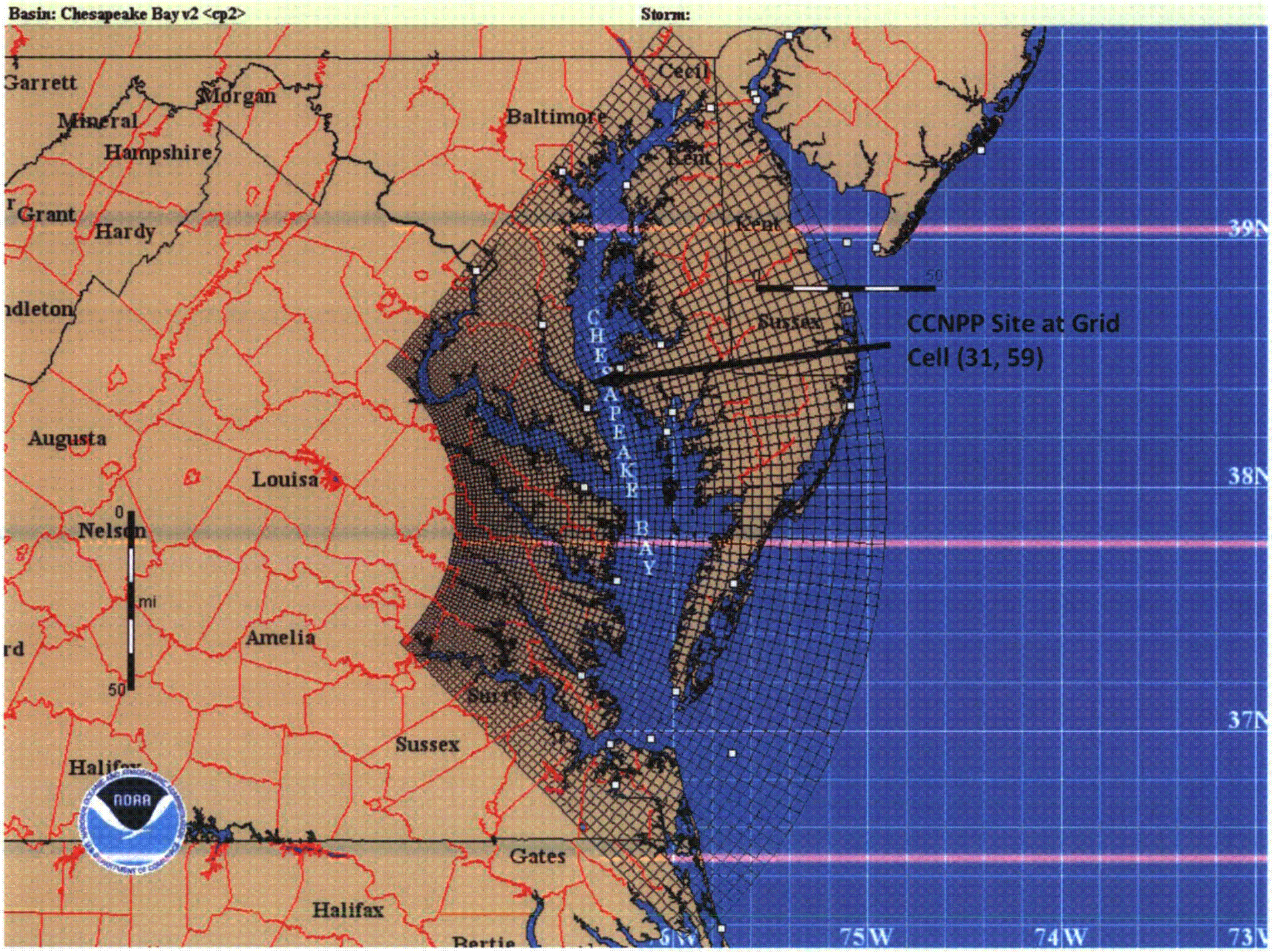


Figure 2.4-7 SLOSH Model Storm Surge Results Showing for HHA iteration No. 1

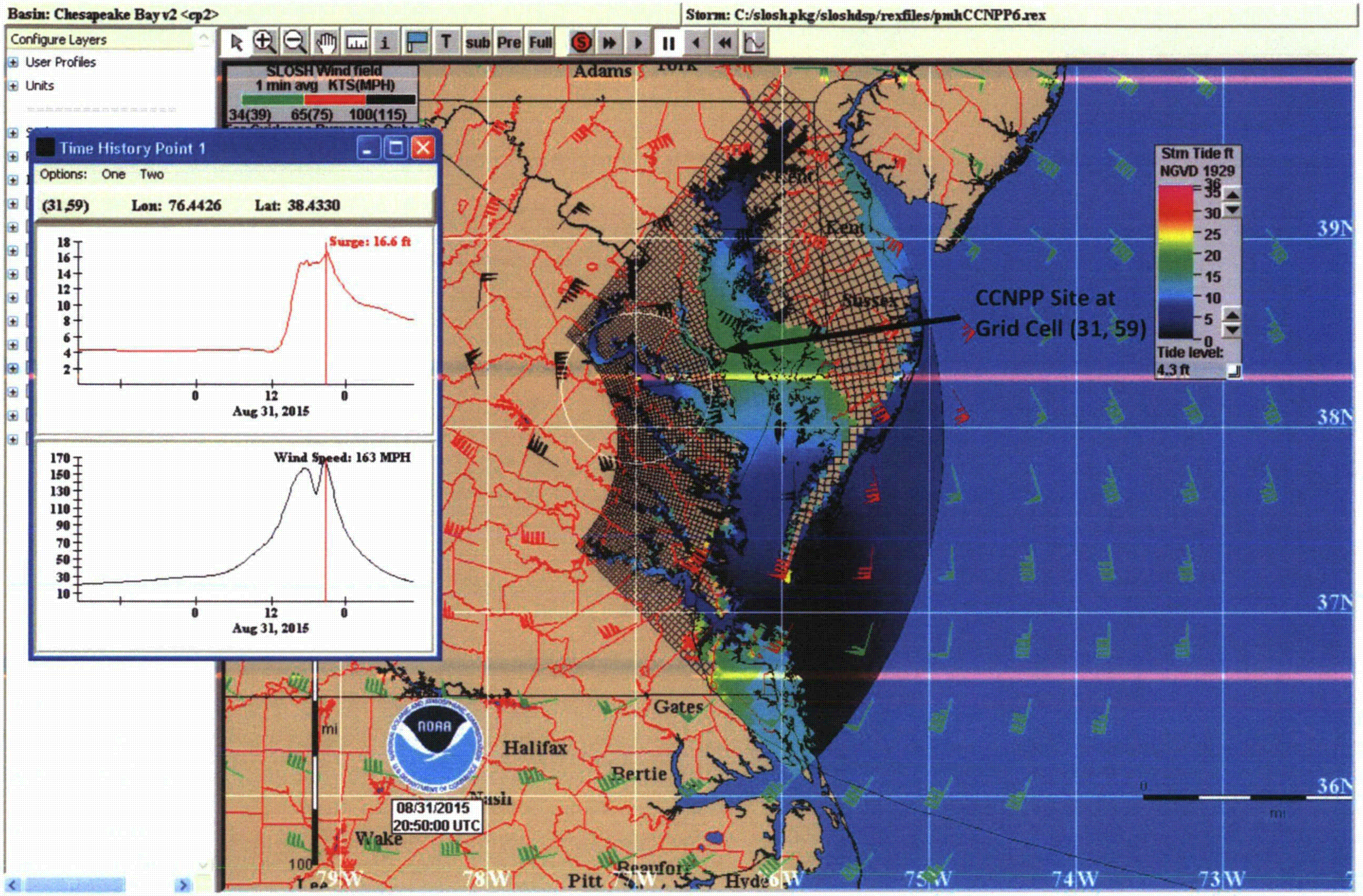
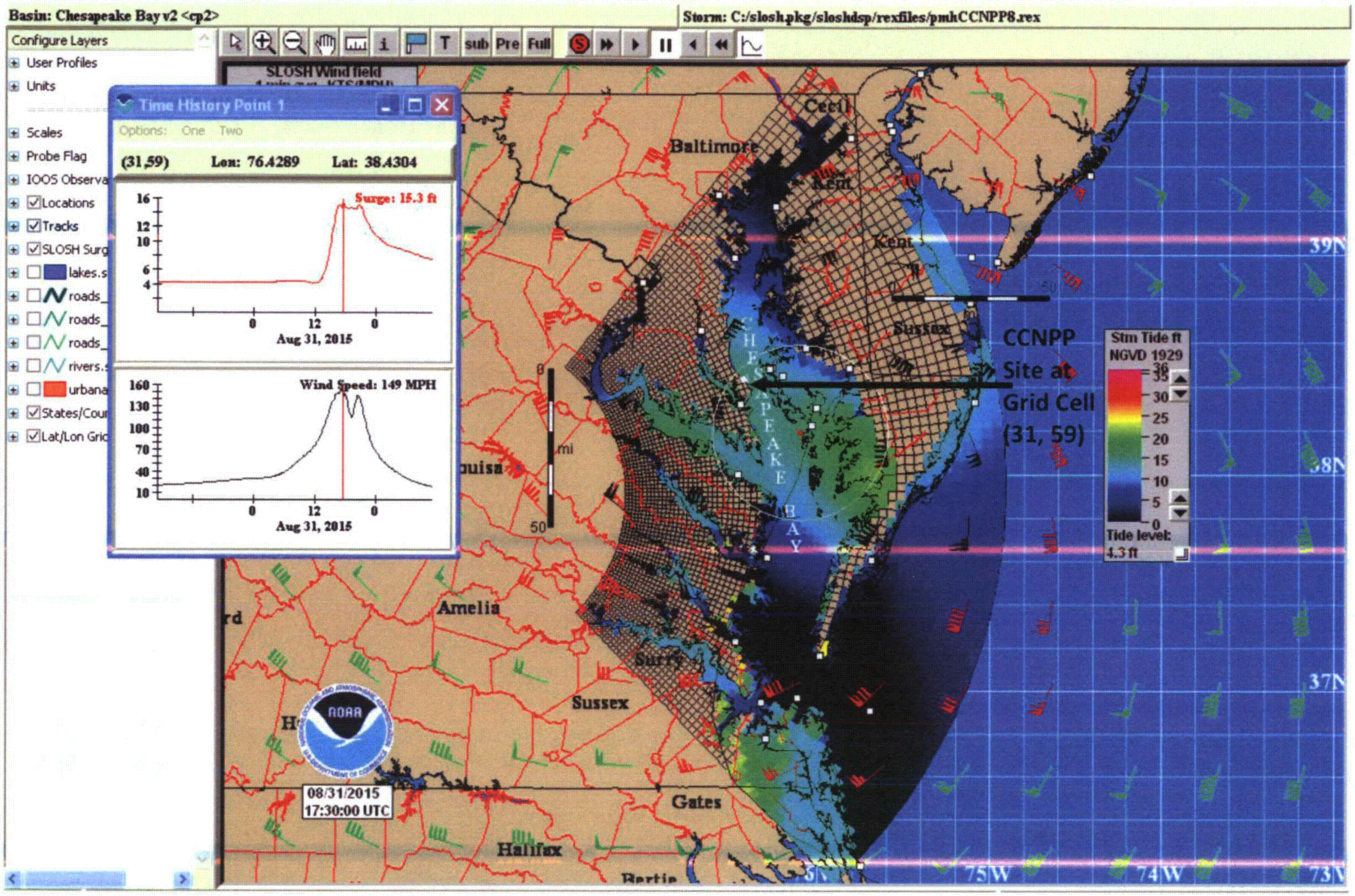


Figure 2.4-8 SLOSH Model Storm Surge Results Showing the Maximum Surge Level for HHA Iteration No. 2



2.5 Seiche

The effects of seiche-induced flooding in the Chesapeake Bay and its impact to the CCNPP Units 1 & 2 safety-related and important-to-safety facilities were evaluated based on a review of the bay's natural and forcing frequencies (or periods), as available from literature.

Wang (Reference 2.5-1) reported that subtidal water level variation in the Chesapeake Bay caused by local wind forcing has a time scale of 2 to 3 days. This behavior of water level variation is similar to that observed by Wang (Reference 2.5-2) and Wang and Elliott (Reference 2.5-3), which suggests that the subtidal sea level fluctuations were seiche driven by longitudinal wind at 2- to 3-day time scales.

Wang (Reference 2.5-1) used measured velocity, salinity, and water level variation in the Chesapeake Bay over a 1-month period in 1975 and correlated the data with measured meteorological data. Two measurements of salinity profiling along the axis of the Chesapeake Bay were also used. Wang (Reference 2.5-2) used a 1-year measurement (1974-1975) of water level variations at six locations in the bay and meteorological observations at two locations. Wang and Elliott (Reference 2.5-3) used water level variation and meteorological data in the Chesapeake Bay measured over a 2-month period in 1974 to investigate the effect of wind forcing on the Chesapeake Bay water level. Wang and Elliott (Reference 2.5-3) also used additional near-bottom current measurement in the Potomac River and 3-day current profiling data during a storm event. Measured water levels in these studies were filtered to remove high frequency tidal fluctuations prior to using in the analysis. The low-pass filtered data represented the subtidal fluctuations in the water level and velocity measurements. These studies identified variation in subtidal fluctuations with varying time scales and hypothesized various physical processes associated with these time scales. The shortest of these time scales with a 2- to 3-day period was associated with local wind forcing that was hypothesized to initiate a seiche motion in the bay.

Wang (Reference 2.5-1, Reference 2.5-2) estimated the resonance period (T) using the classic quarter wavelength theory for semi-enclosed bays:

$$T = 4L(gh)^{-0.5} \qquad \text{Eq. 2.5-1}$$

where L is the length of a semi-enclosed bay, g is gravitational acceleration, and h is the water depth. The resonance period was estimated to be 1.38 days based on a bay length of 270 km (168 mi) and an average water depth of 8.4 m (27.6 ft) used in Wang (Reference 2.5-1). If the bay length of 280 km (174 mi) and average water depths of 8.0 m (26.3 ft) from Wang (Reference 2.5-2) were used, the resonance period would be 1.46 days. However, this estimate adopts a simplistic approach, which is highly dependent on the choice of bay water depth and length and does not account for irregularities in shoreline position, spatial water depth variability, and complex tributary channel interactions.

Using a depth-integrated two-dimensional numerical model, Zhong et al. (Reference 2.5-4) simulated water level variations in the Chesapeake Bay in response to various forcing factors that govern tide and flux in the bay. The resonance period of about 2 days, as obtained by Zhong et al. (Reference 2.5-4), is consistent with the seiche period identified in Wang (Reference 2.5-1, Reference 2.5-2) and Wang and Elliott (Reference 2.5-3).

Zhong et al. (Reference 2.5-4) also concluded that the Chesapeake Bay is a highly dissipative system, with the subtidal energy dissipated through frictional resistance in its shallow water areas. As a result, it has a low amplitude gain between the bay mouth (CBBT) and head (Baltimore) (amplitude ratio of about 1.42) at the resonance period of 2 days, compared to amplitude gains in other semi-enclosed bay systems. The amplitude gain is higher when both offshore sea surface fluctuations and local wind impacts were considered at the same time (about 2.0) compared to amplitude gain from tidal actions alone. Zhong et al. (Reference 2.5-4) also observed a shorter resonance period of 1.6 days in response to an arbitrary sea level rise of 1.0 m (3.3 ft) in the bay, which, as they concluded, showed an increase in the amplitude ratio (1.5). The amplitude ratio between CBBT and Solomons Island is smaller than the amplitude ratio between CBBT and Baltimore.

In addition to the main resonant motion in the bay along its main axis caused by the northeasterly, northwesterly, or southwesterly winds, lateral (easterly) winds near the entrance of the bay may also generate a seiche motion with a smaller resonance period of about 1.6 days (Reference 2.5-5, Reference 2.5-6). Although such optimal wind direction directly affects only the lower bay, it was postulated that the wind-driven current could trigger a mild resonant seiche motion in the rest of the bay.

While seiche oscillations are observed in the Chesapeake Bay due to wind and ocean current–induced forcing, these oscillations do not appear to coincide with hurricane storm surge events. Storm surge hydrographs at different locations in the Chesapeake Bay during the passage of Hurricane Isabel show that the storm surge gradually rises to its peak and then gradually reverts back to the normal water level when the influence of the hurricane is diminished (Reference 2.5-7). Wang (Reference 2.5-2) and Chuang and Boicourt (Reference 2.5-6) indicate the period for hurricane-induced oscillations in the bay is about 4-10 days.

A review of storm surge in the Chesapeake Bay reveals that the upper bay experiences the maximum storm surge impact when a hurricane travelling in the northerly direction passes by west of the bay. As the storm surge from this hurricane enters the Chesapeake Bay, the water level at the lower Chesapeake Bay will increase (set-up) due to the passage of the surge wave. At the same time the water level at the upper Chesapeake Bay is likely to decrease (set-down) due to the counterclockwise pattern of the wind field. Similarly, when the storm surge approaches the upper Chesapeake Bay combined with a southerly wind, the water level in the upper Chesapeake Bay will increase, and the water level in the lower Chesapeake Bay will eventually decrease.

Except for this variation in water level during the passage of a hurricane, historical records in the Chesapeake Bay do not show any significant oscillations affecting the storm surge levels in the Chesapeake Bay (Reference 2.5-8). Once the hurricanes move beyond the Chesapeake Bay region, small oscillations have been observed. However, these oscillations gradually diminish after the passage of the storm surges.

Because the PMSS constitutes the design basis flood elevation at the CCNPP Units 1 & 2 site, as described in Section 2.4, and the natural period of the bay (about 2 days) is considerably smaller than the typical periods of hurricane storm surge forcing (about 4 to 10 days), it is unlikely that any significant seiche events could be generated and coincide with a hurricane storm surge event. Similarly, the large difference in the natural period of the bay and the periods of seismic motions precludes major seismic-induced seiche events from taking place in the bay.

2.5.1 Conclusions

Seiche oscillations are observed in the Chesapeake Bay due to wind and ocean current–induced forcing. However, magnitude of these oscillations is small and the period does not coincide with the natural period of the bay. Also, the PMSS has a longer period of oscillation than the natural period of the bay and a resonance is unlikely. It is therefore concluded that seiche flooding will not impact any safety-related or important-to-safety facilities of the CCNPP Units 1 & 2 plant.

2.5.2 References

2.5-1 Wang, D.-P., *Wind Driven Circulation in the Chesapeake Bay*, Winter 1975, Journal of Physical Oceanography, Volume 9, pages 564-572, 1979.

2.5-2 Wang, D.-P., *Subtidal Sea Level Variation in the Chesapeake Bay and Relations to Atmospheric Forcing*, Journal of Physical Oceanography, Volume 9, pages 413-421, 1979.

2.5-3 Wang, D.-P., and A.J. Elliott. *Non-Tidal Variability in the Chesapeake Bay and Potomac River: Evidence for Non-Local Forcing*, Journal of Physical Oceanography, Volume 8, pages 225-232, 1978.

2.5-4 Zhong, L., M. Li, and M.G.G. Foreman. *Resonance and Sea Level Variability in Chesapeake Bay*, Continental Shelf Research, Vol. 28, pages 2565-2573, 2008.

2.5-5 Valle-Levinson, A., K.-C. Wong, and K.T. Bosley. *Observations of the wind-induced exchange at the entrance to Chesapeake Bay*, Journal of Marine Research, Volume 59, pages 391-416, 2001.

2.5-6 Chuang, W.-S., W.C. Boicourt. *Resonant Seiche Motion in the Chesapeake Bay*, Journal of Geophysical Research, Volume 94, Number C2, pages 2105-2110, 1989.

2.5-7 Hovis, J., W. Popovich, C. Zervas, J. Hubbard, H.H. Shih, and P. Stone. *Effects of Hurricane Isabel on Water Levels*, Data Report, NOAA Technical Report NOS CO-OPS 040, National Oceanic and Atmospheric Administration, Silver Spring, Maryland, 120 pages, 2004.

2.5-8 UniStar Nuclear Services, LLC. *Calvert Cliffs Nuclear Power Plant Unit 3 Combined License Application*, Final Safety Analysis Report, Rev 8, March 2012.

2.6 Tsunami

2.6.1 Probable Maximum Tsunami

Tsunami-induced flooding hazard at the CCNPP site has been evaluated as part of the COLA for CCNPP Unit 3 (Reference 2.6-11). The same approach and methodology are applied for the tsunami flooding reevaluation of Units 1 & 2, supplemented with information from recent tsunami literature and databases and following the guidelines in NUREG/CR-6966 (Reference 2.6-27).

Tsunami events that could affect CCNPP Units 1 & 2 would be caused by local or distant geo-seismic activities. While local tsunamigenic source mechanisms could include submarine or subaerial landslides in the Chesapeake Bay, distant tsunami sources would include submarine fault displacements, submarine landslides, or volcanic eruptions in the Atlantic Ocean. Because the CCNPP site is most likely to be affected by tsunamis generated in the Atlantic Ocean, potential tsunami sources in the Atlantic Ocean were considered when tsunami effects on the CCNPP site were evaluated.

The potential of a subaerial landslide near the site was assessed with geological maps, topographic maps, and CCNPP site reconnaissance. Along the western shoreline of the Chesapeake Bay, slope failure has occurred and appears to be caused by erosion of the base of the cliffs that reach an elevation of about 100 ft NGVD 29. The shallow water depth in the near shore area of the cliffs precludes any credible tsunami generated from a hypothetical cliff failure mechanism that would affect the safety-functions of the plant. Across from the CCNPP site, the eastern shore of the Chesapeake Bay, as shown on USGS topographical maps (References 2.6-18 and 2.6-19), consists of nearly flat terrain, primarily of low and wide tidal flats with a maximum topographic elevation near the eastern shoreline of approximately 7.5 ft NGVD 29. It is therefore evident that the eastern shore of the Chesapeake Bay, opposite the CCNPP site, would not be subject to slope failure. If subaerial landslides near the site were to happen, they would not trigger local tsunami-like waves in the Chesapeake Bay due to the shallow water depths near the shore.

Several tsunami studies identify tsunamigenic sources in the Atlantic Ocean and estimate tsunami impacts on the east coast of the United States. Based on these studies and historical tsunami events recorded along the east coast of the United States, discussed in Section 2.6.2, potential tsunamigenic sources that could affect the coastal region near the entrance of the Chesapeake Bay are identified as:

- A potential submarine landslide off the coast of Norfolk, VA. Submarine landslides in this area along the Virginia and North Carolina continental shelf could produce tsunami amplitudes of 6.6 to 13 ft along beaches from North Carolina to New York (References 2.6-2 and 2.6-21).
- Large tsunamis in the Atlantic Ocean generated by submarine landslides and volcanic flank failure near La Palma in the Canary Islands, which could be triggered by volcanic eruptions. Such tsunamis could propagate across the Atlantic Ocean and reach the U.S. east coast with tsunami amplitudes less than 10 ft (References 2.6-16 and 2.6-6).

- Tsunamis due to submarine fault displacement or volcanic activities near the Caribbean Islands. This area has a subduction zone where the North American Plate (moving west) meets the Caribbean Plate (moving east) (Reference 2.6-9). The maximum tsunami amplitude predicted near Newport News, VA from the Caribbean sources is about 3.1 ft (Reference 2.6-15).

Other potential far-field tsunami sources in the Atlantic Ocean include an active subduction zone near the South Sandwich Islands in the South Atlantic Ocean (Reference 2.6-9), and earthquake zones off the coast of Newfoundland, Canada. Small tsunami amplitude of approximately 0.2 ft near Newport News, VA is predicted from the south Atlantic sources (Reference 2.6-15). Observations at Atlantic City, NJ indicated a tsunami amplitude of about 2.2 ft due to the 1929 earthquake near Grand Banks, Canada (Reference 2.6-12). Tsunami sources from these other areas were excluded when the Probable Maximum Tsunami (PMT) was estimated because of their small intensity.

The PMT amplitude at the CCNPP site was computed for the three potential tsunami sources described in Section 2.6.3. The maximum simulated amplitude at the CCNPP site was obtained from the postulated submarine landslide at the Virginia-North Carolina continental shelf off the coast of Norfolk, VA, and was estimated to be 1.71 ft above the antecedent water level.

2.6.2 Historical Tsunami Record

Selected historical tsunami events (those classified as “probable” or “definite” tsunamis by Reference 2.6-12) with recorded runups in the eastern United States and Canada from 1755 to 2012 are shown in Table 2.6-1 (Reference 2.6-12), and significant events are discussed below.

Figure 2.6-1 shows the location of geo-seismic tsunami source generators in the Atlantic Ocean. From Figure 2.6.1 and Table 2.6-1, five potential tsunamigenic sources could be identified that could affect the CCNPP site. These sources are:

- A submarine landslide in the continental shelf along the east coast of the United States
- Tsunamigenic sources along the Atlantic east coast including those near the Portuguese coast and Canary Islands
- A marginal boundary subduction zone near the Caribbean Islands
- Earthquake zones in the northern Atlantic Ocean primarily near Newfoundland, Canada
- A subduction zone near the South Sandwich Islands in the southern Atlantic Ocean

A review of the historical records and published studies (Reference 2.6-24) indicates that the most severe tsunamis likely to impact the Chesapeake Bay area of the U.S. east coast would be due to the first three sources listed above.

A submarine landslide from the continental shelf along the east coast of the United States is known to have occurred in the late Pleistocene era. This slide is known as the Albemarle-Currituck slide with an estimated volume displacement of 36 cu mi. It is estimated that the size of the generated tsunami wave was several meters at the coast line, roughly equivalent to the height of a storm surge associated with a Category 3 or Category 4 hurricane. However, a large submarine landslide from the continental shelf off the east coast of the United States is a rare event on a human time scale (Reference 2.6-2).

The Chesapeake Bay bolide impact that was found in an exploration of the Chesapeake Bay area by the USGS (Reference 2.6-20) likely generated a paleo-tsunami about 35 million years ago. No estimate is available as to the size of the generated tsunami wave and no tsunami deposits attributable to this tsunami have been found. Also, information gathered during a literature search and geologic reconnaissance indicates that there is no evidence of a paleo-tsunami or paleo-tsunami deposits due to geo-seismic events in the CCNPP site region. A literature search reveals no historical or geologic evidence of seismically-generated seiches in Chesapeake Bay.

The U.S. National Seismograph Network (USNSN), operated by the U.S. Geological Survey, is part of a Global Seismic Network that monitors seismic (earthquake) activity around the world. These networks are able to detect seismic events that are capable of resulting in a tsunami. Soon after an earthquake occurs, seismic activity is recorded by the seismographs, and beamed to a satellite and to the USNSN home base in Colorado, where it is analyzed and warnings (if needed) are issued (Reference 2.6-9). Additionally, NOAA has recently deployed 39 Deep-ocean Assessment and Reporting of Tsunami (DART®) stations at sites in regions with a history of generating destructive tsunamis as part of the NOAA Tsunami Program (Reference 2.6-25). Seven DART stations are located in the Atlantic Ocean.

The most notable historic tsunamis in the Atlantic Ocean that could affect the coastal region at the entrance of the Chesapeake Bay are summarized in the following sections. These tsunamis were generated from the tsunami source generators described above.

2.6.2.1 Tsunami from 1755 Lisbon, Portugal Earthquake

The most notable Atlantic Ocean tsunami that affected the east coast of the United States was generated off the coast of Portugal in 1755. The tsunami was generated at the Gorringe Bank, approximately 124 mi from the Portuguese coast, due to a displacement in the submarine fault. The highest runup from this tsunami was estimated to be approximately 100 ft near Lagos, Portugal. At Lisbon, Portugal a runup height of 40 ft was reported (Reference 2.6-12). The maximum tsunami amplitude along the east coast of the United States was estimated to be approximately 10 ft by numerical simulation (Reference 2.6-7).

2.6.2.2 Tsunami from 1918 Puerto Rico Earthquake

The 1918 earthquake near Puerto Rico had a magnitude of 7.3 in the moment magnitude scale (Mw). It triggered a tsunami with reported runup heights up to 20 ft along the Puerto Rico coast. The earthquake epicenter was located 9.4 mi off the northwest coast of the island within the Puerto Rican Trench and the tsunami was caused by submarine fault displacement. Tsunami amplitude of approximately 0.2 ft was recorded at Atlantic City, NJ located northeast of the CCNPP site (Reference 2.6-12).

2.6.2.3 Tsunami due to 1929 Earthquake at Grand Banks, Newfoundland, Canada

The 1929 earthquake had a moment magnitude (Mw) of 7.2 (Reference 2.6-12) and generated one of the most devastating tsunamis in the northern part of the North American east coast. However, destruction due to this tsunami was mostly confined within the Newfoundland coast. The epicenter of the earthquake was located near the mouth of Laurentian Channel, south of the Burin Peninsula and on the south coast of Newfoundland. The earthquake triggered an underwater landslide that generated a tsunami with a runup height of 88.6 ft at the Burin Peninsula. Water level records at Atlantic City, NJ show that the maximum tsunami amplitude at this location from the 1929 Grand Banks tsunami was 2.2 ft (Reference 2.6-12).

2.6.3 Tsunami Source Generator Characteristics

The tsunami analysis for the CCNPP site was performed in the Chesapeake Bay using tsunami propagation models that considered both nonlinear shallow water equations, including bottom friction and linear shallow water equations without bottom friction. The tsunami waves at the entrance of the Chesapeake Bay were characterized based on the results from published studies on the Atlantic Ocean tsunamis.

Three potential tsunami-generating sources were selected to estimate tsunami heights at the CCNPP site. These sources are selected based on the historical tsunami sources discussed in Section 2.6.2, using the locations shown in Figure 2.6-1 and from published studies. The hypothetical characteristics of these tsunami-generating sources and the tsunami wave characteristics at the entrance of the Chesapeake Bay are as follows:

2.6.3.1 Norfolk Canyon Submarine Landslide, Virginia

- Source: Submarine landslide of continental shelf off the coast of southern Virginia and North Carolina (References 2.6-2 and 2.6-21).
- Sliding Scenario: 36 cu mi of material running out at a speed of 49 to 115 ft/s for 55 minutes (Reference 2.6-21 and 2.6-2).
- Tsunami Parameters: Maximum tsunami amplitude of 13 ft at the Chesapeake Bay entrance with a period of 3,600 seconds.

It is suggested (Reference 2.6-2) that the presence of a system of en echelon cracks along the edge of the continental shelf, just north of the Pleistocene Albemarle-Currituck landslide, likely indicates an initial stage of a large scale slope failure. Because large magnitude earthquakes do not occur in the east coast of the United States or in the vicinity of the Norfolk Canyon, gas hydrate release and interglacial changes are possible triggering mechanisms for the landslide (Reference 2.6-2). Ward (Reference 2.6-21) estimated the landslide parameters of the Norfolk Canyon landslide based on assumptions that the size and volume of a potential landslide would be the same as the mapped debris field of the Pleistocene Albemarle-Currituck landslide. The slide front would advance at the same speed as that of the Storegga landslide in the northern

Atlantic Ocean. Ward (Reference 2.6-21) used these landslide parameters to perform model simulation for the submarine landslide-induced tsunami. While tsunami amplitude at the entrance of the Chesapeake Bay is selected from the model simulation results of Ward (Reference 2.6-21), the tsunami period is estimated based on recorded tsunami periods along the east coast of the United States. The longest wave period recorded from the 1929 Grand Banks submarine landslide-generated tsunami was 40 minutes, recorded at Halifax, Nova Scotia, Canada (Reference 2.6-12). Considering the proximity of the Chesapeake Bay entrance to the tsunami source location, similar to that of Halifax to the Burin Peninsula, a tsunami wave period of similar time span could be approximated. However, because the shallow and wide continental shelf would likely cause the short-period component waves to dissipate before reaching the Chesapeake Bay entrance, the tsunami wave period at the entrance of the Chesapeake Bay was conservatively selected to be 60 min (3,600 sec).

A 2008 USGS study by the Atlantic and Gulf of Mexico Tsunami Hazard Assessment Group (Reference 2.6-24) performed regional modeling studies of the Albemarle-Currituck landslide. Various modeling scenarios were tested to study the effects of landslide parameters (slide duration, simultaneity of scarp failures) and friction factors on the resulting tsunami. Modeling scenarios most conducive to tsunami generation predicted tsunamis with amplitudes exceeding 25 m (82 ft) near the source (Reference 2.6-24, page 178), but also demonstrated the attenuation effects of the shallow continental shelf. While the Chesapeake Bay is outside of the model domain, the study results showed that at the northwest corner of the model boundary, approximately 70 km (43.8 mi) north of Kitty Hawk, NC, simulated maximum wave amplitudes were on the order of 4 m (13 ft) in the worst-case scenarios (Reference 2.6-24, page 178). Because the mouth of the Chesapeake Bay is further from the source, geometric spreading and energy dissipation due to bottom friction and wave breaking at the continental shelf would diminish the wave further still, suggesting that the 4 m (13 ft) amplitude chosen for the present modeling scenarios is conservative.

2.6.3.2 La Palma in Canary Islands

- Source: Lateral collapse of flank of Cumbre Vieja Volcano on La Palma in Canary Islands (References 2.6-16, 2.6-6, and 2.6-21).
- Sliding Scenario: 120 cu mi of material running out 37.3 mi at a mean speed of 328 ft/s (Reference 2.6-21).
- Tsunami Parameters: Maximum tsunami amplitude of 10 ft at the Chesapeake Bay entrance with a period of 3,600 seconds.

Ward (Reference 2.6-21) postulated the sliding scenario and the tsunami source parameters of the Cumbre Vieja volcanic flank failure based on geological evidence of the area, shape of the previous La Palma slide, and past collapses of similar volume elsewhere in the Canaries.

Although Mader and Ward used the same source and sliding scenarios for the Cumbre Vieja volcano flank failure, they obtained vastly different tsunami amplitude distribution along the east coast of the United States. While Mader suggests wave amplitude of 10 ft along the east coast of the United States, Ward suggests a maximum tsunami amplitude of between 10 ft and 25 ft. Pararas-Carayannis indicated that parameters for initial tsunami generation from the postulated landslide and the initial wave properties are incorrectly addressed in Ward, thereby greatly exaggerating the tsunami amplitude along the U.S. coast.

Pararas-Carayannis also pointed out that the initial tsunami period for the postulated landslide scenario would be small producing an intermediate wave condition rather than a shallow water wave condition. The tsunami database of the NOAA National Geophysical Data Center reveals that the maximum tsunami wave period (period of the first wave cycle) ever recorded along the U.S. coast was 100 min at Sitka, Alaska, resulting from the 1938 Shumagin Island, Alaska, earthquake tsunami (Reference 2.6-12). Along the east coast of the United States, the maximum wave period of 30 min was recorded at Charleston, South Carolina, from the 1929 Grand Banks tsunami (Reference 2.6-12). Considering that the La Palma tsunami is postulated for a landslide generated tsunami and the tsunami wave would travel across the Atlantic Ocean where the short-period wave components would be dissipated, the selected wave period of 3,600 seconds is considered conservative.

The selected tsunami amplitude for this tsunami is nearly the same as the simulated tsunami amplitude from the 1755 Lisbon tsunami (Reference 2.6-7). The selected wave period of 3,600 seconds also would provide a representative tsunami condition at the entrance of the Chesapeake Bay for a tsunami event of similar magnitude from this source.

2.6.3.3 Haiti in Caribbean Islands

- Source: Earthquake induced fault displacement (Reference 2.6-15)
- Displacement Scale: Length of 662 mi, width of 298 mi, and peak displacement of 30 ft
- Tsunami Parameters: Maximum tsunami amplitude of about 3.1 ft at the Chesapeake Bay entrance with a period of 5,200 seconds.

Tsunami amplitude and period are obtained from the simulated tsunami hydrograph near Newport News, VA as presented in Figure B-13 of NUREG/ CR-1106 (Reference 2.6-15). The wave period was estimated as the period of the first wave cycle (peak to peak). In NUREG/CR-1106, a linear shallow water wave model was used for the simulation of this tsunami.

2.6.4 Tsunami Analysis

Tsunami simulations were performed within the Chesapeake Bay using a two-dimensional, depth-averaged numerical model, TSU_NLSWE, Version 1.0, as part of the CCNPP Unit 3 COLA (Reference 2.6-11). Because the water depth in the Chesapeake Bay is relatively shallow compared to the wavelength and amplitude of incident tsunamis, nonlinearity of waves and bottom friction effects are considered in the model formulation. The model is capable of simulating wave propagation in shallow waters using the nonlinear shallow water wave equations with bottom friction (NLSWE model), where the bottom friction term is taken as a function of the fluxes in the two horizontal directions and Manning's roughness coefficient. The model can also simulate wave propagation using the linear shallow water wave equations without bottom friction (TSU model).The models use a leap-frog finite-difference scheme to numerically solve the governing partial differential equations. Tsunami simulations were conducted using both linear and nonlinear simulation models for both local and distant tsunami generators. Results were then compared to obtain the bounding tsunami amplitude at the CCNPP Units 1 & 2 site.

2.6.4.1 Governing Equations

The governing equations used in the TSU_NLSWE model are shown below (Reference 2.6-3, Reference 2.6-4):

$$\frac{\partial \eta}{\partial t}+\frac{\partial P}{\partial x}+\frac{\partial Q}{\partial y}=0 \qquad \text{Eq. 2.6-1}$$

$$\frac{\partial P}{\partial t}+\frac{\partial}{\partial x}\left(\frac{P^2}{h}\right)+\frac{\partial}{\partial y}\left(\frac{PQ}{h}\right)+gh\frac{\partial \eta}{\partial x}+\frac{gn^2}{h^{7/3}}P\sqrt{P^2+Q^2}=0 \qquad \text{Eq. 2.6-2}$$

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial y}\left(\frac{Q^2}{h}\right)+\frac{\partial}{\partial x}\left(\frac{PQ}{h}\right)+gh\frac{\partial \eta}{\partial y}+\frac{gn^2}{h^{7/3}}Q\sqrt{P^2+Q^2}=0 \qquad \text{Eq. 2.6-3}$$

Where:

- η represents the free surface displacement from still-water level;
- P and Q are the depth-averaged volume fluxes in the x and y directions, respectively;
- t is time;
- g is the acceleration of gravity;
- h is the water depth below the still-water level; and
- n is Manning's roughness coefficient.

Linearization of the governing equations in the TSU model option neglects the effect of the convective terms in the equations of motion (second and third terms in Eq. 2.6-2 and Eq. 2.6-3). Additionally, bottom friction effects (the last term in Eq. 2.6-2 and Eq. 2.6-3) are neglected in the TSU model option. As a result, simulation results using the TSU model option provide a conservative upper bound solution for tsunami propagation in a shallow water environment such as the Chesapeake Bay.

A leap-frog, finite-difference scheme is employed to solve both the nonlinear and linear shallow water equations on a staggered grid in time and space, as shown in Figure 2.6-2. The equation of the continuity is approximated with an explicit, central-difference scheme. Approximation of the linear terms in the equations of motion also uses a central-difference scheme. An upwind scheme is applied to approximate the convection terms in the equations of motion. An implicit scheme is utilized for the bottom friction terms, as the friction term becomes a source of instability if it is represented using explicit scheme (References 2.6-3 and 2.6-4).

Discretization of linear governing equations in finite-difference form generates numerical dispersion, which is a form of numerical error (Reference 2.6-23). This numerical dispersion can be used as a surrogate for the physical dispersion neglected in the linear form of the shallow water equation by appropriately selecting the computational time step and grid spacing. For a fixed grid model with varying water depth, the accuracy of the linear model, therefore, is limited because of inherent model requirements of different grid sizes for different water depths. Yoon (Reference 2.6-23) overcomes this limitation by separately calculating the computational grid spacing (termed as "the hidden grid spacing") at each time step, based on the dispersion criterion provided as input to the model. The computations are then performed on the hidden grids. At the end of each time step, the results are interpolated back at user-specified grid locations from the hidden grids. This technique has shown a considerable improvement in the accuracy of the solution of linear shallow water equations. This hidden grid approach was employed in developing the TSU_NLSWE model. The finite-difference schemes used in the models are represented in Figure 2.6-3 and Figure 2.6-4.

2.6.4.2 Model Simulations

Verification of the TSU_NLSWE code was performed by comparing model simulation results against an analytical solution of Gaussian hump propagation developed by Carrier (Reference 2.6-1). Comparison of both the linear and nonlinear numerical solutions against the analytical solution resulted in good agreement for deep water. The model was also tested for a constant water depth of 10 m (32.8 ft) and the grid size of 360 m by 360 m (1181 ft by 1181 ft), which represents the relatively shallow depths of the Chesapeake Bay. The shallow water results for the linear simulation showed good agreement with the analytical solution. For the nonlinear simulation, a qualitative comparison showed that the numerical results appropriately predicted the nonlinear wave deformation and dissipation by bottom friction. A quantitative comparison for the nonlinear option was not possible as no analytical solution is available for the general nonlinear case. Yoon (Reference 2.6-23) applied the linear model to simulate the propagation of the 1983 Nihonkai-Chubu tsunami. This earthquake-generated tsunami originated near the east rim of the Sea of Japan and impacted the Japanese coast. A comparison of model results with measured data along the Japan Sea coast showed that satisfactory agreement was obtained using the linear model (Reference 2.6-23).

In addition, the TSU_NLSWE model was applied to simulate the tsunami event in San Francisco Bay generated by the 1964 Alaskan earthquake. The tsunami was triggered by the 9.2 magnitude (moment magnitude) Great Alaska Earthquake of 1964 that occurred on March 28, 1964 (Reference 2.6-10). At the time of occurrence, the earthquake was the most powerful seismic event in North America and the second most powerful in the recorded history of the world. Although the epicenter of the earthquake was on land, the shock waves were felt at the sea floor, causing a large submarine land displacement resulting in a large tsunami. According to NOAA records, the tsunami was generated at 3:36 a.m. Greenwich Mean Time (GMT) with an estimated maximum tsunami water height near the source of about 67 m (220 ft) above MSL, as obtained from eyewitness accounts. The tsunami wave swept through Prince William Sound, the

Kodiak Islands and propagated south to the coasts of British Columbia, Washington, Oregon and California (Reference 2.6-10).

Both linear and non-linear models were applied to simulate tsunami propagation in San Francisco Bay. The simulated maximum tsunami amplitudes were compared with tsunami water heights recorded at five locations in San Francisco Bay. The recorded maximum water heights were estimated as one-half of the difference between the maximum tsunami crest and trough after tidal variations were removed from the data. The stations where tsunami water heights are available include the NOAA tide gauges at San Francisco, Sea Cliff, Oakland and Alameda, and an eyewitness account at Sausalito. A comparison of model simulated water levels in the bay with observed data generally shows good agreement.

2.6.4.2.1 Chesapeake Bay Model Extent and Boundary Conditions

The TSU_NLSWE model code was used to provide estimates of tsunami wave heights at the CCNPP Units 1 & 2 site from both distant and local generators. Simulations were performed within the Chesapeake Bay for the three potential tsunami sources described in Section 2.6.2.3 that potentially produce the PMT flood hazard at the CCNPP site. The potential tsunamigenic sources are discussed in Section 2.6.1, and the source characteristics are described in Section 2.6.3. The characteristics of the incident tsunami waves at the entrance of the Chesapeake Bay and the computational cases for linear and nonlinear model simulation options are summarized in Table 2.6-4. Simulations were performed to obtain tsunami amplitudes for an initial water level condition corresponding to MSL at the CBBT tide gauge. The PMT was then determined considering the simulated maximum tsunami amplitude at the CCNPP Units 1 & 2 site, an antecedent water level condition based on the 10 percent exceedance high spring tide, sea level anomaly, and long-term sea level rise, as adopted in Section 2.6.5.

The Chesapeake Bay model domain extends approximately 290 km (180 mi) from near Plume Tree Point, Virginia to near the mouth of the Susquehanna River, including portions of the major river channels. Stream flow in the rivers and tidal variations were ignored in the tsunami simulations. A zero-flux condition was applied across fixed land boundaries. Flooding and drying of grid cells was not considered in the model.

Incoming tsunami amplitudes and periods for different cases, as presented in Table 2.6-4, were applied as regular sinusoidal waves along an internal boundary between Plume Tree Point, VA and Cape Charles, VA as shown in Figure 2.6-5. The internal boundary was based on implementing a radiation boundary (Reference 2.6-5). Implementation of the internal boundary, where the incoming tsunami is applied as a perturbation, requires that all outgoing waves are absorbed at the external boundaries of the model without reflection. This requirement was enforced by implementing non-reflective, absorbing layers (defined as "sponge" layers) for 10 grid lines along the boundaries. The procedure for defining a sponge layer is described by Larsen (Reference 2.6-5). All outgoing waves, in terms of surface displacement and volume fluxes, are absorbed over the thickness of the sponge layers. Sponge layers were also implemented for the Patuxent, Potomac, Rappahannock and York Rivers along the west and Pocomoke Sound along the east boundary of the domain. Locations of the sponge layers are also shown in Figure 2.6-5. Boundaries for the other rivers, including the northern end of the Chesapeake Bay, were considered closed (no-flux) and fully reflective.

2.6.4.2.2 Chesapeake Bay Bathymetry and Model Grid

Bathymetric data for the Chesapeake Bay were obtained from the NOAA National Ocean Services program (Reference 2.6-13). The digital elevation model (DEM) data have a spatial resolution of 30 m by 30 m (98.4 ft by 98.4 ft) with coverage in 7.5-minute by 7.5-minute blocks (Reference 2.6-13). The depth soundings used to generate the bathymetry were surveyed over a period from 1859 to 1993. Thirty-six surveys were conducted in the 1859-1918 period, 37 in the 1930s, 91 in the 1940s, 66 in the 1950s, 25 in the 1960s, 24 in the 1970s, 14 in the 1980s, and 4 in the 1990s (Reference 2.6-13). The total range of sounding data is from 12.1 to -165.4 ft at MLW with depths below MLW represented as negative values. The DEM data use the Universal Transverse Mercator (UTM) Zone 18N projected coordinate system with the North American Datum of 1927 (NAD27) for the horizontal coordinate system. The vertical datum is relative to MLW, where MLW is the average of all low water tides at a location over a 1983-2001 19-year period (or tidal epoch). The NOAA bathymetric data for the Chesapeake Bay are shown in Figure 2.6-12.

Bathymetric data were converted from local MLW datums to a global datum applicable for the entire model domain. MSL at the CBBT, which corresponds to the 1983-2001 tidal epoch, was adopted as the reference datum for the model and also assumed to be MSL for the CCNPP Units 1 & 2 site. This assumption is conservative as the difference between the MSL and MLW at the CBBT station is the maximum inside the Chesapeake Bay. The MLW-MSL relationship at CBBT is given on the NOAA website (Reference 2.6-14). Note that model results were converted to NGVD 29 elevations for comparison with elevations of safety-related and important-to-safety SSCs. The datum conversion relationship at the NOAA Cove Point, Maryland tide station (station 8577188) was used for this purpose. At the Cove Point station, MSL is 0.64 ft higher than the NGVD 29 datum.

A square grid spacing of 360 m by 360 m (1181 ft by 1181 ft) was used to analyze tsunami wave propagation in the Chesapeake Bay. Typically, 10 to 20 grid points per wave length are recommended to accurately represent wave propagation in models based on the shallow water equations. Given a tsunami wave length of 33,260 m (109,093 ft), estimated based on the amplitude and period of the tsunami wave incident to the bay, the tsunami wave internal to the bay would be represented by about 92 grid points using the 360 m by 360 m (1181 ft by 1181 ft) grid and therefore adequately resolved. The effect of grid size on simulated tsunami water level at the CCNPP Units 1 & 2 site was evaluated by comparing simulated results for grid sizes of 240 m by 240 m (787 ft by 787 ft) and 300 m by 300 m (984 ft by 984 ft). Figure 2.6-13 shows the variation of simulated water level at the CCNPP Units 1 & 2 site for the three different grid sizes using the nonlinear model.

These results show that the maximum water level at the CCNPP Units 1 & 2 site are essentially the same for the 300 m by 300 m (984 ft by 984 ft) and 360 m by 360 m (1181 ft by 1181 ft) grids, while the maximum water level for the 240 m by 240 m (787 ft by 787 ft) was slightly lower. Based on this sensitivity analysis and a computational time requirement to satisfy dispersion criteria, a grid size of 360 m by 360 m (1181 ft by 1181 ft) was adopted for the computational domain. The numbers of grids in the two horizontal directions are 223 (east-west direction) and 790 (north-south direction). The bathymetric data used in the model, based on the 360 m by 360 m (1181 ft by 1181 ft) grid, are shown in Figure 2.6-5.

2.6.4.2.3 Numerical Simulation Cases

Numerical simulations were performed for three cases corresponding to the three tsunami generator sources identified in Section 2.6.1 and Table 2.4-2. For each case, simulations were performed with both linear (TSU) and nonlinear (NLSWE) models. The nonlinear NLSWE model includes the effects of wave dissipation due to bottom friction. To represent bottom friction, a constant Manning's roughness coefficient of 0.025 was used for the entire model domain for all three cases. The selected value represents natural channels in a good condition, as reported by Imamura (Reference 2.6-3). Table 2.6-3 summarizes the model simulation conditions.

Model simulations were performed for a period of about 10 hours, which was selected by considering the tsunami travel time from the entrance of the Chesapeake Bay to the CCNPP Units 1 & 2 site and the incoming tsunami period. A simulation time step of 5 seconds was selected based on a numerical stability criterion.

Wave characteristics generated along the internal boundary are shown in Figure 2.6-6 (Case 1 Nonlinear), Figure 2.6-14 (Case 1 Linear), Figure 2.6-7 (Case 2 Nonlinear), Figure 2.6-15 (Case 2 Linear), Figure 2.6-8 (Case 3 Nonlinear), and Figure 2.6-9 (Case 3 Linear). These figures show that the water levels at three locations along the boundary agree reasonably well with the assumed incoming sinusoidal tsunami waves from the three potential tsunami sources. The three locations on the boundary are shown in Figure 2.6-5.

2.6.5 Tsunami Water Levels

The numerical simulation results of tsunami propagation in the Chesapeake Bay for different cases are summarized in Table 2.6-4. Contour maps of maximum computed water levels in the Chesapeake Bay for Case 1 for the nonlinear and linear simulations are shown in Figure 2.6-16 and Figure 2.6-17, respectively. These results show that the incoming tsunami waves dissipate quickly as they propagate up the Chesapeake Bay. The amounts of dissipation are similar in both the non-linear and linear simulations. The effects of wave non-linearity and bottom friction, accounted for in the nonlinear (NLSWE) model, result in additional dissipation of wave heights within the Chesapeake Bay. Therefore, simulation results from the linear (TSU) model are more conservative, providing greater amplitude at the CCNPP Units 1 & 2 site.

Variations in simulated water levels with time at the CCNPP Units 1 & 2 site for the selected tsunami scenarios are shown in Figure 2.6-10 and Figure 2.6-11. These results show that the maximum tsunami amplitude at the CCNPP Units 1 & 2 site is associated with Case 1, the Norfolk Canyon submarine landslide scenario. The maximum tsunami amplitude was obtained with the linear (TSU) model. As shown in Table 2.6-4, the maximum tsunami amplitude at the CCNPP Units 1 & 2 site is 0.326 m (1.07 ft), as referenced to MSL. The results indicate that the linear solution of the shallow water equations provides a bounding estimate of tsunami amplitude at the CCNPP Units 1 & 2 site.

An assessment of the sensitivity of model results and the PMT estimate to model inputs and assumptions indicated that there is uncertainty associated with simulating wave propagation in very shallow water depths near land boundaries. In particular, Synolakis (Reference 2.6-17) indicates that runup estimates from linear models are accurate provided that the ratio of tsunami amplitude to water depth is small. Ward (Reference 2.6-21) recommends that tsunami computations using linear models be confined to grid points where the water depth is greater than the amplitude of incoming tsunamis. At shallower water depths, Ward argues that waves no longer amplify because of increasing bottom friction, and that the limiting amplitude approximates the tsunami runup height. Because the Chesapeake Bay model includes water depths less than the amplitude of incoming tsunamis, the simulated maximum amplitude, taken from the linear model, was increased to provide assurance that these values are not underestimated. The basis for the factor used to increase the maximum amplitude is described below.

To quantitatively assess the effects of confining the tsunami computations to grid points where the ratio of tsunami amplitude to water depth is relatively large, a series of model sensitivity simulations were performed wherein the minimum allowable water depth (cutoff depth) was varied in the model. Three simulations with cutoff depths of 0.5 m (1.64 ft), 1.0 m (3.28 ft), and 2.0 m (6.56 ft) for both linear and nonlinear models were conducted using the Case 1 tsunami generator. In each simulation, portions of the domain having water depths less than the cutoff depth were eliminated from the model domain. The shallow water areas of the Chesapeake Bay affected by imposing cutoff depths include the western shoreline near the Potomac River mouth and upstream of the CCNPP Units 1 & 2 site. The simulated water levels at the CCNPP Units 1 & 2 site are shown in Figure 2.6-18 and Figure 2.6-19. The relative increases in maximum amplitude at the CCNPP Units 1 & 2 site as a function of cutoff depth are summarized in Table 2.6-5.

The results from these simulations show that the amplitudes of the tsunami wave peaks and troughs at the CCNPP Units 1 & 2 site generally increase with increasing cutoff depth for both linear and nonlinear models. Note that the maximum relative increase in amplitude (77 percent) occurred in the linear simulation with a 1.0 m cutoff depth; however, the maximum water level for this simulation appeared during the third wave peak, unlike the other cases wherein the relative increase was greatest for the first peak. Excluding this apparently anomalous case, the scenarios using a cutoff depth of 2.0 m resulted in the greatest amplitude change relative to the base case, on the order of 60 percent. Consequently, the selected maximum water level from the Case 1 linear simulation was increased by 60 percent (0.64 ft) to obtain the PMT water level at the CCNPP Units 1 & 2 site. Therefore, the maximum tsunami amplitude at the CCNPP Units 1 & 2 site was determined to be (1.07 ft + 0.64 ft =) 1.71 ft.

The PMT water level at the CCNPP Units 1 & 2 site was determined by adding an appropriate antecedent water level and a tsunami runup height to the computed tsunami amplitude. The antecedent water level was established as 4.34 ft NGVD 29, which accounts for the 10 percent exceedance high spring tide (2.17 ft NGVD 29), a sea level anomaly (1.1 ft NGVD 29), and long-term sea level rise (1.07 ft NGVD 29). The PMT water level is therefore (1.71 ft + 4.34 ft =) 6.05 ft NGVD 29.

Mader (Reference 2.6-7) indicates that tsunami runup is about 2 to 3 times the deep-water tsunami amplitude. Madsen and Fuhrman (Reference 2.6-8) describe a methodology to estimate tsunami runup on plane beaches employing the surf similarity parameter. Because the Chesapeake Bay bathymetry varies considerably from natural beaches, Madsen and Fuhrman's method may underestimate tsunami runup at the CCNPP Units 1 & 2 site. Therefore, a tsunami runup of 3 times the maximum tsunami amplitude in the Chesapeake Bay near the site, as recommended by Mader (Reference 2.6-7), was used to provide a conservative estimate of runup. The runup height therefore was estimated as (3 x 1.71 ft =) 5.13 ft.

The PMT high-water level, considering the maximum tsunami amplitude, antecedent conditions and runup, was therefore estimated as (6.05 ft + 5.13 ft =) 11.18 ft NGVD 29, rounded up to 11.5 ft NGVD 29.

The numerical simulation indicates that the tsunami waves would experience significant dissipation when propagating up the Chesapeake Bay. Incoming tsunami waves with an amplitude of 13 ft at the internal boundary dissipated over a distance of about 90 mi to an adjusted wave amplitude of 1.71 ft at the CCNPP Units 1 & 2 site when propagating over relatively shallow water depths.

The simulated travel time for the tsunami to arrive at the CCNPP Units 1 & 2 site from the model boundary near the Chesapeake Bay entrance was found to be about 3.5 hours. Note that the periods of the incoming tsunami wave were selected to be 1 hour (3,600 seconds) for Cases 1 and 2, and 1.44 hours (5,200 seconds) for Case 3.

Because the PMT maximum water level of 11.5 ft NGVD 29 is much less than the PMSS still-water level at the CCNPP Units 1 & 2 site, as established in Section 2.4, the PMT maximum water level would not constitute the controlling flood elevation at the CCNPP Units 1 & 2 site.

2.6.6 Hydrography and Harbor or Breakwater Influences on Tsunami

The Dominion Cove Point Liquefied Natural Gas (LNG) facility near Cove Point has a platform that is approximately 1,500 ft long. The platform is located approximately 4 mi southeast of the CCNPP Units 1 & 2 site. The platform is aligned with the main flow direction in the Chesapeake Bay and, therefore, will not cause any obstruction to tsunami propagation. The effect of the platform was not considered in tsunami model simulations. The bathymetric influence on tsunami propagation was included in the model simulation by the water depth.

2.6.7 Effects on Safety-Related Facilities

The safety-related and important-to-safety facilities for CCNPP Units 1 & 2, with the exception of the saltwater cooling pumps, are located in the main plant area where the grade elevation is about 45 ft NGVD 29 (Reference 2.6-26), substantially higher than the postulated PMT runup level of 11.5 ft NGVD 29. Tsunami flooding and associated hazards such as debris, waterborne missiles, sedimentation and erosion will not have any adverse impact in this main plant area.

The PMT runup level of 11.5 ft NGVD 29 will also not challenge the safe functioning of the saltwater cooling pumps housed in the intake structure on the shore because the configuration of the intake eliminates any tsunami-induced flooding concern. As described in CCNPP Units 1 & 2 UFSAR (Reference 2.6-26, Section 2.8), the intake structure has a 50-ft wide open deck at elevation 10 ft NGVD 29 on the bay side, with openings for the trash rakes and racks, stop logs and traveling screens. Behind the open deck is an enclosure housing the non-safety related circulating water pumps and the safety-related saltwater cooling pumps. The roof of the pump room is at elevation 28.5 ft NGVD 29 and has water tight hatches to provide access to the pumps for maintenance. An intake structure air supply unit is mounted on each saltwater pump hatch, and an air exhaust vent is mounted on each circulating water pump hatch. To minimize entry of moisture into the pump room, each air supply unit and air exhaust vent housing is provided with louvers designed for efficient moisture separation. The personnel door located at the north end of the intake structure is of a watertight design.

Impacts from debris and waterborne projectiles that potentially associate with tsunami events will not affect the operation of the saltwater cooling pumps as the intake is protected by the forebay baffle wall, trash racks and rakes, and traveling water screens. In addition, the PMT amplitude and runup level, estimated to be 1.71 ft and 11.5 ft NGVD 29, are much lower than the PMSS evaluated in Section 2.4. Considerations such as the hydrostatic and hydrodynamic forces, sedimentation and erosion on the operation of the saltwater pumps and the intake structure will be bounded by the storm surge induced flooding described in Section 2.4.

The maximum tsunami drawdown, as can be seen in Figure 2.6-19, is less than 0.5 m (1.64 ft) from the linear model simulation. This tsunami trough magnitude is in the range of tidal and wind waves typically experienced in the bay. Therefore, tsunami drawdown is not expected to have any adverse impact on CCNPP Units 1 & 2. Furthermore, the postulated drawdown trough is much smaller than the wind-wave amplitudes coincidental with the PMSS, as described in Section 2.4, and any attendant impact to the site would be bounded by the impacts from the storm surge events.

2.6.8 Conclusions

The PMT runup elevation, based on numerical model simulations, is predicted to be 11.5 ft NGVD 29 at the plant, which is below the maximum storm surge elevation with coincidental wave runup, as documented in Section 2.4. Therefore, the PMT would not be the controlling flood mechanism for the CCNPP Units 1 & 2 plant. Also, any attendant impact of the PMT would be bounded by the impacts from the storm surge event. This reevaluation result is consistent with the CCNPP Units 1 & 2 UFSAR (Reference 2.6-26), which states that tsunamis are not expected to affect the safe functioning of the plant.

2.6.9 References

2.6-1 Carrier, G., T. Wu and H. Yeh, *Tsunami Runup and Drawdown on a Plane Beach*, Journal of Fluid Mechanics, pages 79 through 99, 2003.

2.6-2 Driscoll, N. et al, *Potential for Large-scale Submarine Slope Failure and Tsunami Generation along the U.S. mid-Atlantic Coast*, Geology, Volume 28, Number 5, May 2000.

2.6-3 Imamura, F., Yalciner, A. C. and G. Ozyurt, *Tsunami Modeling Manual*, 2006.

2.6-4 Intergovernmental Oceanographic Commission, *Numerical method of tsunami simulation with the leap-frog scheme*, IUGG/IOC Time Project, Manuals and Guides, UNESCO, 1997.

2.6-5 Larsen, J. and H. Dancy, *Open Boundaries in Short Wave Simulations - A New Approach*, Coastal Engineering, Volume 7, pages 285 through 297, 1983.

2.6-6 Mader, C., *Modeling the La Palma Landslide Tsunami, Science of Tsunami Hazards*, Volume 19, pages 150 through 170, 2001.

2.6-7 Mader, C., *Modeling the 1755 Lisbon Tsunami, Science of Tsunami Hazards*, Volume 19, Number 2, 2001.

2.6-8 Madsen, P.A. and D.R. Fuhrman, *Analytical and Numerical Models for Tsunami Run-up*, In: *Tsunami and Nonlinear Waves*, A. Kundu (Ed.), Springer.

2.6-9 Description of Historical Tsunami, Website: http://www.state.me.us/doc/nrimc/mgs/explore/hazards/tsunami/jan05.htm, Date accessed: February 22, 2007.

2.6-10 National Academy of Sciences, *The Great Alaska Earthquake of 1964*, Committee on the Alaska Earthquake, Washington D.C.

2.6-11 UniStar Nuclear Services, LLC. *Calvert Cliffs Nuclear Power Plant Unit 3 Combined License Application*, Final Safety Analysis Report, Rev 8. March 2012.

2.6-12 National Geophysical Data Center / World Data System (NGDC/WDS) Global Historical Tsunami Database, Boulder, CO, USA. Available at http://www.ngdc.noaa.gov/hazard/tsu_db.shtml. Accessed: November 13 and 14, 2012.

2.6-13 Bathymetry Data of the Chesapeake Bay, NOAA, Website: http://egisws01.nos.noaa.gov/servlet/BuildPage?template=bathy.txt&parm1=M130&B1=Submit, Date accessed: September 6, 2006.

2.6-14 Water levels of Chesapeake Bay Bridge Tunnel, NOAA, Website: http://tidesandcurrents.noaa.gov/data_menu.shtml?stn=8638863%20Chesapeake%20Bay %20Bridge%20Tunnel,%20VA&type=Datums, Date accessed: February 24, 2007.

2.6-15 U.S. Nuclear Regulatory Commission, *Tsunami Atlas for the Coasts of the United States*, NUREG/CR-1106, 1979.

2.6-16 Pararas-Carayannis, G., *Evaluation of the Threat of Mega Tsunami Generation from Postulated Massive Slope Failures of Island Stratovolcanoes La Palma,* *Canary Islands, and on the Island of Hawaii*, Science of Tsunami Hazards, Volume 20, pages 251 through 277, 2002.

2.6-17 Synolakis, C., *The Run-up of Solitary Waves*, Journal of Fluid Mechanics, Volume 185, pages 523-545.

2.6-18 U.S. Geological Survey, *Taylors Island Quadrangle, Maryland, 7.5 Minute Series Orthophotomap (Topographic)*, Taylors Island, MD, N3822.5–W7615/7.5, 1982.

2.6-19 U.S. Geological Survey, *Hudson Quadrangle, Maryland, 7.5 Minute Series Orthophotomap (Topographic)*, Hudson, MD, N3830– W7615/7.5, 1982.

2.6-20 U.S. Geological Survey, *The Chesapeake Bay Bolide Impact: A New View of Coastal Plain Evolution*, United States Geological Survey Fact Sheet 49-98, USGS, 1998, Website: http:// marine.usgs.gov/fact-sheets/fs49-98/, Date accessed: May 7, 2009.

2.6-21 Ward, S., *Landslide Tsunami*, Journal of Geophysical Research, Volume 106, Number 6, 2001.

2.6-22 Ward, S., and S. Day, *Cumbre Vieja Volcano – Potential Collapse and Tsunami at La Palma, Canary Island*, American Geophysical Union, 2001.

2.6-23 Yoon, S., *Propagation of Distant Tsunamis over Slowly Varying Topography*, Journal of Geophysical Research, Volume 107, Number C10, 2002.

2.6-24 Atlantic and Gulf of Mexico Tsunami Hazard Assessment Group, 2008, *Evaluation of Tsunami Sources with the Potential to Impact the U.S. Atlantic and Gulf Coasts - A Report to the Nuclear Regulatory Commission*: U.S. Geological Survey Administrative Report.

2.6-25 Deep-ocean Assessment and Reporting of Tsunamis (DART®) Description, NOAA National Data Buoy Center. Website: http://www.ndbc.noaa.gov/dart/dart.shtml. Date accessed: November 5, 2012.

2.6-26 Calvert Cliffs Nuclear Power Plant Units 1 & 2, *Updated Final Safety Analysis Report*, Section 2.8, Rev. 36.

2.6-27 U.S. Nuclear Regulatory Commission, *Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America*, NUREG/CR-6966, March 2009.

Table 2.6-1 Selected Historical Tsunamis Arriving at the East Coast of US and Canada (Reference 2.6-12)

Date	Country	City	Latitude	Longitude	Earthquake Magnitude [(4)]	Tsunami Cause	Maximum Tsunami Water Height
11/1/1755	Portugal	Lisbon	36.000	-11.000	MMI XI	Earthquake	98.4 ft (30.00 m) (Lagos)[(1)] 9.8 ft (3 m) (East Coast)[(3)]
9/24/1848	Canada	Fishing Springs Harbor, Newfoundland	52.616	-55.766	-	Landslide	-
6/27/1864	Canada	SW Avalon Peninsula, Newfoundland	46.500	-53.700	–	Earthquake	–
9/1/1886	USA	Charleston, SC	32.900	-80.000	Mw 7.7	Earthquake	–
9/1/1895	USA	High Bridge, NJ	40.667	-74.883	Mfa 4.3	Earthquake	–
12/6/1917	Canada	Halifax, Nova Scotia	44.663	-63.583	-	Explosion	23.0ft (7 m) (Halifax) [(1)]
10/11/1918	USA Territory	Mona Passage, Puerto Rico	18.500	-67.500	Mw 7.3	Earthquake	20 ft (6.10 m) (Punta Agujereada)[(1)] 0.2 ft (0.06 m (Atlantic City)[(2)]
11/18/1929	Canada	Grand Banks, Newfoundland	44.690	-56.000	Mw 7.2	Earthquake and Landslide	23 ft (7.00 m (Taylor's Bay)[(1)] 2.2 ft (0.68 m) (Atlantic City)[(2)]
8/4/1946	Dominican Republic	Northeastern Coast	19.300	-68.900	Mw 7.8	Earthquake	16.4 ft (5.00 m) (Rio Boba)[(2)] –
8/8/1946	Dominican Republic	Northeastern Coast	19.710	-69.510	Mw 7.4	Earthquake	2 ft (0.60 m) (San Juan)[(2)] –
5/19/1964	USA	Long Island, NY	40.800	-73.100	–	Landslide	0.92 ft (0.28 m) (Plum Island)[(2)] –
12/26/2004	Indonesia	Off W. Coast Of Sumatra	3.295	95.982	Mw 9.1	Earthquake	167 ft (50.90 m) (Labuhan, Indonesia)[(1)] 0.36 ft (0.11 m) (Atlantic City)[(2)]
10/28/2008	USA	ME	43.800	-69.700	-	Unknown	12 ft (3.6 m) (Booth Bay Harbor) [(1)]

[(1)]Eyewitness measurement
[(2)]Tide gauge measurement
[(3)] This value is an estimate based on modeling results from Reference 2.6-7, and does not come from Reference 2.6-12.
[(4)] Mw = moment magnitude scale; Mfa = felt area magnitude scale, MMI = Modified Mercalli Intensity Scale of 1931.

Table 2.6-2 Tsunami Wave Characteristics at the Entrance of the Chesapeake Bay

Case	Amplitude	Period (seconds)	Source Location
1	13 ft (4 m)	3,600	Norfolk Canyon submarine landslide
2	10 ft (3 m)	3,600	Canary Island submarine landslide
3	3.1 ft (0.9 m)	5,200	Haiti earthquake

Note: model setup uses SI units

Table 2.6-3 Summary of Numerical Analysis for the Tsunami Propagation

Parameter	Value
Governing equation	Nonlinear shallow water equation and linear shallow water equation
Computational domain	223 (east-west) by 790 (north-south)
Grid space	1,181 ft by 1,181 ft (360 m by 360 m) square
Time step	5 seconds
Bathymetry data	NOAA Chesapeake Bay Digital Elevation Model (resolution: 98.4 ft by 98.4 ft (30 m by 30 m))
Reference water level	Local mean sea level of Chesapeake Bay Bridge Tunnel
Manning's roughness coefficient	0.025

Note: model setup uses SI units

Table 2.6-4 Simulated Maximum Tsunami Magnitude

Case No.	Maximum Tsunami Amplitude	Remarks
1	0.51 ft (0.155 m) 1.07 ft (0.326 m)	Nonlinear and bottom friction Linear without bottom friction
2	0.43 ft (0.131 m) 0.80 ft (0.245 m)	Nonlinear and bottom friction Linear without bottom friction
3	0.42 ft (0.127 m) 0.86 ft (0.262 m)	Nonlinear and bottom friction Linear without bottom friction

Note: model simulation results are in SI units

Table 2.6-5 Simulated Maximum Tsunami Magnitude at Site for Various Cutoff Depths for Case 1

Simulation Condition		Maximum Amplitude	% Amplitude Change
Cutoff Depth	Model Option		
0.0 m	Nonlinear	0.51 ft (0.155 m)	-
	Linear	1.07 ft (0.326 m)	-
0.5 m	Nonlinear	0.57 ft (0.175 m)	12.9
	Linear	1.39 ft (0.423 m)	29.8
1.0 m	Nonlinear	0.65 ft (0.198 m)	27.7
	Linear	1.89 ft (0.577 m)	77.0
2.0 m	Nonlinear	0.80 ft (0.244 m)	57.4
	Linear	1.72 ft (0.524 m)	60.7

Note: model simulation results are in SI units

Figure 2.6-1 Map of Tsunami Source Generators

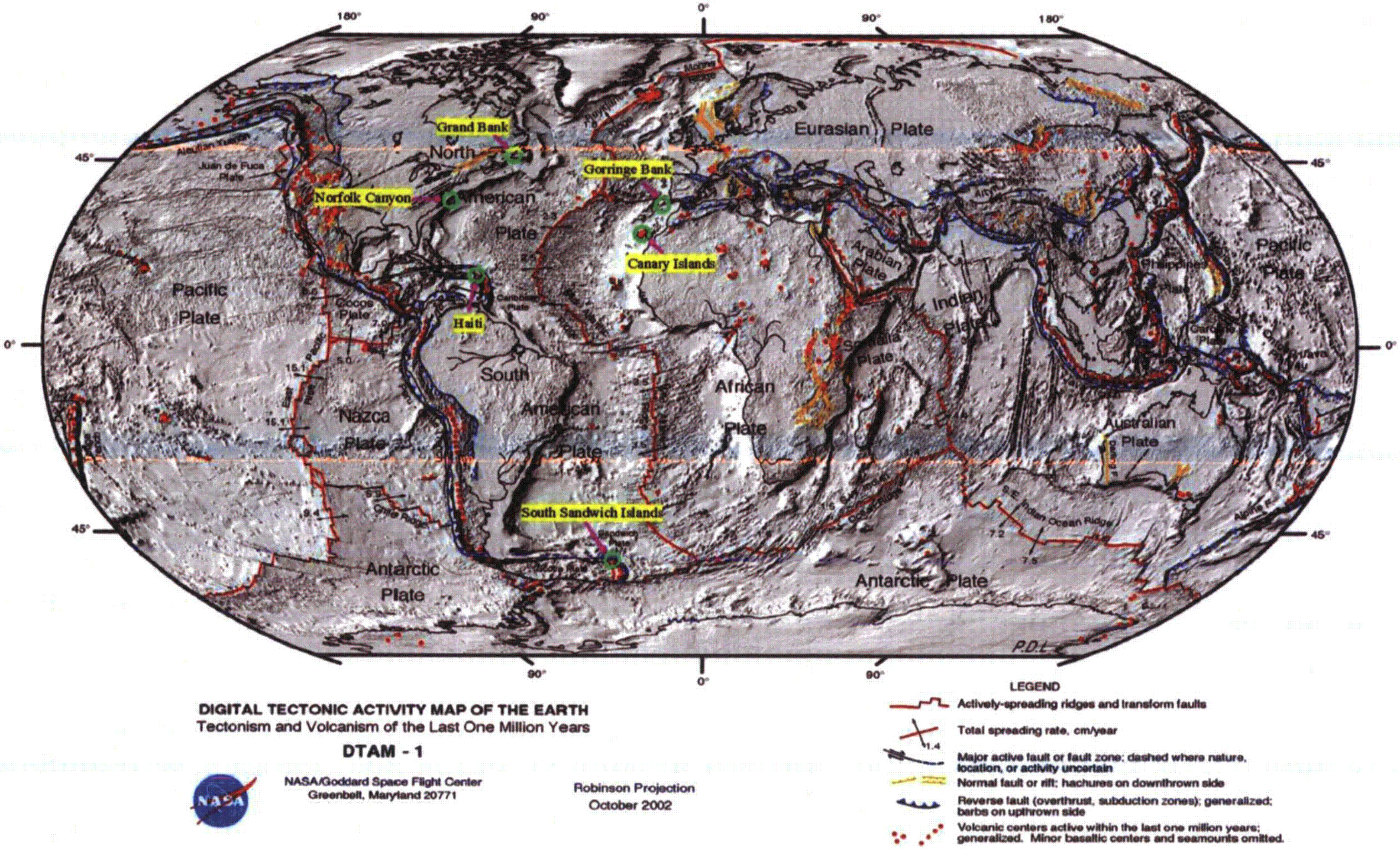


Figure 2.6-2 Staggered Grid for Leap-Frog Scheme

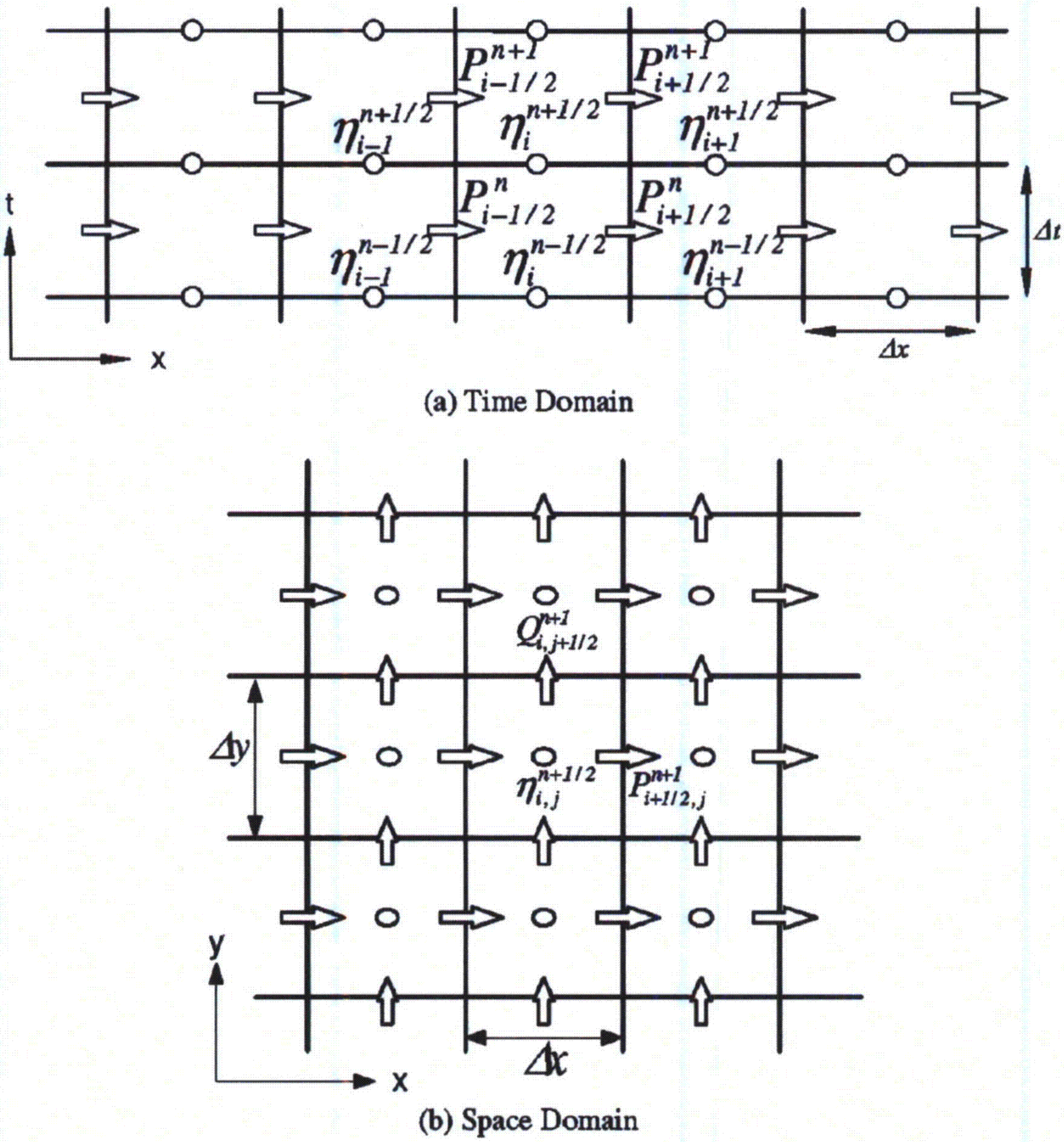
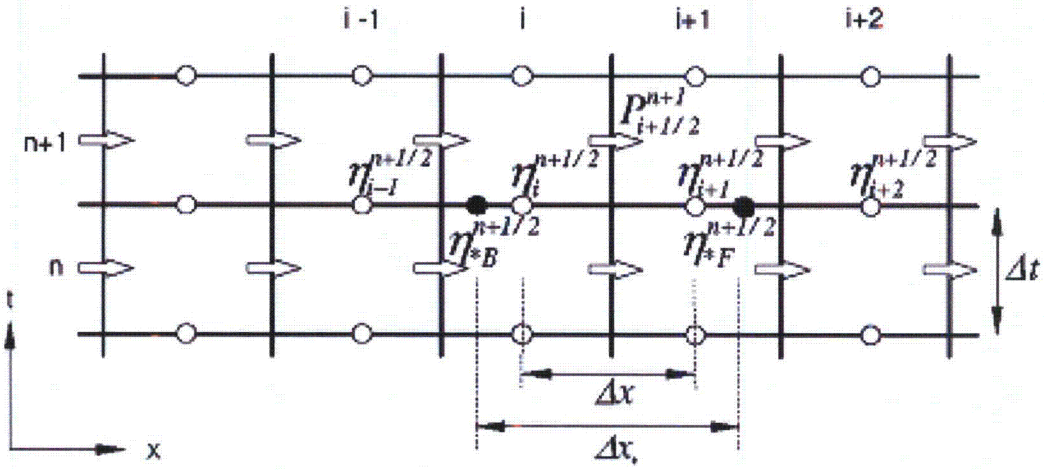
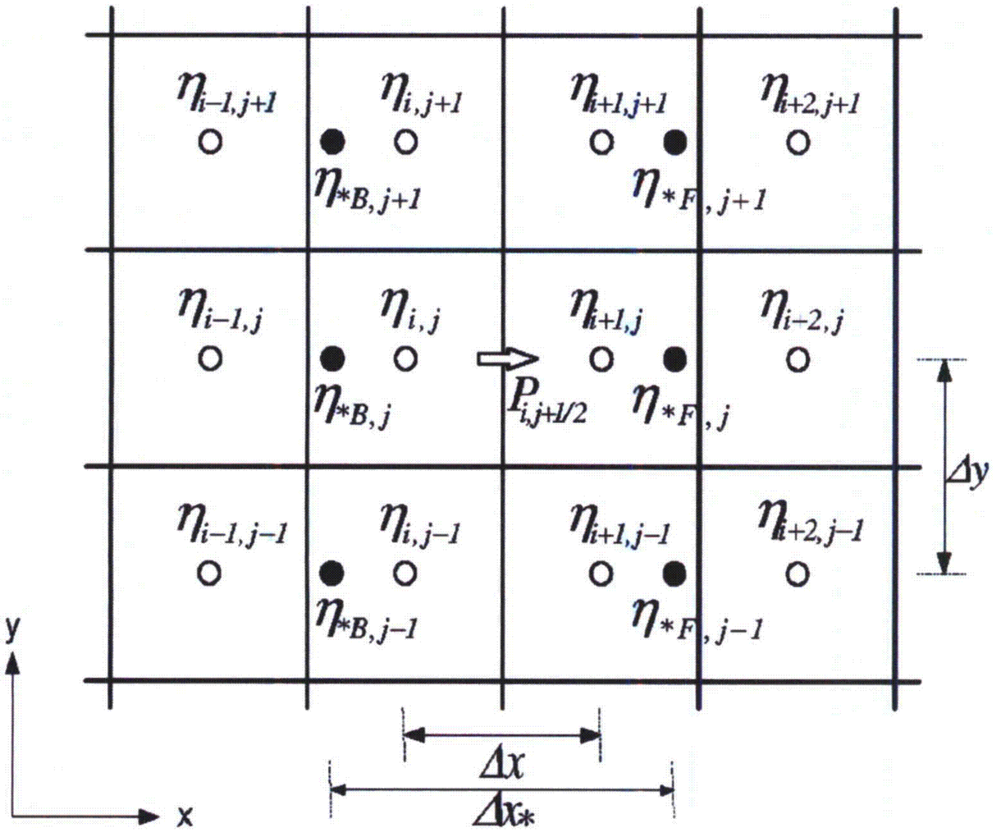


Figure 2.6-3 Time Grid Scheme for Assignment of Variables



○ Actual Grid Point, ● Hidden Grid Point

Figure 2.6-4 Spatial Grid Scheme for Assignment of Variables



○ Actual Grid Point, ● Hidden Grid Point

Figure 2.6-5 Computational Domain and Model Bathymetry for Tsunami Simulation in Chesapeake Bay

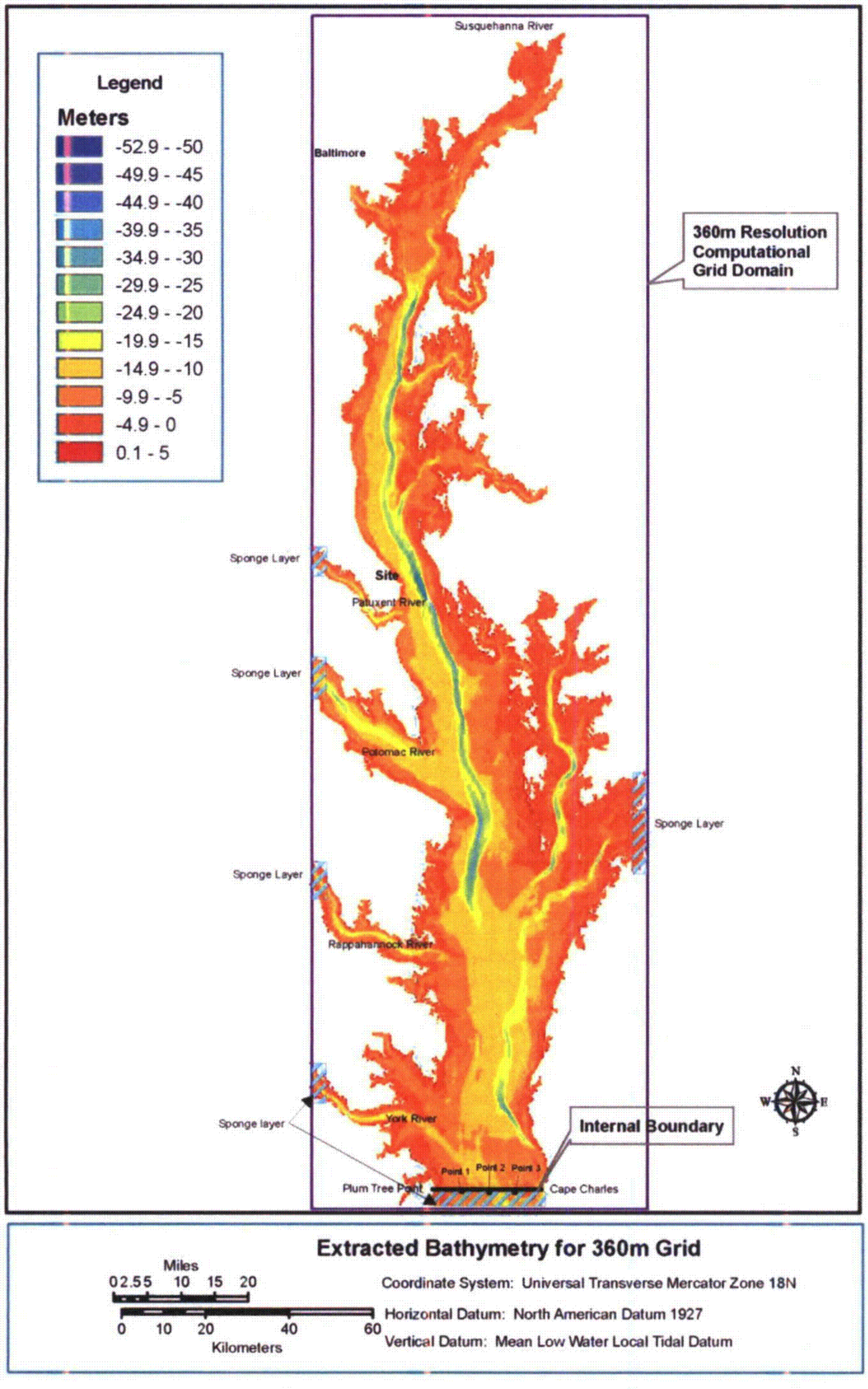


Figure 2.6-6 Water Levels (MSL) Along Internal Boundary for Case 1, Nonlinear Model

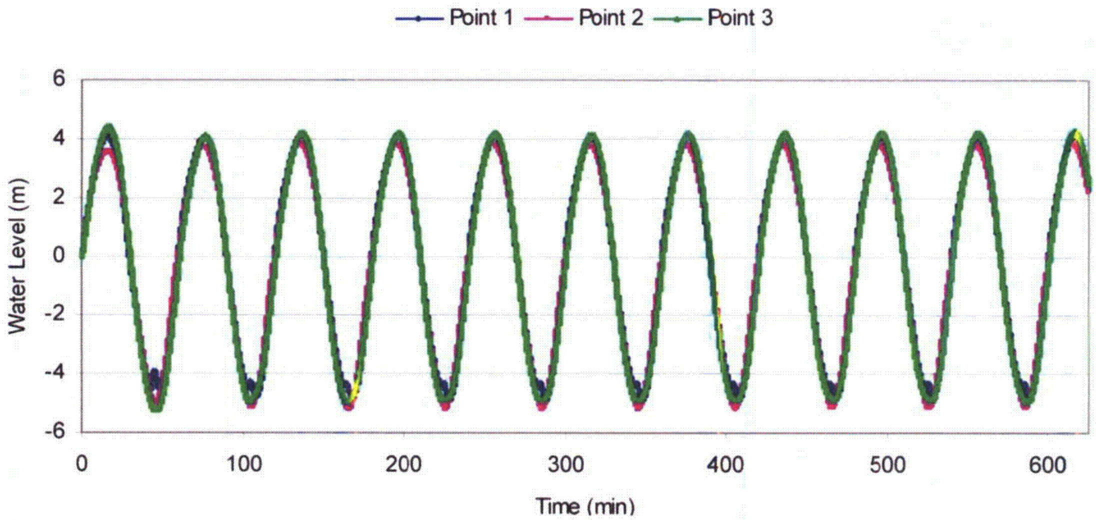


Figure 2.6-7 Water Levels (MSL) Along Internal Boundary Case 2, Nonlinear Model

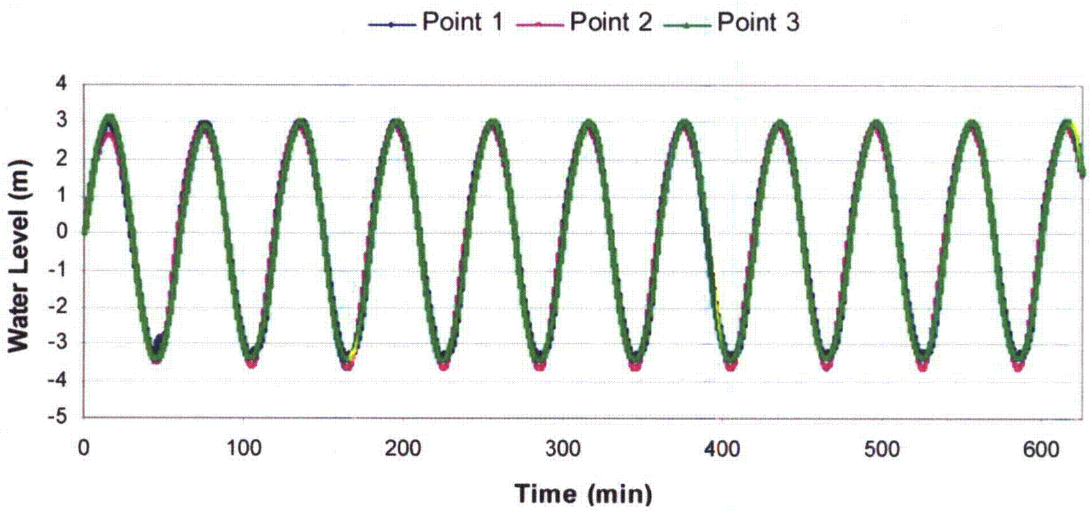


Figure 2.6-8 Water Levels (MSL) Along Internal Boundary for Case 3, Nonlinear Model

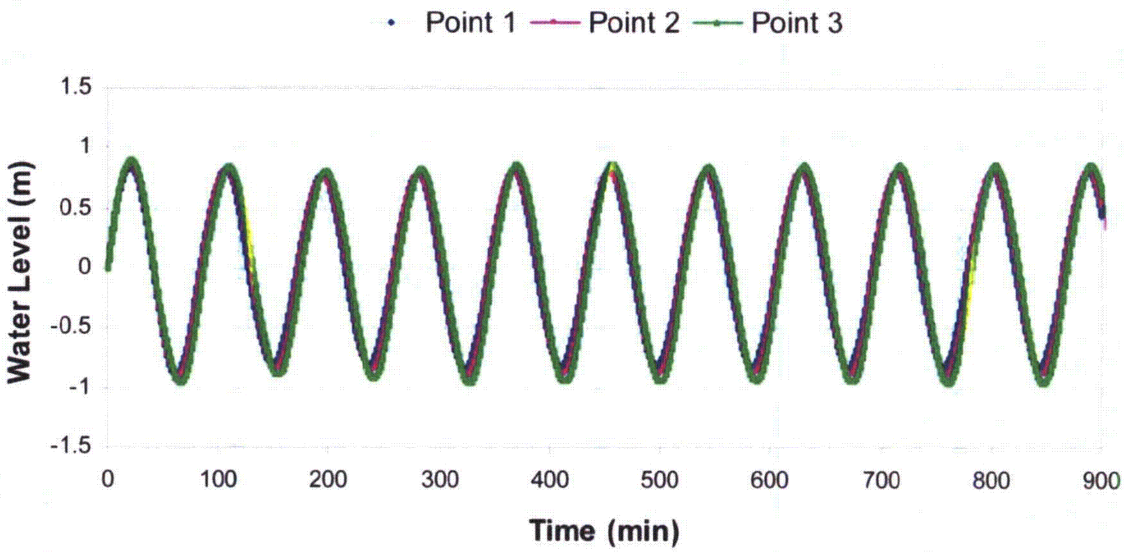


Figure 2.6-9 Water Levels (MSL) Along Internal Boundary for Case 3, Linear Model

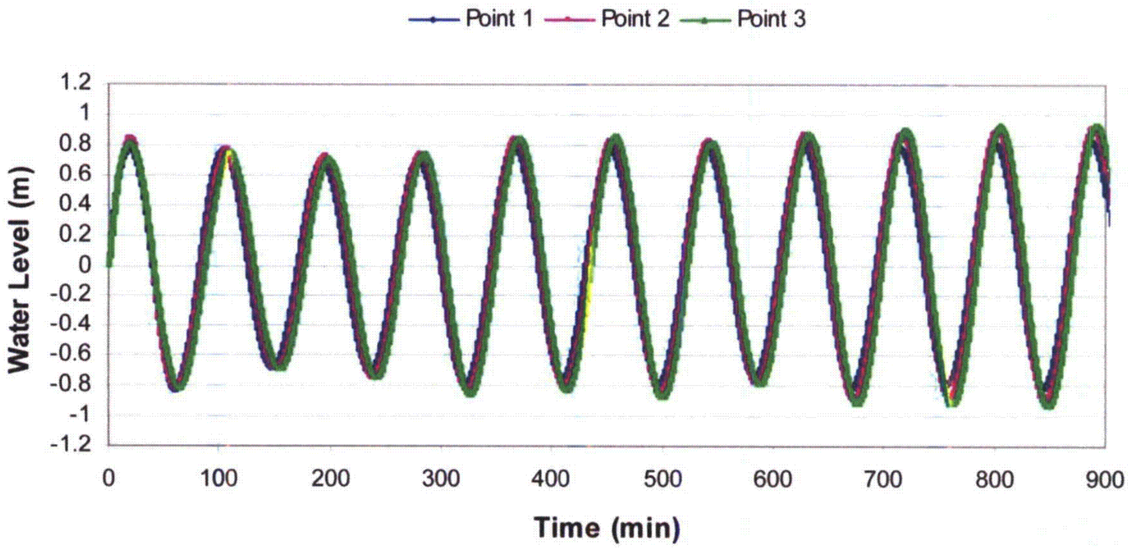


Figure 2.6-10 Time History of Tsunami Water Levels (MSL) at the CCNPP site, Case 1 through 3, Nonlinear Model

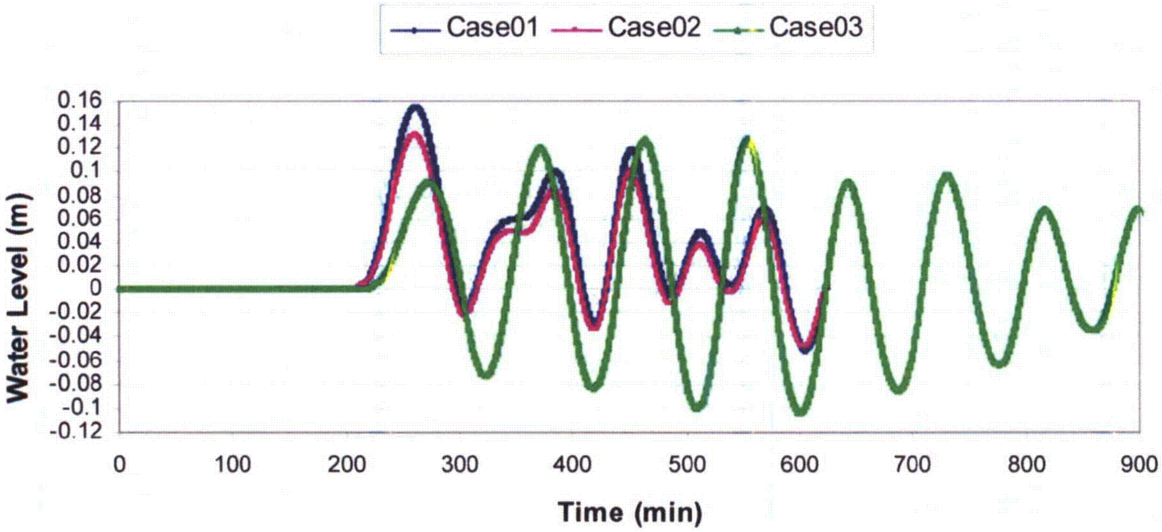


Figure 2.6-11 Time History of Tsunami Water Levels (MSL) at the CCNPP site, Case 1 through 3 Linear Model

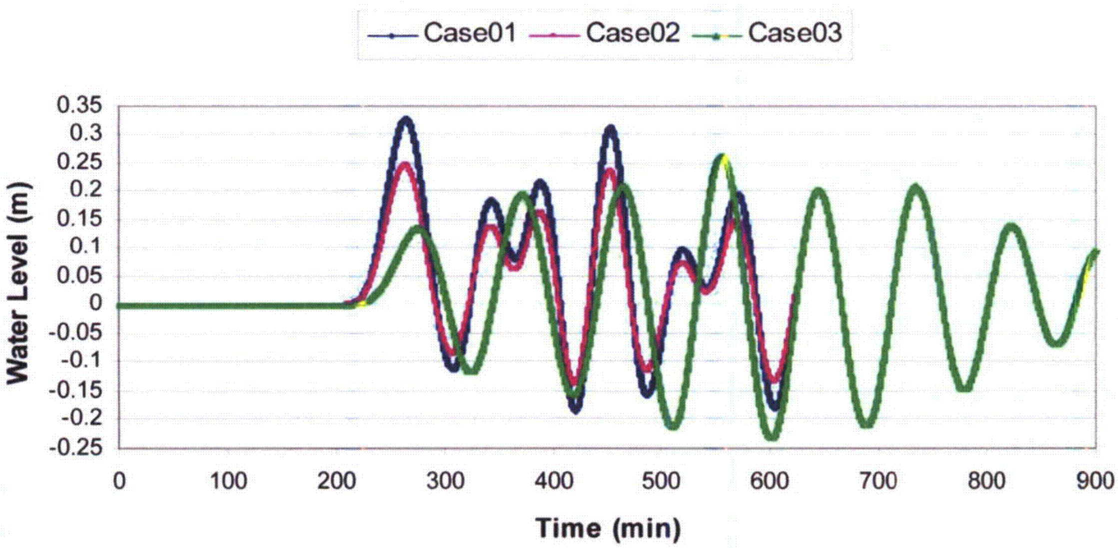


Figure 2.6-12 Chesapeake Bay Digital Elevation Model from NOAA

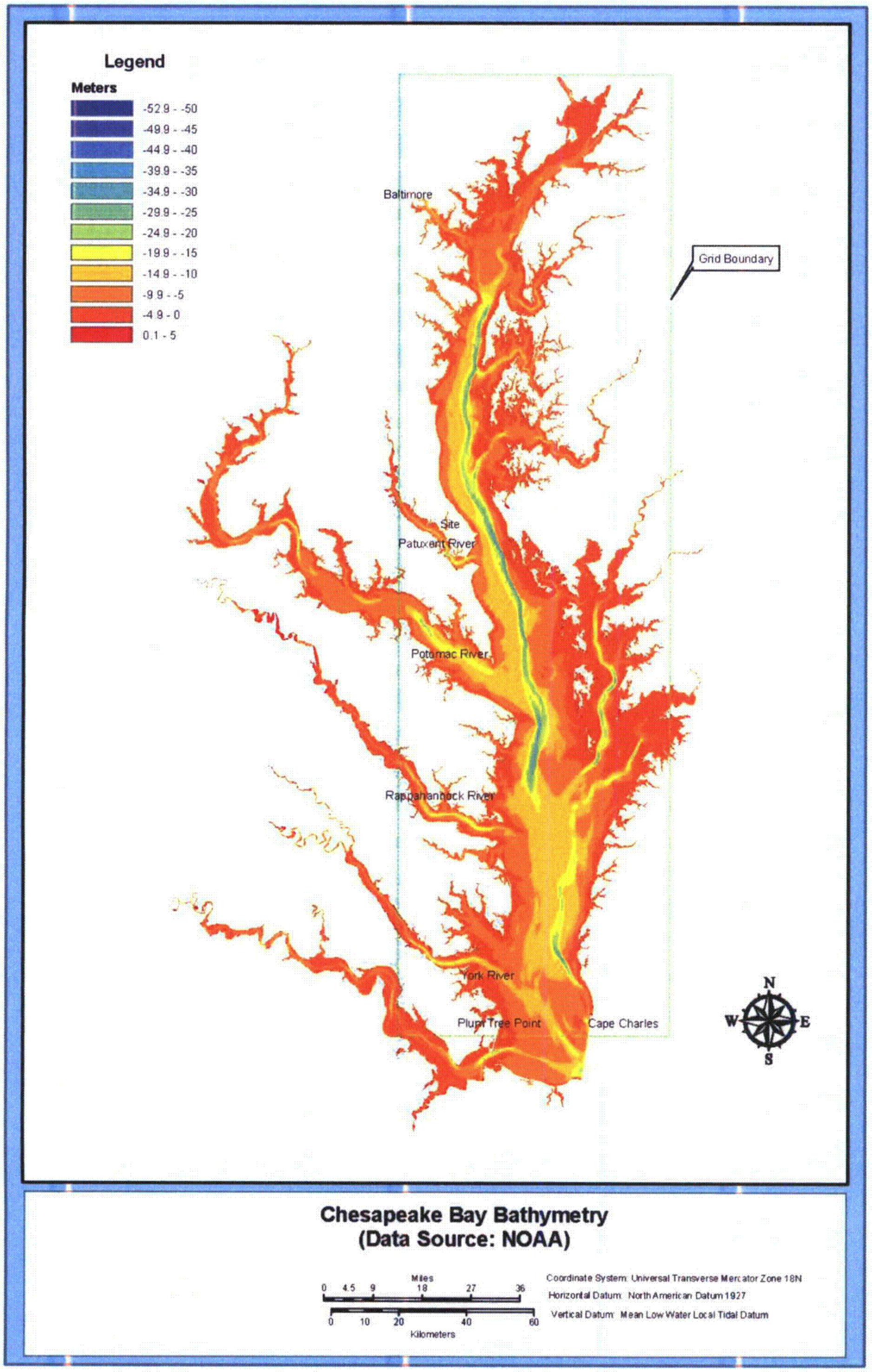


Figure 2.6-13 Comparison of Simulated Water Levels (MSL) at the CCNPP Site for Different Grid Sizes for Case 1, Nonlinear Model

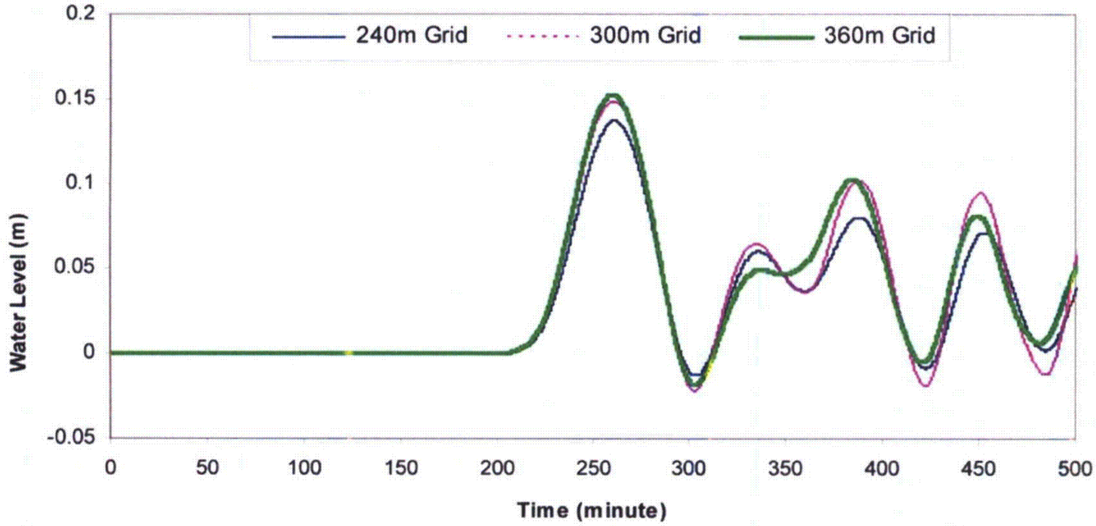


Figure 2.6-14 Water Levels (MSL) along Internal Boundary for Case 1, Linear Model

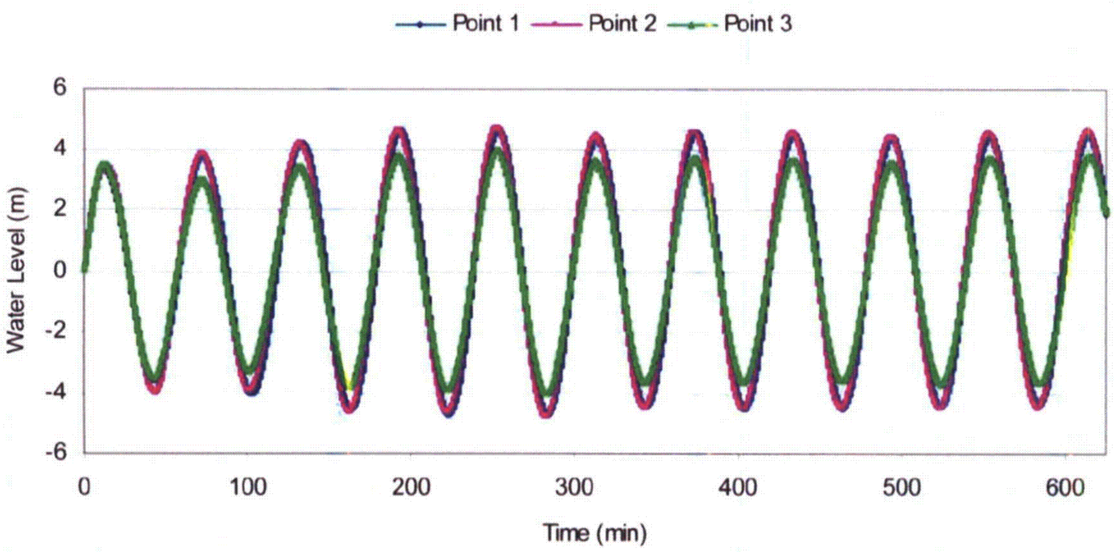


Figure 2.6-15 Water Levels (MSL) along Internal Boundary for Case 2, Linear Model

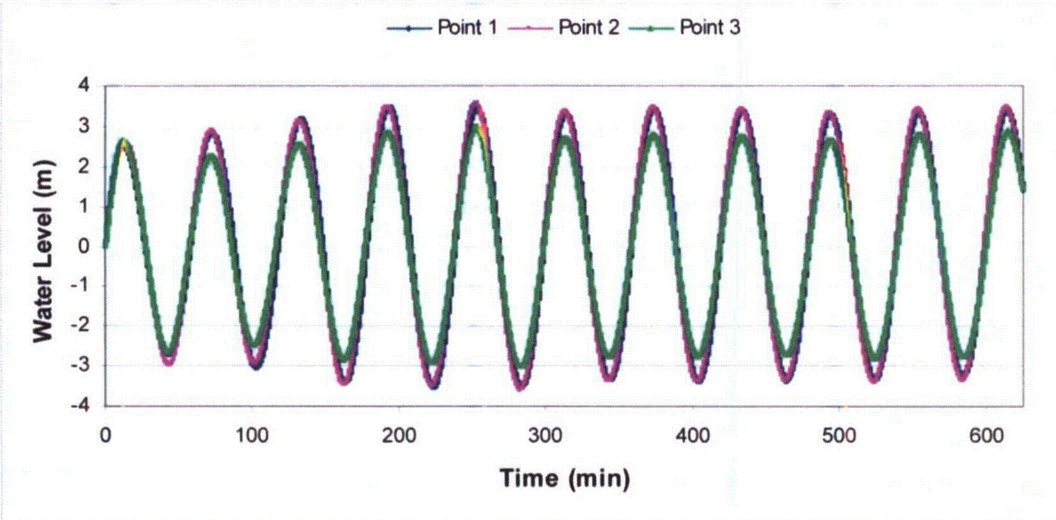


Figure 2.6-16 Contour of Maximum Water Levels (MSL) for Case 1, Nonlinear Model

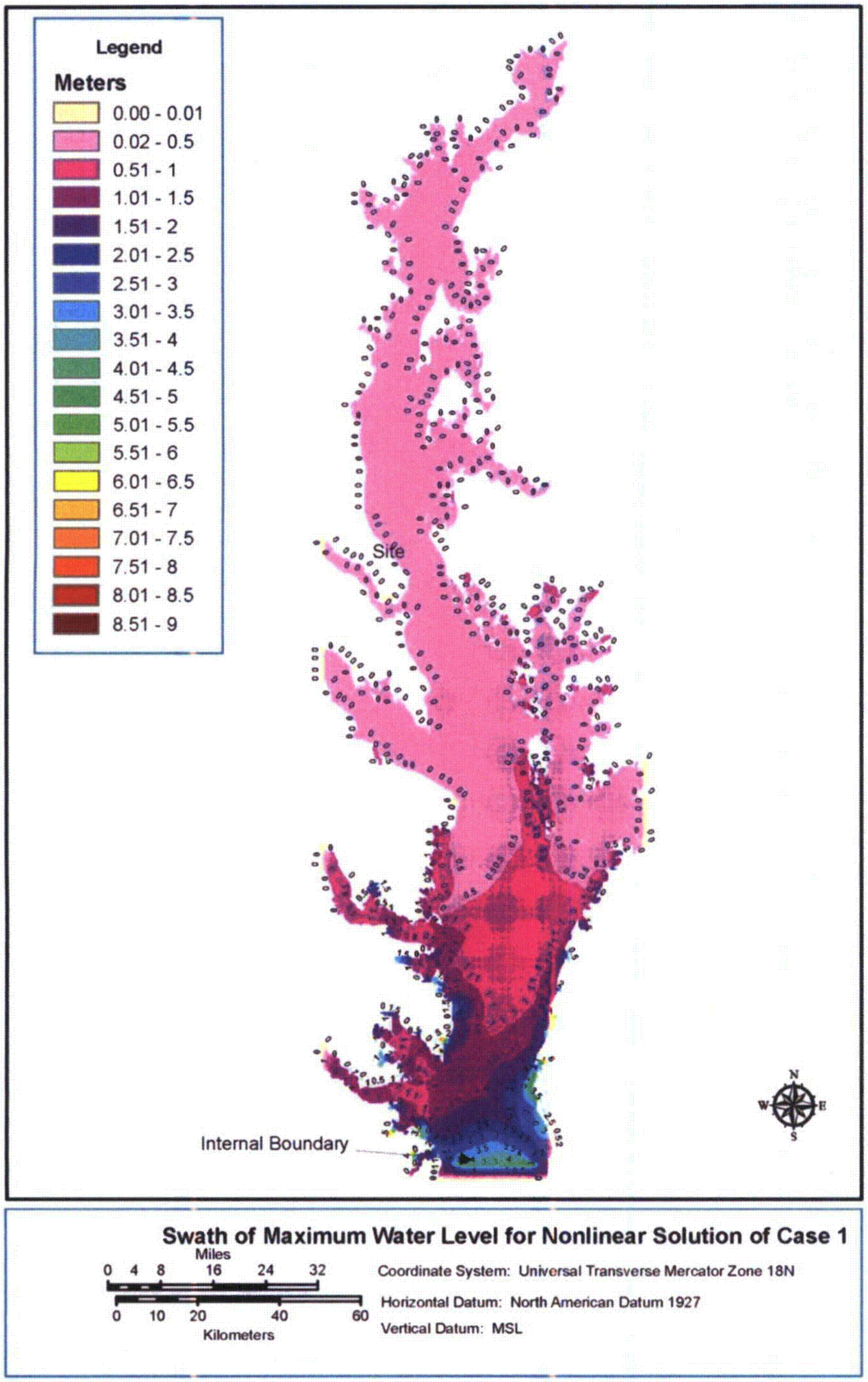


Figure 2.6-17 Contour of Maximum Water Levels (MSL) for Case 1, Linear Model

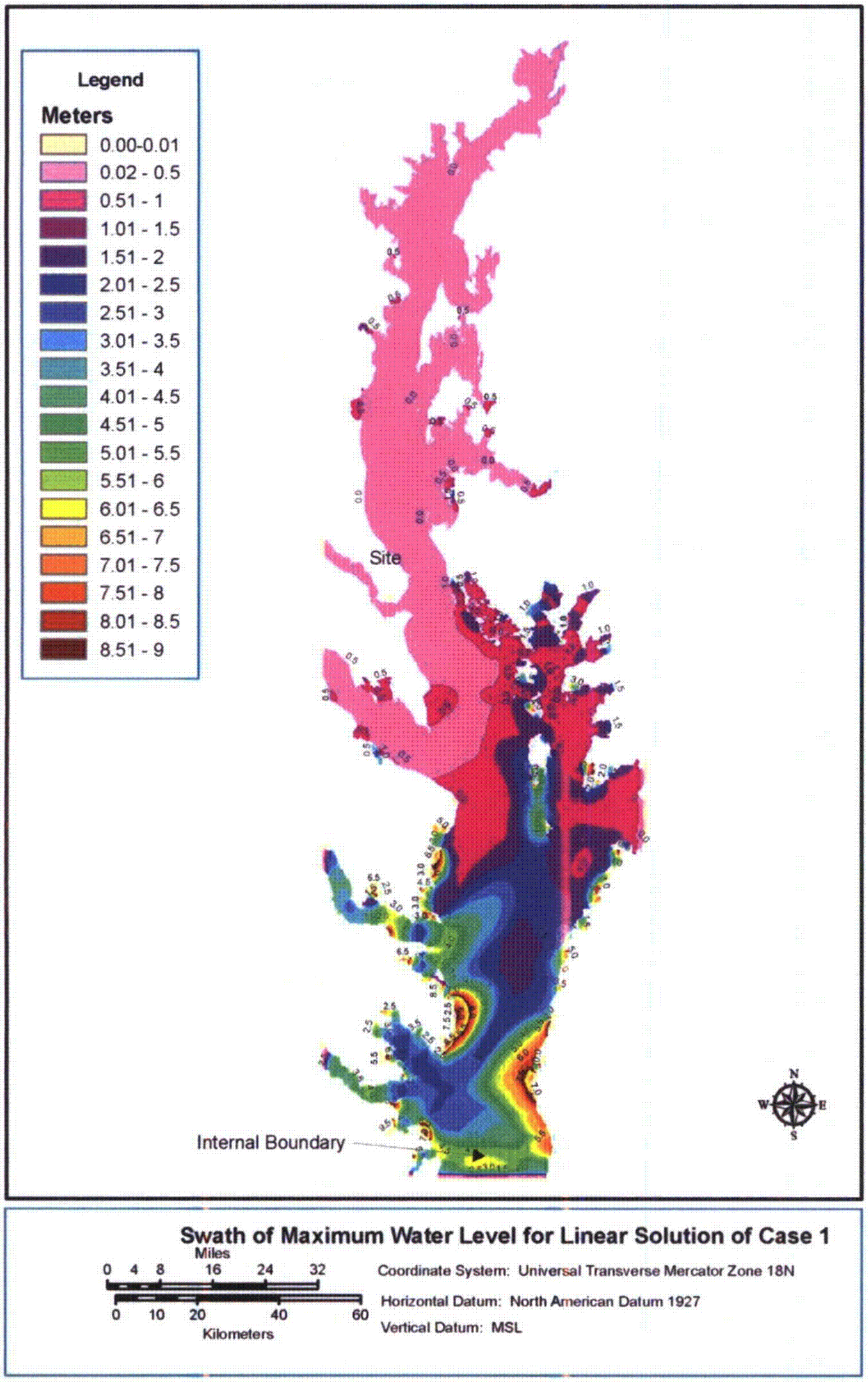


Figure 2.6-18 Time History of Tsunami Water Levels (MSL) at the CCNPP Site for Different Cutoff Depths, Case 1, Nonlinear Solution

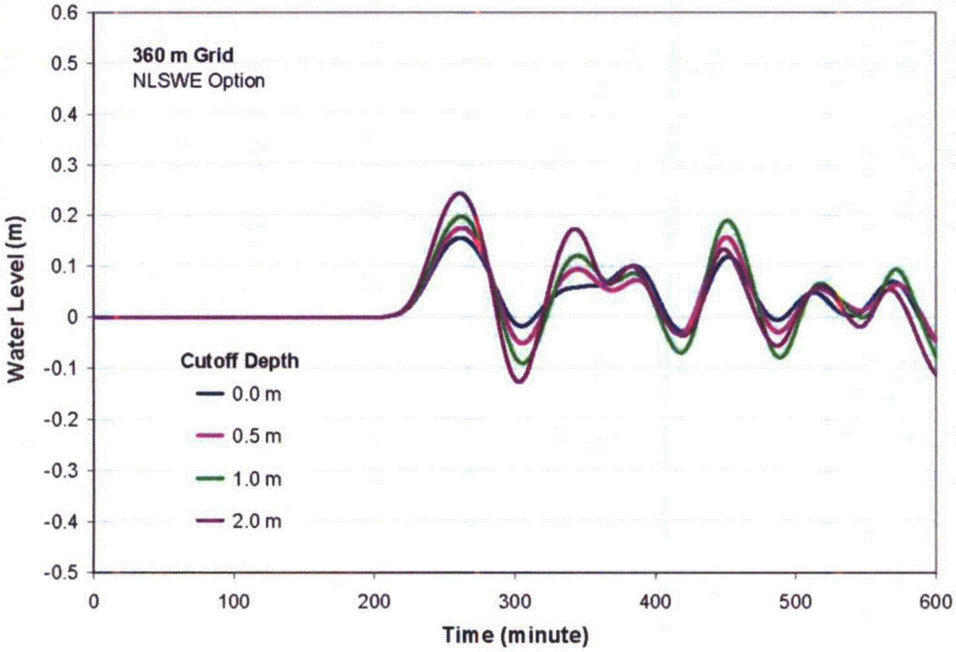
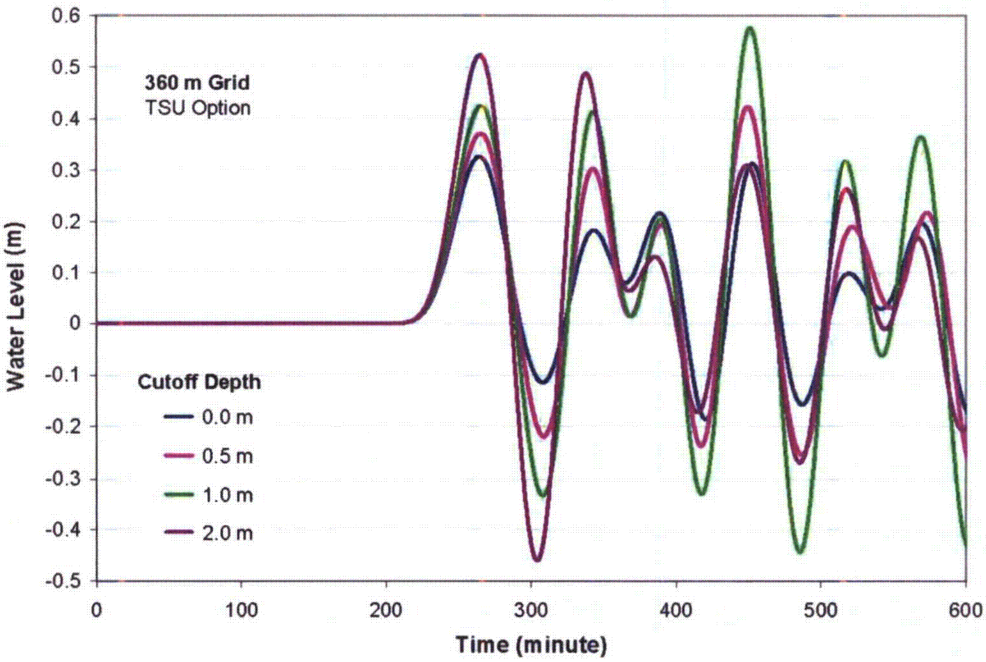


Figure 2.6-19 Time History of Tsunami Water Levels (MSL) at the CCNPP Site for Different Cutoff Depths, Case 1, Linear Solution



2.7 Ice Induced Flooding

2.7.1 Ice Conditions

Ice induced flooding at a nuclear power plant site could occur by the following mechanisms:

- Breach of ice jams causing flooding at site
- Ice blockage of the drainage system causing flooding

Historical data characterizing ice conditions at the CCNPP site have been collected and the effects evaluated. These data include ice cover and thickness observations in the Chesapeake Bay and its tributaries, ice jam records, and long term air temperature measurements from the nearby Patuxent River Naval Air Station meteorological tower (WBANID 13721). Patuxent River Naval Air Station is approximately 10 mi south of the CCNPP site on the same (western) shore of the Chesapeake Bay. It maintains a data record from 1945 to present.

2.7.2 Historical Ice Formation

The climate at the CCNPP site is part of the Chesapeake Bay climate system. Based on air temperature data summaries collected at Patuxent River Naval Air Station from 1971 through 2000, the monthly average air temperature normal in the region ranges from about 36.1°F in January to 78.1°F in July, while the monthly minimum air temperature normal for January is 28.3°F and for February is 29.9°F (Reference 2.7-1).

Daily air temperatures measured at the Patuxent River Naval Air Station meteorological station (Reference 2.7-5) indicate that below freezing temperatures occur typically between the months of November and March. However, maximum accumulated freezing degree-days, as defined in Section 2.7.3, occur mostly in January and February.

Observations of ice cover conditions in the Chesapeake Bay indicate that the winters of 1977 through 1981 were unusually cold and icing conditions were more severe than normal. The winter of 1977 was the coldest and iciest winter on record in the region. The ice and snow coverage of the Chesapeake Bay was about 85 percent, compared to normal conditions of about 10 percent (Reference 2.7-2).

2.7.3 Surface Ice Sheet

The maximum ice thickness that could form at the CCNPP site was estimated using historical air temperature data from the nearby Patuxent River Naval Air Station meteorological tower for the period of 1945 through 2006. Surface ice thickness can be estimated as a function of accumulated freezing degree-days (AFDD) using the modified Stefan equation (Reference 2.7-3). AFFD is obtained by summing the freezing degree-days for each day, which is the difference between the freezing point (32°F) and the average daily air temperature. For the water years 1946 through 2006, the maximum AFDD is 265.3°F days occurring on February 9, 1977 with the corresponding ice thickness estimated to be approximately 13 in. This estimate is conservative in regards to seawater ice thickness because it assumes a freshwater freezing point of 32°F (0°C).

Because the Chesapeake Bay is brackish, the freezing point will be depressed, which will mitigate the formation of surface ice. The conservatism is apparent when the 13 in estimate is compared to the 2 to 8 in ice thicknesses observed south of the Chesapeake Bay Bridge in early February of 1977, the iciest winter on record for the region (Reference 2.7-2). Subsequent analysis of air temperature data for water years 2007-2011 and partial water year 2012 (ending in August) showed that the maximum AFDD for this period is significantly smaller than the AFDD estimated for 1977, the iciest winter on record. Ice formation in the Chesapeake Bay, however, will not cause any adverse flood risk to the safety-related or important-to-safety facilities as they are located at a minimum elevations of 28.5 ft NGVD 29.

2.7.4 Potential for Ice Jam

Although the tributaries to the Chesapeake Bay are prone to ice formation, there has been no major ice jam formation or flooding recorded due to breaching of ice jams on the Patuxent River in recent history. Three ice jam incidents are recorded to have occurred on the river's tributaries: two on the Little Patuxent River, at Savage, Maryland, and one on the Western Branch near Largo, Maryland (Reference 2.7-4). The incidents on the Little Patuxent occurred in January of 1944 and February of 1948. However, because Savage, Maryland is about 62 river mi from the mouth of Patuxent River, the ice jam formation or breaching could not have had any effect on the CCNPP site. Similarly, the incident on the Western Branch in January 1961 could not have caused any impact at the site since the confluence of the Western Branch and the Patuxent is about 40 mi upriver. There were no recorded ice jams in the immediate vicinity of the site, and the streams close to the site have small drainage areas and therefore do not have the potential to cause ice jam induced flooding of Units 1 & 2.

2.7.5 Effect of Ice and Snow Accumulation on Site Drainage

Air temperature measurements at the Patuxent River Naval Air Station meteorological station indicate that mean daily temperatures at the site have periodically fallen below freezing for multiple consecutive days in winter (Reference 2.7-5). This introduces the possibility of ice blockage of small catch basins; storm drains; culverts and roof drains. As described in Section 2.1, the local drainage analysis of the CCNPP Units 1 & 2 site assumed that all catch basins, storm drains, and culverts would be blocked by ice, snow or other obstructions, rendering them inoperative during a local PMP event. Therefore, the flood hazard to the safety-related or important-to-safety facilities due to temporary blockage of site drainages as a result of any mechanism including an ice event is evaluated in the local PMP analysis documented in Section 2.1. According to the operating records of existing CCNPP Units 1 & 2, there have been no flooding incidents caused by ice blockage of storm drains on the site.

2.7.6 Conclusions

As established in the subsections above, the results of the evaluation of historical temperature data in the vicinity of CCNPP Units 1 & 2 and a search of the USACE Ice Jam Database indicate that ice induced flooding is not a credible hazard that will adversely impact the safety functions of the plant. The flood hazard to the safety-related and important-to-safety facilities due to temporary blockage of site drainages by any mechanism including an ice event is evaluated as part of the local PMP analysis documented in Section 2.1.

2.7.7 References

2.7-1 National Oceanic and Atmospheric Administration, *Climatography of the United States, No. 81-18, Monthly Station Normals of Temperature, Precipitation, and Heating and Cooling Degree Days 1971 to 2000*, Maryland (and Washington D.C.), February 2002.

2.7-2 Foster, J., *Mariners Weather Log*, Volume 26, Number 2, Ice Observations on the Chesapeake Bay 1977-1981, National Weather Service, 1982.

2.7-3 U.S. Army Corps of Engineers, *Method to Estimate River Ice Thickness Based on Meteorological Data,* ERDC/CRREL Technical Note 04-3, June 2004.

2.7-4 Ice Jam Database, U.S. Army Corps of Engineers. Available at https://rsgisias.crrel.usace.army.mil/icejam/, accessed September 19, 2012.

2.7-5 NCDC Climate Data Online, NOAA. Available at http://www7.ncdc.noaa.gov/CDO/cdoselect.cmd?datasetabbv=GSOD; Patuxent River Naval Air Station (Station ID: 72404013721), accessed August 7, 2012.

2.8 Channel Migration or Diversion

2.8.1 Historical Channel Diversion

The Chesapeake Bay was formed toward the end of the last ice age, which marked the end of the Pleistocene epoch. As the glaciers retreated, large volumes of melting ice resulted in the ancestral Susquehanna River eroding older coastal plain deposits and forming a broad river valley. Subsequently, rising sea levels inundated the continental shelf and reached the mouth of the Chesapeake Bay about 10,000 years ago. Continued sea level rise eventually submerged the ancestral Susquehanna River Valley, creating the Chesapeake Bay. The Chesapeake Bay assumed its present dimensions about 3,000 years ago (Reference 2.8-1).

Given the seismic, topographical, geologic, and thermal evidence in the region, there is very limited potential for upstream diversion or rerouting of the Chesapeake Bay (due to channel migration, river cutoffs, ice jams, or subsidence) and adversely impacting safety-related or important-to-safety facilities or water supplies. Also, there is no major stream outfall to the Chesapeake Bay near the site. Most streams within the CCNPP property area drain towards John Creek and subsequently the Patuxent River. Therefore it is very unlikely that the shoreline would be affected by fluvial processes near the site.

The safety-related intake structure for CCNPP Units 1 & 2 is located on the Chesapeake Bay shore with a surrounding grade elevation of approximately 10 ft NGVD 29. High cliffs, reaching elevations greater than 100 ft NGVD 29, exist up-coast and down-coast of the existing intake structure along the shoreline of the Chesapeake Bay. Approximately 3,700 ft of the shoreline, including the CCNPP Units 1 & 2 intake embayment and the shoreline southeast of the intake structure to the existing barge jetty (as shown in Figure 1.1-3), are stabilized against shoreline erosion. The main plant area of CCNPP Units 1 & 2 is located at a grade elevation of about 45 ft NGVD 29 and set back approximately 300 ft from the Chesapeake Bay shoreline.

Both long-term and short-term sediment processes are responsible for shoreline erosion of the Chesapeake Bay. The slow rise in sea level, approximately 1.3 ft over the last century (Reference 2.8-2), is the primary long-term process causing the shoreline to recede. Waves and surges due to occasional hurricanes may considerably change coastal morphology. These short-term erosive waves often reach the high, upland banks out of the range of normal tides and waves.

Shoreline locations near the CCNPP site in 1848, 1942 and 1993 are shown in Figure 2.8-1 (Reference 2.8-3). The local rate of shoreline change in the vicinity of the CCNPP site, as estimated by the Maryland Department of Natural Resources (MDNR), is shown in Figure 2.8-2 (Reference 2.8-4). The rate of shoreline erosion southeast of the existing barge jetty and near the CCNPP Unit 3 site has been estimated by MDNR to be between 2 ft and 4 ft per year. North of the existing CCNPP Units 1 & 2 intake structure, MDNR has estimated the shoreline change to be between 2 ft per year accretion and 4 ft per year erosion. The stabilized shoreline near the intake structures prevents any shoreline retreat.

Observations of the shoreline near the site indicate that the steep slopes fail along irregular, near-vertical surfaces. These slope failures appear to be caused by shoreline erosion along the base of the cliffs, which results in undercutting a portion of the cliff. When the overlying weight of unconsolidated coastal plain deposits exceeds the shear strength of the soils, a portion breaks away from the cliff and drops to the beach level along a near-vertical failure surface. Shoreline processes, such as waves or tidal currents, erode the deposits that have fallen to the beach and transport the sand, silt and clay materials comprising these deposits along the beach.

The hill slope southeast of CCNPP Units 1 & 2 is recessed from the beach and approximately 3700 ft of the shoreline is protected against erosion by an existing shoreline protection structure as shown in Figure 1.1-3 and Figure 2.8-2. It is therefore unlikely that the shoreline at this location will retreat due to the shoreline erosion processes described above.

The occurrence of shoreline erosion immediately southeast of the barge jetty indicates that the net sediment transport in this area is likely directed towards the southeast with the jetty acting as a sediment barrier. Because water supply to the intake structure is withdrawn from the Units 1 & 2 intake forebay, which is located approximately 2,000 ft northwest of the barge jetty, any failure of steep slopes south of the jetty, as detailed in Figure 1.1-3, is not likely to result in sufficient transport of material north of the jetty. As such, these types of failures are not likely to impact the water supply to the intake structure. Northwest of the existing CCNPP Units 1 & 2 intakes, Figure 2.8-2 indicates a low shoreline erosion potential (between 2 ft per year erosion and 2 ft per year accretion) for a distance of approximately 2,000 ft. Slope failures in this area may drop cliff materials on the beach, which will be gradually eroded and transported by waves and tidal currents. Any failure of this slope is not likely to result in blockage of the water supply to the intake structure, because the sediment transport rates associated with wave action and tidal currents are limiting. Additionally, because the CCNPP Units 1 & 2 power block area is set back approximately 300 ft from the shoreline, it is unlikely that shoreline erosion south of the barge jetty would impact CCNPP Units 1 & 2.

2.8.2 Conclusions

Approximately 3700 ft of the shoreline near the CCNPP Units 1 & 2 plant is protected against shoreline erosion. The low and moderate potential shoreline erosion northwest and southeast of the intake structure is unlikely to affect safe functioning of the intake structure or other safety-related plant SSCs. This conclusion is consistent with the findings in CCNPP Units 1 & 2 UFSAR (Reference 2.8-6).

2.8.3 References

2.8-1 Chesapeake Bay Program, *Chesapeake Bay – Introduction to an Ecosystem*, EPA 903-R-04-003, CBP/TRS 230/00, 2004.

2.8-2 Chesapeake Bay Program, *Sediment in the Chesapeake Bay and Management Issues: Tidal Erosion Processes*, Tidal Sediment Task Force of the Sediment Workgroup under the Nutrient Subcommittee, CBP-TRS276-05, 2005.

2.8-3 Coastal and Estuarine Geology Program, Shoreline Change Maps, Maryland Geological Survey, Website: http://www.mgs.md.gov/coastal/maps/schangepdf.html, Date accessed: January

4, 2007 (Shoreline Changes, Cove Point Quadrangle, MD, Maryland, 7.5 Minute Series (Orthophotoquad), Maryland Department of Natural Resources, Maryland, 2001).

2.8-4 Maryland Department of Natural Resources, Maryland Shorelines Online, Website: http://shorelines.dnr.state.md.us/shoreMapper/standard/, Date accessed: February 7, 2007.

2.8-5 UniStar Nuclear Services, LLC. *Calvert Cliffs Nuclear Power Plant Unit 3 Combined License Application*, Final Safety Analysis Report, Rev 8, March 2012.

2.8-6 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

Figure 2.8-1 Change in the Chesapeake Bay Shoreline Position near the CCNPP Site Between 1848, 1942 and 1993

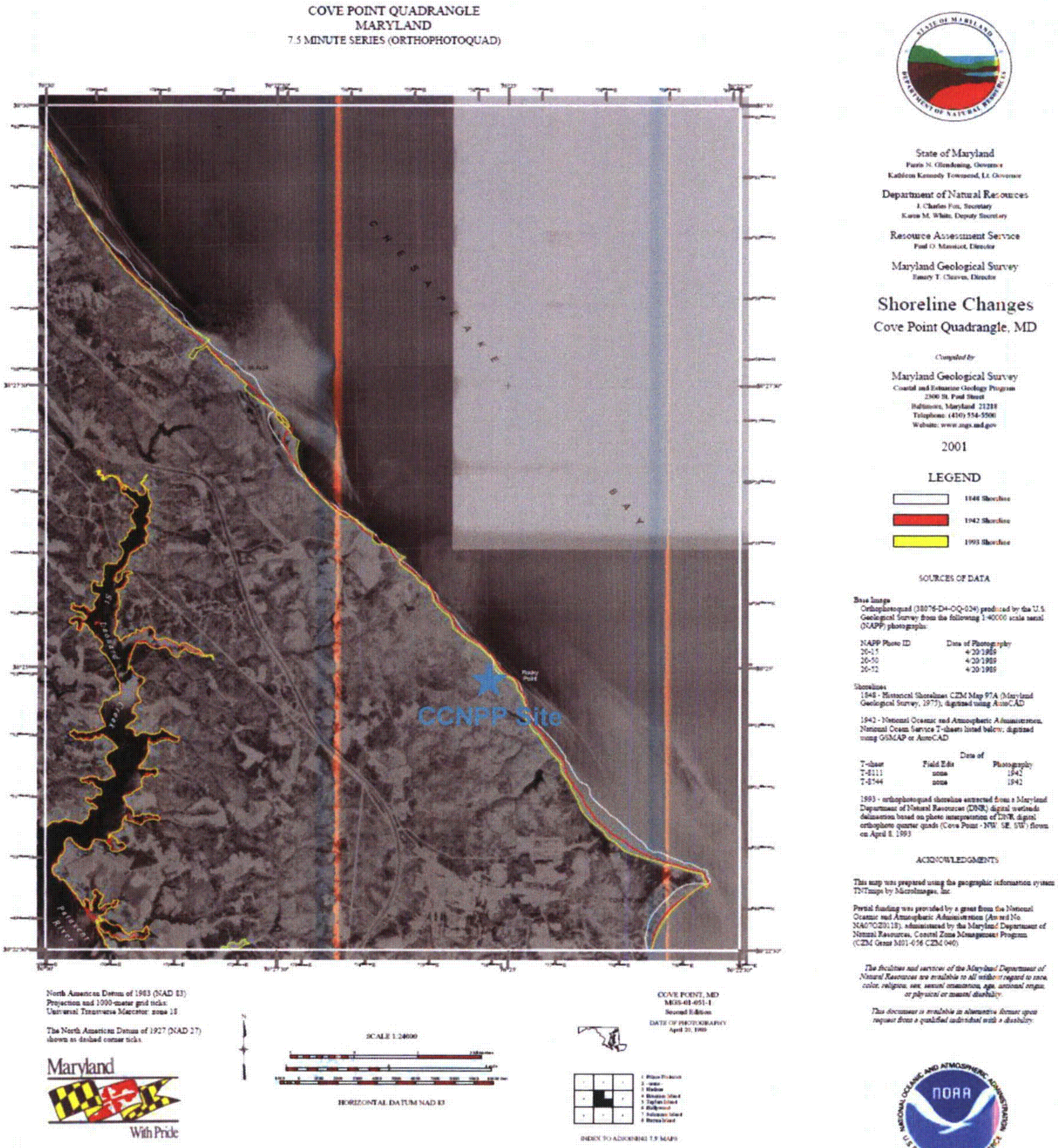
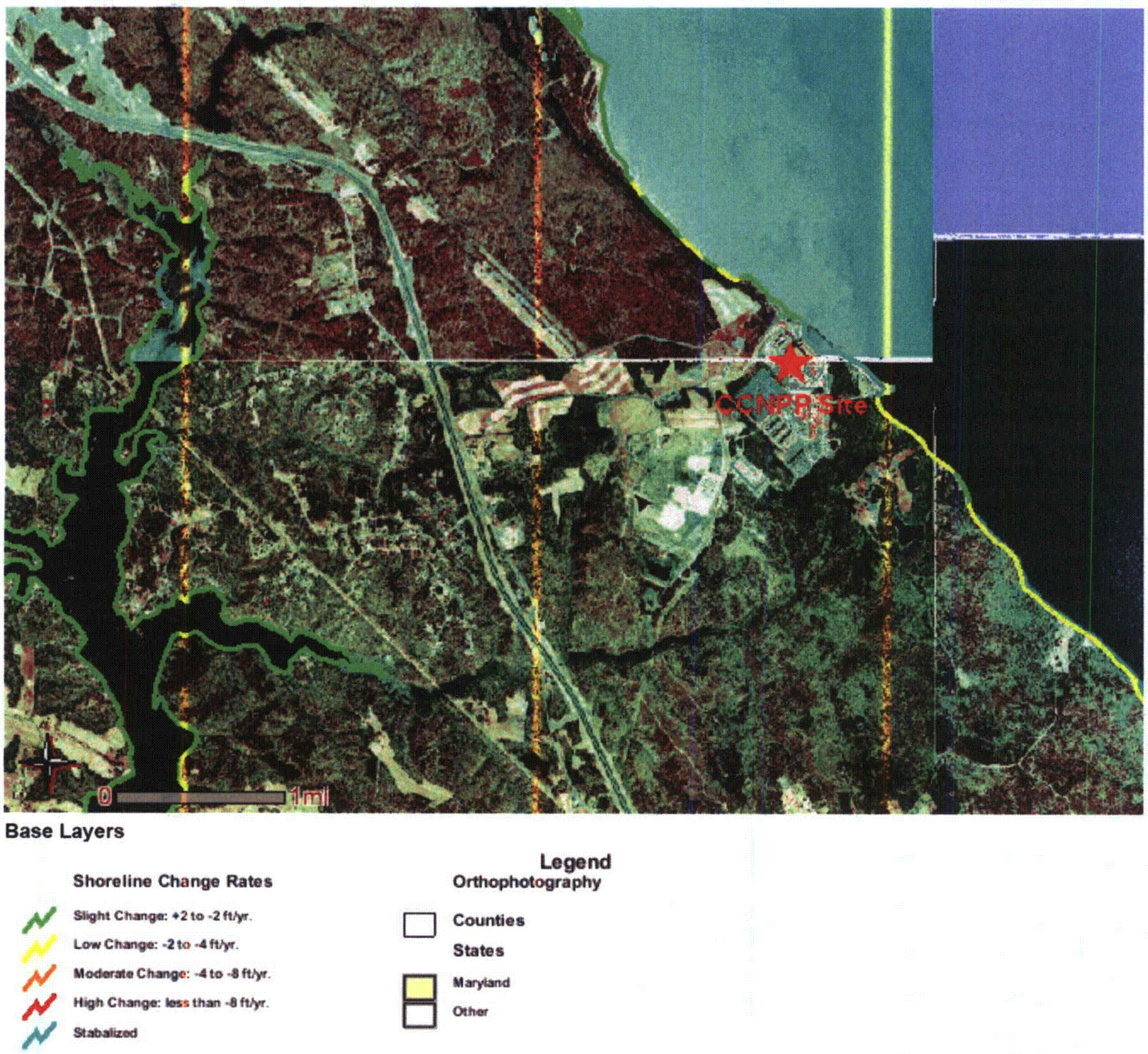


Figure 2.8-2 Chesapeake Bay Shoreline Erosion Rates near the CCNPP Site Estimated by Maryland Department of Natural Resources



2.9 Combined Effect Flooding

Combined effect of different flood-causing mechanisms is evaluated based on the guidelines presented in RG 1.206 (Reference 2.9-1), NUREG/CR-7046 (Reference 2.9-2), NUREG/CR-6966 (Reference 2.9-3), NEI 12-08 (Reference 2.9-4) and ANSI/ANS-2.8-1992 (Reference 2.9-5). Particularly, the evaluation is based on ANSI/ANS-2.8-1992 (Reference 2.9-5) and NUREG/CR-7046 (Reference 2.9-2), which provides detailed guidance on selecting combined events criteria for power reactor site locations on inland streams, open or semi-enclosed bodies of water, and enclosed bodies of water.

Flooding reevaluations for different flood mechanisms are described in Sections 2.1 through 2.8. As identified in the CCNPP Units 1 & 2 UFSAR (Reference 2.9-6) and CCNPP Unit 3 COLA (Reference 2.9-7) and as described in this report, flooding in streams and rivers (Section 2.2), dam failure (Section 2.3), seiche (Section 2.5), ice-induced flooding (Section 2.7), and channel diversion (or shoreline erosion) (Section 2.8) would have no impact to any safety-related or important to safety SSCs of the CCNPP Units 1 & 2. These flood mechanisms are screened out as not applicable to the site based on topographic, geological, and hydrologic setup. Therefore, a combined effect flooding assessment for these mechanisms is not necessary.

Reevaluation the local PMP event in Sections 2.1 identifies that the Auxiliary and Turbine Buildings would be flooded from a local PMP event. Although ANSI/ANS-2.8-1992 (Reference 2.9-5) did not provide combined events criteria for a local PMP event, it specifies that for precipitation flooding (as applicable for streams and rivers) an antecedent rain equal to 40 percent of the PMP or 500-yr rain, whichever is less, should be combined with the PMP event. This combined with the requirement that a sequential cyclonic storm could precede or follow the PMP with a 3 to 5 day period between storms (Reference 2.9-5), the local PMP analysis postulated a rainfall event equivalent to 40 percent of the PMP occurring prior to the PMP event with a 3 to 5 day period between them. The saturated ground condition as a result of the consecutive storms was accounted for in the hydrological model with a runoff curve number of 98, which represents an impervious ground condition to maximize surface runoff.

The effect of hurricane storm surge is reevaluated in Section 2.4, which shows that the intake structure roof would be flooded by the maximum storm surge and coincidental wind wave runup. The combined events criteria in ANSI/ANS-2.8-1992 for a shore location on an open or semi-enclosed body of water (Section 9.2.2.1 in Reference 2.9-5) recommend that the hurricane storm surge be combined with an appropriate antecedent water level condition and wave runup from the hurricane wind. The initial water level used in this study includes the 10 percent exceedance high spring tide of 2.17 ft NGVD 29, initial rise of 1.1 ft, and a long-term sea level rise of 1.07 ft. This provided an antecedent water level of 4.34 ft NGVD 29 for the SLOSH model. The simulated storm surge elevation was then combined with wave runup to estimate the maximum water level at the intake structure. The wave runup was calculated using a significant wave height of 10.9 ft and wave period of 5.6 seconds. The significant wave height was reduced to the breaking wave height of 5.8 ft on the intake deck. The resulting wave runup was calculated to be 13.8 ft assuming an equivalent slope of 16.7 degrees. The PMSS elevation was then combined with wave runup to produce a peak water surface elevation of 31.3 ft NGVD 29.

For the tsunami flooding evaluation, the guidance in NUREG/CR-7046 (Reference 2.9-2) and NUREG/CR-6966 (Reference 2.9-3) with respect to the 10 percent exceedance high spring tide and long-term sea level rise, as described in Section 2.6, are followed.

2.9.1 Conclusions

The reevaluation of flood causing mechanisms for the CCNPP Units 1 and 2 site indicates that the local PMP flooding and the maximum storm surge including wave runup are the controlling flooding mechanisms for the main plant and intake area, respectively. Appropriate combined event criteria for these flooding events were included in the evaluations. Although not a design basis flood event, combined event assessment was made for the maximum tsunami flooding in accordance with NUREG/CR-7046 (Reference 2.9-2) and NUREG/CR-2966 (Reference 2.9-3). Other flood mechanisms are not relevant to the plant and are screened out based on the topographic, geologic and hydrologic characteristics of the site and the nearby region.

2.9.2 References

2.9-1 U.S. Nuclear Regulatory Commission, *Combined License Applications for Nuclear Power Plants,* Regulatory Guide 1.206, June 2007.

2.9-2 U.S. Nuclear Regulatory Commission, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America*, NUREG/CR-7046, November 2011.

2.9-3 U.S. Nuclear Regulatory Commission, *Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America*, NUREG/CR-6966, March 2009.

2.9-4 Nuclear Energy Institute, *Overview of External Flooding Reevaluations*, NEI 12-08, Rev. 0, August 2012.

2.9-5 Determining Design Basis Flooding at Power Reactor Sites, American National Standards Institute/American Nuclear Society, ANSI/ANS-2.8-1992, Nuclear Standard 2.8, 1992.

2.9-6 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

2.9-7 Unistar Nuclear Services, LLC., *Calvert Cliffs Nuclear Power Plant Unit 3, Combined License Application*, Rev. 8, March 2012.

3 COMPARISON OF CURRENT AND REEVALUATED FLOOD-CAUSING MECHANISMS

This section summarizes the comparison of current design basis flood elevations at various safety-related and important-to-safety SSCs against corresponding reevaluated flood elevations from the same flood-causing mechanisms. The comparison shows that the reevaluated flood elevation due to the maximum storm surge exceeds the current design basis flood elevation at the intake structure. Also, the reevaluated flood elevations at the Auxiliary Building and Turbine Building floor entrances/openings are higher than the current design basis flood elevation from local intense precipitation (or local PMP). The Auxiliary Building and Turbine Building floor entrances/openings would be flooded with a varying water depth between 0.1 ft and 2.0 ft for a maximum duration of about 1.5 hr. The personnel and equipment hatches at the grade elevation of 45 ft NGVD 29 for the Containment Buildings are watertight and not affected during the local intense precipitation event. The EDG and the Station Blackout Buildings are not affected during the local intense precipitation event and would remain flood free. Table 3.0-1 shows the comparison of flood elevations from all flood-causing mechanisms for the current and reevaluated design basis conditions. Interim flood protection measures for the safety-related and important-to-safety SSCs are described in Chapter 4 of this report.

There is no safety concern at the power block due to scouring during local intense precipitation as the power block is mostly concrete paved. Also, the risk of debris impact to CCNPP Units 1 & 2 is relatively low under the local intense precipitation event, as the contributing drainage area to the CCNPP Units 1 & 2 power block is mostly covered with an impervious surface with some woods areas. The steep slope between the power block and the substation is covered with riprap and protected by gabion baskets. Therefore, the potential for erosion affecting the safe functioning of the plant is expected to be low.

At the intake structure, the roof would be inundated with a maximum water depth of approximately 2.8 ft at peak storm surge with coincident wave runup. Because the reevaluated storm surge elevation with wave runup exceeded the CLB flood elevation for the intake structure, attendant impacts on the intake structure including pump access hatches and air exhaust vents will be evaluated during the integrated assessment as described in Chapter 5. As described in Section 1.2, the current design of the intake structure accounted for the impacts that sections of baffle wall could dislodge and impact the intake structure as waterborne missiles.

Table 3.0-1 Current Design Basis Flood Elevations for Safety-Related and Important-to-Safety SSCs

Flooding Mechanism	Flood Critical Structure	Current Flood Protection Elevation ft NGVD 29	Current Design Basis Flood Level ft NGVD 29	Reevaluated Flood Level ft NGVD 29
Local Intense Precipitation	Containment Buildings, Auxiliary Building, EDG(1) Building, SBO(2) Building, Turbine Building(3)	45.0, 45.5(4),	44.8(4)	45.1(4) – 47.0
Flooding in Streams and Rivers	No Flooding Expected	No Flooding Expected	No Flooding Expected	No Flooding Expected
Upstream Dam Failures	No Flooding Expected	No Flooding Expected	Not Evaluated	No Flooding Expected
Storm Surge (including wave runup)	Intake Structure	28.5	27.1	31.3
Seiche	No Flooding Expected	No Flooding Expected	Not Evaluated	No Flooding Expected
Tsunami (including runup)	Intake Structure	28.5	No Flooding Expected	11.5 (No Flooding Expected)
Ice Induced Flooding	No Flooding Expected	No Flooding Expected	Not Evaluated	No Flooding Expected
Channel Migration or Diversion(5)	No Flooding Expected	No Flooding Expected	No Flooding Expected	No Flooding Expected

Notes:
(1) Emergency Diesel Generator Building
(2) Station Blackout Building is augmented safety-related
(3) Turbine Building is Seismic Category II; see Chapter 4 for further discussions
(4) At the Emergency Diesel Generator and SBO Buildings
(5) Shoreline protection measures exist and no erosion expected

3.1 Local Intense Precipitation

The CCNPP Units 1 & 2 UFSAR (Reference 3.1-1) indicates that the site drainage design, which employs a network of surface ditches and underground culverts, allows storm water drainage away from the site and eliminates any flooding impact to the site. The plant area has a site grade elevation of approximately 45.0 ft NGVD 29. Several entrance openings with roll-up doors for the Auxiliary and Turbine Buildings are located at site grade.

The UFSAR also evaluates a flooding elevation at the EDG and SBO Buildings, where site grading design provides a system of swales that direct surface runoff from the local PMP event to the Chesapeake Bay without producing drainage or flooding problems for the buildings. The runoff to the Chesapeake Bay does not depend on the site's storm drain system. The results of the runoff and backwater analyses indicate that during the PMP storm, the swale system will convey the surface runoff with a maximum water level of 44.8 ft NGVD 29 near the Diesel Generator Buildings (EDG and SBO). This water level is below the floor grade of the Diesel Generator Buildings, which is 45.5 ft NGVD 29, and thus eliminates the potential for flooding of the Diesel Generator Buildings during the local PMP event. The local PMP analysis used HMR 51 and HMR 52 and employed U.S. Army Corps of Engineers (USACE) computer program HEC-1 and HEC-2 to compute surface runoff and peak water levels for the ditches and swales (Reference 3.1-1).

The reevaluated flooding impacts due to the local intense precipitation are presented in Section 2.1. The reevaluation also used HMR 51 and HMR 52 to obtain the point rainfall intensities and used USACE computer models HEC-HMS and HEC-RAS to simulate peak runoff and corresponding maximum water levels in the drainage paths within the power block area. The analysis is performed assuming underground storm drains and culverts, as well as roof drains are clogged and not functioning during the local PMP storm event. The simulated water levels in the power block area varied between 45.1 ft to 47.0 ft NGVD 29 resulting in a range of flooding water depths from 0.1 ft to 2.0 ft at the entrances/openings of the Auxiliary and Turbine Buildings. The simulated peak flow velocities varied between 0.8 ft/s to 10.0 ft/s, which are not expected to produce any erosion hazards as described in Section 2.1. The EDG and SBO Buildings are not affected during the local intense precipitation event as the maximum water level of 45.1 ft NGVD 29 remains below the floor entrance elevation of 45.5 ft NGVD 29. Interim flooding protection measures for the Auxiliary and Turbine Buildings are described in Chapter 4.

3.1.1 References

3.1-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

3.2 Flooding in Streams and Rivers

The CCNPP Units 1 & 2 UFSAR (Reference 3.2-1) explains that because the topography at the site is gently rolling with steeper slopes along stream courses and that the site is well drained by short, intermittent streams, it is unlikely that the CCNPP site will be subject to any flooding. A drainage divide, which is generally parallel to the coastline, extends across the site as shown in Figure 1.1-2. The area to the east of the divide comprises about 20 percent of the site and includes the plant area. This area drains into the Chesapeake Bay. The area west of the drainage divide is drained by tributaries of Johns Creek and Woodland Branch, which flow into St. Leonard Creek and subsequently into the Patuxent River. It is possible that high intensity rain storms may cause water to back up in some valleys due to local constrictions in the stream beds, but this would be a temporary situation.

Reevaluation of PMF induced flooding, as discussed in Section 2.2, identifies three streams that could potentially impact the flood level within the CCNPP property boundary. The first is Johns Creek and its tributary (Branch 3), located southwest of the CCNPP site west of the drainage divide, as shown in Figure 1.1-2. The other two are unnamed creeks, Branch 1 and Branch 2, located southeast of CCNPP Units 1 & 2 and east of the drainage divide.

Branch 1 and Branch 2 drain small basin areas south-southwest of the CCNPP Units 1 & 2 site and discharge directly to the Chesapeake Bay. Flooding in these streams will not affect the safety functioning of the CCNPP Units 1 & 2 plant.

As described in Section 2.3, a PMF analysis performed on Johns Creek indicates a maximum PMF water surface elevation of 65 ft NGVD 29 on Johns Creek near the CCNPP site. However, because the drainage divide west of the CCNPP Units 1 & 2 site is located at an elevation higher than 100 ft NGVD 29, CCNPP Units 1 & 2 would not be impacted from hazards associated with flooding in streams and rivers.

3.2.1 References

3.2-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

3.3 Dam Breaches and Failures

The CCNPP Units 1 & 2 UFSAR (Reference 3.3-1) does not include assessment of any flooding potential due to dam break in the vicinity of the site.

The reevaluation assessment, while it did not identify any dam on St. Leonard Creek or Johns Creek, identified two dams – Brighton Dam and Rocky Gorge Dam – on the Patuxent River. As evaluated in Section 2.3, the increase in water level in the Patuxent River near the site due to hypothetical simultaneous failures of the two dams where the entire reservoir water volume is instantaneously added to the Patuxent River water without escape to the Chesapeake Bay would be less than 2 ft. This water level rise in the Patuxent River could create a backwater condition for St. Leonard Creek and possibly Johns Creek. However, this hypothetical water level rise is not expected to have any flooding impact to the safety functioning of the CCNPP Units 1 & 2 site, even when combined with a PMF or a similar event. Consequently, reevaluated flooding due to dam breaches or failures will not exceed current flood design basis.

3.3.1 References

3.3-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

3.4 Storm Surge

The PMSS constitutes the flooding design basis at the intake structure. The CCNPP Units 1 & 2 UFSAR (Reference 3.4-1) uses the following methodology to develop the PMSS elevation at the site:

- PMH parameters were identified from NOAA technical report HUR 7-97, the predecessor to NWS Technical Report NWS-23. The PMH parameters include:
    - Central pressure of 26.94 inches of Mercury
    - Asymptotic pressure of 30.12 inches of Mercury, which provides a central pressure deficit of 3.18 inches of Mercury or 107.7 mb
    - Radius of maximum wind of 30 statute mi
    - Forward speed of 23 mi/hr
    - Maximum wind speed of 124.7 mi/hr at the radius of maximum wind
    - PMH path that approaches the coast from the east, curving northward after it passes inland west of the Chesapeake Bay
- The PMH wind field developed from the PMH parameters used the methodology described in report HUR 7-97.
- The storm surge elevation in the open water in front of the Chesapeake Bay entrance was obtained following methods described in USACE Technical Report 4. The maximum hurricane surge of 17.32 ft NGVD 29 coincidental with the normal high tide was obtained at the Chesapeake Bay entrance.
- Hurricane surge levels within the bay were evaluated adopting the methodology proposed by Bretschneider in USACE Miscellaneous Paper 3-59. The surge elevation of 15.6 ft NGVD 29 was obtained near the site.
- Coincident significant wave height and peak period were estimated at 11.4 ft and 9.0 seconds, respectively
- Wave runup was confirmed by physical scale model tests, which show a maximum wave runup elevation of 27.1 ft NGVD 29 with a runup height of 9.5 ft corresponding to a maximum (1 percent) wave height of 14 ft. The final model tests included an adverse slope for the intake structure. The runup elevation from physical model tests did not overtop the intake structure top elevation at 28.5 ft NGVD 29.

The reevaluated maximum storm surge uses NWS-23 to define the PMH parameters, which are used in the NOAA's storm surge simulation model SLOSH. The reevaluation also adopts the HHA approach, as described in NUREG/CR-7046 (Reference 3.4-2). The final set of parameters is selected based on a detailed parameter sensitivity analysis, as described in Section 2.4. The selected parameters are as below:

- PMH parameters are from NWS-23. The PMH parameters include:
    - Central pressure deficit between 118-124 millibars at landfall along with options for steady-state and decaying intensity hurricane
    - Radius of maximum wind of 28 to 40 nautical mi
    - Forward speed of 15.7 to 30.5 mi/hr
    - Maximum wind speed of 111 mi/hr at the intake structure
    - PMH path similar to that of Hurricane Connie except that the hurricane track bends and approaches the Chesapeake Bay shoreline near the site at a normal direction (Section 2.4)
- The PMH parameters were evaluated in the SLOSH model so that the surge elevation at the site could be maximized.
- A range of still water levels between 17.5 and 19.9 ft NGVD 29 was obtained from SLOSH model results.
- Estimated significant wave height is 10.9 ft with breaking wave height varying between 5.84 and 10.9 ft at the intake structure with and without accounting for the deck width, respectively.
- Wave runup varies between 13.8 and 37.2 ft for the range of wave and intake configuration considered.
- The reevaluated storm surge elevation including wind-wave runup was obtained as (17.5 + 13.8 =) 31.3 ft NGVD 29.

As can be seen from the reevaluation, the PMSS elevation including wave runup exceeds the roof elevation of the 28.5 ft NGVD 29, which could potentially affect the roof top hatch to the saltwater system pumps. Interim flood protection measures for the intake structure are described in Section 4.3.

3.4.1 References

3.4-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43., 2012.

3.4-2 U.S. Nuclear Regulatory Commission, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America*, NUREG/CR-7046, November 2011.

3.5 Seiche

Although no evaluation on seiche was performed as part of the current flood design basis for the CCNPP Units 1 & 2 site, seiche flooding is reevaluated in Section 2.5 from published literature. Records of water level variations at various locations in the Chesapeake Bay show evidence of seiche oscillation. However, there is no major forcing event that governs the water level in the Chesapeake Bay that will coincide with the bay's natural frequency of about 2 days. It is therefore concluded that seiche flooding will not have impact to any safety-related or important-to-safety facilities of the CCNPP Units 1 & 2 plant.

3.6 Tsunami

The CCNPP Units 1 & 2 UFSAR (Reference 3.6-1) provides a summary of recorded historical tsunami runups in the U.S. east coast. Based on the evaluation of the historical tsunami data, the UFSAR concludes that the occurrence of tsunamis in the Atlantic Ocean is infrequent, and it is unlikely that the site would be subjected to a significant tsunami effect. Also, the maximum expected tsunami would result in only minor wave action, and the maximum expected storm surge and coincidental wind-wave effects would be the critical flood design basis.

The tsunami flooding reevaluation study, as presented in Section 2.6, identifies potential tsunami sources in the Atlantic Ocean and evaluates from literature potential tsunami heights at the mouth of the Chesapeake Bay. The study then simulates tsunami propagation in the bay and tsunami wave height near the site using depth-integrated two-dimensional numerical models with both non-linear and linear approximations. Combining with the antecedent water level condition as prescribed in NUREG/CR-7046 (Reference 3.6-2) and NUREG/CR-6966 (Reference 3.6-3) and accounting for the long-term sea level rise, the maximum expected tsunami water level including tsunami runup at the site is obtained at 11.5 ft NGVD 29. This elevation remains below the intake structure roof elevation of 28.5 ft NGVD 29 and therefore the maximum tsunami flooding would have no impact to the safe functioning of the CCNPP Units 1 & 2.

3.6.1 References

3.6-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

3.6-2 U.S. Nuclear Regulatory Commission, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America*, NUREG/CR-7046, November 2011.

3.6-3 U.S. Nuclear Regulatory Commission, *Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America*, NUREG/CR-6966, March 2009.

3.7 Ice Induced Flooding

Ice induced flooding at a nuclear power plant site could occur by the following mechanisms:

- Breach of ice jams causing flooding at the site
- Ice blockage of the drainage system causing flooding

Although CCNPP Units 1 & 2 UFSAR (Reference 3.7-1) summarizes historical snowstorms and impacts to transmission lines due to freezing precipitation, there is no discussion on ice induced flooding from the two mechanisms. Historical data characterizing ice conditions at the CCNPP site were reevaluated for potential flooding impacts at the CCNPP Units 1 & 2 site. These data include ice cover and thickness observations in the Chesapeake Bay and its tributaries, ice jam records, and long-term air temperature measurements from the nearby Patuxent River Naval Air Station meteorological tower (WBANID 13721) (Reference 3.7-2). The results of the evaluation of historical temperature data in the vicinity of CCNPP Units 1 & 2 and a search of the USACE Ice Jam Database (Reference 3.7-3) indicate that ice induced flooding is not a credible hazard that will adversely impact the safety functions of the plant.

The flood hazard to the safety-related or important-to-safety facilities due to temporary blockage of site drainages by any mechanism including an ice event is evaluated as part of the local PMP analysis documented in Section 2.1.

3.7.1 References

3.7-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

3.7-2 NCDC Climate Data Online, NOAA. Available at http://www7.ncdc.noaa.gov/CDO/cdoselect.cmd?datasetabbv=GSOD; Patuxent River Naval Air Station (Station ID: 72404013721), accessed August 7, 2012.

3.7-3 Ice Jam Database, U.S. Army Corps of Engineers. Available at https://rsgisias.crrel.usace.army.mil/icejam/, accessed September 19, 2012

3.8 Channel Migration or Diversion

The CCNPP Units 1 & 2 UFSAR (Reference 3.8-1) provides a summary of historical shoreline change near the plant. Based on the evaluation of the historical shoreline position data and field measurement of shoreline location from an unidentified monument location southeast of the site it was concluded that shoreline erosion could continue at a maximum rate of 2 ft (0.6 m) per year. Also, approximately 3700 lineal ft of shore protection was placed in front of the plant area and the shore protection consisted of onsite material placed in front of the cliffs and faced with filter cloth and layered riprap.

The reevaluation of shoreline erosion potential indicates that near the CCNPP Units 1 & 2 intake structure, the maximum shoreline erosion rate remained nearly the same (up to 1993) as documented by MDNR (Reference 3.8-2). The stabilized shoreline near the intake structures prevents any shoreline retreat. Any failure of cliff slopes near the site is not likely to result in blockage of the water supply to the intake structure. The results of the revaluation indicate that channel migration or diversion is not a credible hazard that will adversely impact the safety functions of the plant.

3.8.1 References

3.8-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

3.8-2 Maryland Department of Natural Resources, Maryland Shorelines Online, Website: http://shorelines.dnr.state.md.us/shoreMapper/standard/, Date accessed: February 7, 2007.

3.9 Combined Effect Flooding

Combined effect flooding is discussed as part of the reevaluation of flood-causing mechanisms, where applicable, based on the guidelines presented in RG 1.206 (Reference 3.9-1), NUREG/CR-7046 (Reference 3.9-2), NUREG/CR-6966 (Reference 3.9-3), NEI 12-08 (Reference 3.9-4) and ANSI/ANS-2.8-1992 (Reference 3.9-5). The combined effect for three flooding mechanisms, local intense precipitation, storm surge and tsunami, are compared to the current licensing basis in the following paragraphs. Other flooding mechanisms are screened out based on the topographic, geologic and hydrologic settings of the site and the surrounding region.

The local PMP analysis performed for the EDG Building in the Units 1 & 2 UFSAR (Reference 3.9-6) conservatively assumed the entire contributing catchment area as impervious. The analysis then used a runoff curve number of 98 to represent the impervious surface condition. The reevaluated local PMP analysis combined an antecedent rain equal to 40 percent of the local PMP, 3 to 5 dry days between them, as recommended in the ANSI/ANS-2.8-1992 (Reference 3.9-5). To account for the saturated ground condition due to the antecedent rain, the reevaluation performed in Section 2.1 also used a runoff curve number of 98. This combined events condition is comparable to the reevaluation as summarized in Section 2.9.

CCNPP Units 1 & 2 UFSAR (Reference 3.9-6) used the normal high tide and sea level anomaly of approximately 1.35 ft NGVD 29 as the antecedent water level for the storm surge estimation. The estimated storm surge elevation of 17.6 ft NGVD 29 was then combined with the wind-wave runup (9.5 ft) on the intake structure determined through the physical model tests for the estimated maximum (1 percent) wave height of 14.0 ft. The maximum storm surge water level thus obtained was (17.6 + 9.5 =) 27.1 ft NGVD 29. This combined events condition is comparable to the reevaluation as summarized in Section 2.9.

While the CLB qualitatively evaluates to screen out tsunami impacts to the plant, the reevaluation performed numerical model simulations of tsunami wave propagation, as documented in Section 2.6. Following the guidance in NUREG/CR-7046 (Reference 3.9-2) and NUREG/CR-6966 (Reference 3.9-3) the ambient conditions of 10 percent exceedance high spring tide with consideration of long-term sea level rise is used in the PMT estimate.

The results of the combined effects analysis has not introduced any additional flood hazards beyond those described above.

3.9.1 References

3.9-1 U.S. Nuclear Regulatory Commission, *Combined License Applications for Nuclear Power Plants,* Regulatory Guide 1.206, June 2007.

3.9-2 U.S. Nuclear Regulatory Commission, *Design-Basis Flood Estimation for Site Characterization at Nuclear Power Plants in the United States of America*, NUREG/CR-7046, November 2011.

3.9-3 U.S. Nuclear Regulatory Commission, *Tsunami Hazard Assessment at Nuclear Power Plant Sites in the United States of America*, NUREG/CR-6966, March 2009.

3.9-4 Nuclear Energy Institute, *Overview of External Flooding Reevaluations*, NEI 12-08, Rev. 0, August 2012.

3.9-5 Determining Design Basis Flooding at Power Reactor Sites, American National Standards Institute/American Nuclear Society, ANSI/ANS-2.8-1992, Nuclear Standard 2.8, 1992.

3.9-6 Calvert Cliffs Nuclear Power Plant Inc., Updated Final Safety Analysis Report, Rev. 43, 2012.

4 INTERIM EVALUATION AND ACTIONS TAKEN OR PLANNED

4.1 Regulatory Background

The NRC 10 CFR 50.54(f) Request For Information letter dated March 12, 2012, (Reference 4.1-1) 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 the NRC (Reference 4.1-1) 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 flooding 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.

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.

4.1.2 References

4.1-1 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*, ADAMS Accession No. ML12053A340, March 12, 2012.

4.2 Interim Flood Protection Measures for Auxiliary and Turbine Buildings

Chapter 2 of this report contains the reevaluation of flood hazards at CCNPP Units 1 & 2 using the present-day regulatory guidance and methodologies. Section 3.0 of this report summarizes the comparison of the flood elevation to the reevaluated flood elevation for each applicable flood-causing mechanism. Two reevaluated flood mechanisms, local PMP and PMSS for both CCNPP Unit 1 and CCNPP Unit 2 exceeded the design basis flood and are discussed below. It should be noted that PMT evaluation has identified a resultant water level of 11.5 ft NGVD 29. As stated in Section 2.6, this flood level, which is significantly less than the PMSS, and the associated effects are bounded by the PMSS analysis. Therefore, no additional actions are required.

The reevaluated local PMP flood duration and flood elevation have increased. This will impact the amount of water ingress into safety-related structures, specifically at the Auxiliary Building personnel access and rollup doors. The Turbine Building doors are also susceptible to flooding, but it has been determined that in a worst-case water intrusion, all doors are assumed open and that the resultant water depth within the building would not affect any safety-related equipment. Condition report CR-2013-001914 has been entered into the corrective action process to address the PMP issue.

The PMSS water level including wave runup has increased above the current CLB. Specifically the reevaluated storm surge level will overtop the intake structure roof. This surge will cause ingress of the water into the intake structure through the ventilation louvers. This water ingress could affect the operation of the salt water pump motors, which are safety-related SSCs. Condition report CR-2013-001913 has been entered into the corrective action process to address the PMSS issue.

Since the PMSS and the local PMP flood elevations for CCNPP Units 1 & 2 are not bounded by the CLB flood elevations at the site, 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.

4.2.1 Local PMP Reevaluation

Sections 2.1 and 3.1 explain that some of the safety-related and important-to-safety SSCs would be flooded during the local intense precipitation events. These SSCs include the safety-related Auxiliary Building and the non-safety-related Turbine Building. The interim flooding protection measures and flood management approach that will be undertaken for these structures are described below.

4.2.1.1 Auxiliary Building

The exterior door openings or entrances from outside to the Auxiliary Building are located along the West Plant Road on the southwestern wall of the building. The area along the West Plant Road and the Auxiliary Building has a grade elevation of approximately 45.0 ft NGVD 29. The entrance elevation for the Auxiliary Building is located at the grade elevation. There are two types of entrances for the Auxiliary Building: personnel access doors and equipment access roll-up doors. Personnel access doors are typically small, 3 ft to 3.67 ft wide, while the width of the roll-up doors varies from about 11 ft to 16 ft. The southwestern wall of the Auxiliary Building is fitted with six personnel access doors and three roll-up doors that would require interim flood protection measures.

As described in Subsection 2.1, these doors will be exposed to a maximum flooding depth of about 2.0 ft during the local PMP event. As can be inferred from the flow hydrographs presented in Section 2.1, the 2.0 ft maximum flood depth will sustain for a very short duration (less than 15 min) near the southwest corner of the plant. The total PMP flood level would remain above grade level for about 90 minutes, however this duration encompasses the entire event which includes slow buildup to the maximum level of 2 ft above grade and subsequent level decrease to a non-flood condition. Because these doors are not watertight and water entry into the Auxiliary Building may affect the safe functioning of the plant, appropriate flooding protection measures are planned for these doors and entrances for the local PMP event.

4.2.1.2 Turbine Building Flood Protection

The Turbine Building is a Seismic Category II non-safety-related structure. However, it houses the safety-related Auxiliary Steam Generator Feed Pumps, Auxiliary Control Panel and Service Water Pumps and provides access to CCNPP 1 & 2 intake structure at sub-grade elevations. The Turbine Building also provides access to Units 1 & 2 Control Rooms at 45 ft NGVD 29. The Turbine Building floors include openings and stair wells that would guide water flows to the basement floor elevation at 12 ft NGVD 29 if external flooding of the Turbine Building occurs.

The CCNPP Units 1 & 2 UFSAR (Reference 4.2-1) considers potential internal flooding of the Turbine Building due to a rupture in the circulating water expansion joint while the circulating water pump is in operation. The UFSAR (Table 9-17A, Chapter 9) (Reference 4.2-1) indicates that the incident would flood the Turbine Building, but the flooding elevation would remain below the critical flood elevation of 18 ft NGVD 29. The Auxiliary Steam Generator Feed Pumps, Auxiliary Control Panel, Service Water Pumps, and Intake Structure are protected by watertight doors, and flooding in the Turbine Building below elevation 18 ft NGVD 29 will not affect any safety-related equipment. A conservative estimate of the time required for the floodwater to reach 18 ft NGVD 29 in the Turbine Building is approximately 28 minutes based on the operating flow rate of 260,000 gpm of the circulating water pump at a level of 12 ft NGVD 29 and 230,000 gpm above 12 ft NGVD 29. Due to the increasing flood elevation and the low head of the pump, the UFSAR (Reference 4.2-1) concludes that the time required for floodwater to affect the safe functioning of the plant would be greater than 40 minutes.

The provisions for internal flooding suggest that flooding from external events in the Turbine Building, such as during a local intense precipitation event, will have no safety impact given that (1) the flood flows from the external event is guided to the basement floor without affecting the safe functioning of the plant along its path, and (2) the flood depth at the basement level remains below the critical flood elevation of 18 ft NGVD 29. An evaluation was conducted to estimate floodwater inflow to the Turbine Building during a local PMP event if the doors and accesses to the building were not flood protected. The floodwaters that enter the Turbine Building would need to flow to the lowest (basement floor) elevation through the openings and stair wells on the Turbine Building floors without affecting the CCNPP Units 1 & 2 Control Room access doors. Section 2.1 shows that the northwest wall of the Unit 1 Turbine Building will be flooded to a maximum water depth of about 0.1 ft while the southeast wall of the Unit 2 portion of the Turbine Building will be flooded to a maximum water depth of about 0.9 ft. The outside access doors along the southwestern wall of the Turbine Building, one for each Unit, would be flooded to a higher maximum water depth, 0.3 ft for Unit 1 and 2 ft for Unit 2. The entrances and openings include 3 ft to 3.67 ft wide personnel access doors and 16-ft-wide roll-up doors.

Roof runoff from the Turbine Building and the North Service Building may accumulate on the concrete terraces northwest and southeast of the North Service Building. The runoff from these areas would drain towards the Chesapeake Bay flowing over the steep slope to the northeast. The runoffs from the roof of the Turbine and North Service Buildings are conservatively estimated using the rational method and the 5-minute local PMP rainfall intensity. The runoff may accumulate temporarily on the terraces and ingress into the Turbine Building through two doors, one for each Unit.

The total flood water volume that may enter the Turbine Building during the local PMP event is estimated conservatively assuming a broad-crested weir type flow through the completely open doors and entrances. The floodwater is also assumed to remain at the maximum water depth for the entire duration of flooding above 45 ft NGVD 29. Using a weir coefficient of 3.0, the total flood water volume that may enter the Turbine Building from the local PMP event is approximately 760,000 cu ft. The water volume from the Turbine Building internal flooding can be estimated using a conservative combination of circulating water pump flow rate and flooding duration. Consequently, the water volume from the rupture of the expansion joint can be conservatively estimated when the circulating water pump is operating at the smaller flow rate of 230,000 gpm and for 28 minutes. The resulting water volume obtained is approximately 861,000 cu ft.

The local PMP flood inflow volume to the Turbine Building is smaller than the water volume from the rupture of the circulating water expansion joint when the circulating water pump is operating at a flow rate of 230,000 gpm and for 28 minutes. This comparison shows that the safe functioning of the plant will not be affected if the floodwater from the local PMP event is allowed to enter the Turbine Building and guided to the basement level. The resulting water level in the Turbine Building will remain below the critical flood level of 18 ft NGVD 29. Accordingly, no flood protection measures for the Turbine Building doors or entrances at the grade level of 45 ft NGVD 29 would need to be implemented.

However, the site severe weather procedure will be revised to mitigate this water intrusion either through sandbagging, or the use of other commercially available flood barriers like inflatable barriers, self inflating flood bags etc., at the access paths or implementing measures to ensure that any floodwater that enters the Turbine Building would be guided to the lowest elevation without affecting the safe functioning of the plant.

4.2.2 References

4.2-1 Calvert Cliffs Nuclear Power Plant Inc., *Updated Final Safety Analysis Report*, Rev. 43, 2012.

4.3 Interim Actions

Interim actions will be taken by CCNPP to provide a level of assurance the plants will be safe during a flood event until the total plant response to the reevaluated hazard is determined by the integrated assessment.

The local PMP and the PMSS events do not cause an immediate flooding concern at CCNPP. The time between the prediction of a severe precipitation event including local PMP and the potential flooding event will be greater than 24 hours, giving the plant time to initiate potential flood mitigation measures. The notification time for a storm surge including the PMSS resulting from a severe hurricane including the PMH is greater than 24 hours and allows for interim actions to mitigate these impacts.

Calvert Cliffs Nuclear Power Plant Station Administrative Procedure, EP-1 – 108, Severe Weather Preparation (Reference 4.3-1), will be revised to include the interim actions discussed below.

4.3.1 Interim Evaluations and Actions Planned

For a local intense precipitation event, the immediate actions that will be put in place are the use of sand bags, or the use of other commercially available flood barriers like inflatable barriers, self inflating flood bags etc., at the access/entrance points as well as procedure changes to ensure the access pathway doors are closed for all doors susceptible to the reevaluated local PMP level. Based on the short duration of flooding, 90 minutes, and the short duration of the maximum flood height of 2 ft, it is reasonable to assume that these actions will be sufficient to preclude any effect on safety-related SSC's. Additional actions being evaluated include verification that the drainage paths are not blocked as opposed to that currently assumed in the reevaluated PMP analysis, as well as the use of portable pumps to remove any accumulated water in the Auxiliary Building.

The final long-term action and final design of flood protection measures will be determined during the integrated assessment.

For the storm surge event, to mitigate the reevaluated PMSS elevation and minimize the water ingress into the intake structure, CCNPP will provide panel covers, which will be placed over the intake structure ventilation louvers. These covers would only be put in place prior to the arrival of the hurricane at the CCNPP site. The lead time available for the installation of these covers will coincide with a unit shutdown to comply with the Plant Technical Procedure ERPIP-3.0 Attachment 20, Severe Weather (Reference 4.3-2). As the cause of the PMSS is a result of the PMH, it is intended to have these covers and associated procedures in place prior to the start of the 2013 hurricane season. These louvers currently have existing brackets for these covers and have supporting calculations. Fabrication of new aluminum or equivalent waterproof material covers and procedures for installation will be complete by June 2013. The use of these covers will be evaluated as a permanent solution to the reevaluated PMSS during the integrated assessment.

The lead time available for the installation of these covers and the placement of the sand bags will not present any additional burden on operations personnel as they will be installed by maintenance personnel well in advance of either flooding event.

4.3.2 References

4.3-1 Constellation Energy, *Severe Weather Preparation, Calvert Cliffs Nuclear Power Plant Station Administrative Procedure EP-1-108*, Revision 00400.

4.3-2 Constellation Energy, *Immediate Actions, Calvert Cliffs Nuclear Power Plant Technical Procedure ERPIP-3.0*, Revision 05100.

5 ADDITIONAL ACTIONS

5.1 Regulatory Background

As stated in Section 4.1, Regulatory Background, the following long term action is required:

> *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.*

CCNPP will be performing an integrated assessment in accordance with JLD- ISG-2012-05, Guidance for Performing an Integrated Assessment for Flooding (Reference 5.1-1).

5.1.1 References

5.1-1 U.S. Nuclear Regulatory Commission, *Guidance for Performing the Integrated Assessment for External Flooding*, Japan Lessons Learned Project Directorate, Internal Staff Guidance, JLD-ISG-2012-05, Revision 0, November 2012.

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 Calvert Cliffs Nuclear Power Plant, Unit 1 and 2.	March 12, 2015
Implement interim actions to address higher flooding hazards, relative to the current design basis, as described in this submittal.	June 1, 2013