UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555-0001

April 16, 2015

Mr. George H. Gellrich, Vice President
Exelon Generation Company, LLC
Calvert Cliffs Nuclear Power Plant
1650 Calvert Cliffs Parkway
Lusby, MD 20657-4702

SUBJECT: CALVERT CLIFFS NUCLEAR POWER PLANT, UNITS 1 AND 2 – STAFF ASSESSMENT OF RESPONSE TO 10 CFR 50.54(f) INFORMATION REQUEST – FLOOD-CAUSING MECHANISM REEVALUATION (TAC NOS. MF3097 AND MF3098)

Dear Mr. Gellrich:

By letter dated March 12, 2012, the U.S. Nuclear Regulatory Commission (NRC) issued a request for information pursuant to Title 10 of the *Code of Federal Regulations*, Section 50.54(f) (hereafter referred to as the 50.54(f) letter). The request was issued as part of implementing lessons learned from the accident at the Fukushima Dai-ichi nuclear power plant. Enclosure 2 to the 50.54(f) letter requested licensees to reevaluate flood-causing mechanisms using present-day methodologies and guidance.

By letter dated March 12, 2013, Calvert Cliffs Nuclear Power Plant, LLC responded to this request for Calvert Cliffs Nuclear Power Plant Units 1 and 2. This response was supplemented by letters dated February 10, 2014, and March 27, 2014.

The NRC staff has reviewed the information provided and, as documented in the enclosed Staff Assessment, determined that you provided sufficient information in response to the 50.54(f) letter. Because the reevaluated flood-causing mechanism was not bounded by your current plant-specific design-basis hazard, the NRC staff anticipates submittal of an integrated assessment in accordance with Enclosure 2, Required Response 3, of the 50.54(f) letter. In addition, the staff has identified an issue that resulted in an open item. This open item is documented and explained in the attached staff assessment and need to be addressed as part of the integrated assessment.

If you have any questions, please contact me at (301) 415-3733 or email at Robert.Kuntz@nrc.gov.

Sincerely,

Robert F. Kuntz, Senior Project Manager
Hazards Management Branch
Japan Lessons-Learned Division
Office of Nuclear Reactor Regulation

Docket Nos. 50-317 and 50-318

Enclosure:
Staff Assessment of Flood Hazard
Reevaluation Report

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STAFF ASSESSMENT BY THE OFFICE OF NUCLEAR REACTOR REGULATION

RELATED TO FLOODING HAZARD REEVALUATION REPORT

NEAR-TERM TASK FORCE RECOMMENDATION 2.1

RELATED TO THE FUKUSHIMA DAI-ICHI NUCLEAR POWER PLANT ACCIDENT

CALVERT CLIFFS NUCLEAR POWER PLANT, UNITS 1 AND 2

DOCKETS NO. 50-317 AND 50-318

1.0 INTRODUCTION

By letter dated March 12, 2012 (NRC, 2012a), the U.S. Nuclear Regulatory Commission (NRC) issued a request for information to all power reactor licensees and holders of construction permits in active or deferred status, pursuant to Title 10 of the *Code of Federal Regulations* (10 CFR), Section 50.54(f) "Conditions of license" (hereafter referred to as the "50.54(f) letter"). The request was issued in connection with implementing lessons learned from the 2011 accident at the Fukushima Dai-ichi nuclear power plant as documented in the "Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident." (NRC, 2011b).[1] Recommendation 2.1 in that document recommended that the staff issue orders to all licensees to reevaluate seismic and flooding for their sites against current NRC requirements and guidance. Subsequent Staff Requirements Memoranda associated with Commission Papers SECY-11-0124 (NRC, 2011c) and SECY-11-0137 (NRC, 2011d), directed the NRC staff to issue requests for information to licensees pursuant to 10 CFR 50.54(f).

Enclosure 2 to the 50.54(f) letter requested that licensees reevaluate flood hazard for their respective sites using present-day methods and regulatory guidance used by the NRC staff when reviewing applications for early site permits (ESPs) and combined licenses (COLs). The required response section of Enclosure 2 specified that NRC staff would provide a prioritization plan indicating Flooding Hazard Reevaluation Report (FHRR) deadlines for individual plants. On May 11, 2012, the staff issued its prioritization of the FHRRs (NRC, 2012b).

If the reevaluated hazard for all flood-causing mechanisms is not bounded by the current plant design basis flood hazard, an integrated assessment will be necessary. The FHRR and the responses to the associated requests for additional information (RAIs) will provide the hazard input necessary to complete the integrated assessment report as described in Enclosure 2, Required Response 3, of the 50.54(f) letter.

[1] Issued as an enclosure to Commission Paper SECY-11-0093 (NRC, 2011a).

By letter dated March 12, 2013 (CENG, 2013a), Calvert Cliffs Nuclear Power Plant, LLC (the licensee) provided the FHRR for the Calvert Cliffs Nuclear Power Plant (Calvert Cliffs, CCNPP), Units 1 and 2. The licensee stated in FHRR Section 4.3 that interim actions and procedures exist and that these interim actions and procedures will be reevaluated and updated, as determined by the integrated assessment. The licensee responded to the staff's RAIs by letters dated February 10, 2014 (CENG, 2014b), and March 7, 2014 (CENG, 2014c). Because a reevaluated flood-causing mechanism is not bounded by the current plant-specific design-basis hazard, the staff anticipates submittal of an integrated assessment. The staff will prepare an additional staff assessment to document its review of the integrated assessment.

The licensee submitted a separate Flooding Walkdown Report associated with Near-Term Task Force Recommendation 2.3 (CENG, 2012b). The staff has prepared a separate staff assessment to document its review of the licensee's Flooding Walkdown Report (NRC, 2014b).

2.0 REGULATORY BACKGROUND

2.1 Applicable Regulatory Requirements

As stated above, Enclosure 2 to the 50.54(f) letter requested that licensees reevaluate flood hazards for their respective sites using present-day methods and regulatory guidance used by the NRC staff when reviewing applications for ESPs and COLs. This section describes present-day regulatory requirements that are applicable to the FHRR.

Section 50.34(a)(1), (a)(3), (a)(4), (b)(1), (b)(2), and (b)(4), of 10 CFR, describes the required content of the preliminary and final safety analysis reports (FSARs), including a discussion of the facility site with a particular emphasis on the site evaluation factors identified in 10 CFR Part 100. The licensee should provide any pertinent information identified or developed since the submittal of the preliminary safety analysis report in the FSAR.

Section 50.54(f) of 10 CFR states that a licensee shall at any time before expiration of its license, upon request of the Commission, submit written statements, signed under oath or affirmation, to enable the Commission to determine whether or not the license should be modified, suspended, or revoked. The 50.54(f) letter, requested licensees reevaluate the flood-causing mechanisms for their respective sites using present-day methodologies and regulatory guidance used by the NRC for the ESP and COL reviews.

General Design Criterion 2 in Appendix A of 10 CFR Part 50 states that structures, systems, and components (SSCs) important to safety at nuclear power plants must be designed to withstand the effects of natural phenomena such as earthquakes, tornados, hurricanes, floods, tsunamis, and seiches without loss of capability to perform their intended safety functions. The design bases for these SSCs are to reflect appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area. The design bases are also to have sufficient margin to account for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

Section 50.2 of 10 CFR defines the design-basis as the information that identifies the specific functions that an SSC of a facility must perform, and the specific values or ranges of values chosen for controlling parameters as reference bounds for design which each licensee is required to develop and maintain. These values may be (a) restraints derived from generally accepted "state of the art" practices for achieving functional goals, or (b) requirements derived from an analysis (based on calculation or experiments, or both) of the effects of a postulated accident for which an SSC must meet its functional goals.

Section 54.3 of 10 CFR defines the "current licensing basis" as: "the set of NRC requirements applicable to a specific plant and a licensee's written commitments for ensuring compliance with and operation within applicable NRC requirements and the plant-specific design basis (including all modifications and additions to such commitments over the life of the license) that are docketed and in effect." This includes 10 CFR Parts 2, 19, 20, 21, 26, 30, 40, 50, 51, 52, 54, 55, 70, 72, 73, 100 and appendices thereto; orders; license conditions; exemptions; and technical specifications, as well as the plant-specific design-basis information, as documented in the most recent FSAR. The licensee's commitments made in docketed licensing correspondence, which remain in effect, are also considered part of the current licensing basis.

Present-day regulations for reactor site criteria (Subpart B to 10 CFR Part 100 for applications on or after January 10, 1997) state, in part, that the physical characteristics of the site must be evaluated and site parameters established such that potential threats from such physical characteristics will pose no undue risk to the type of facility proposed to be located at the site. Factors to be considered when evaluating sites include the nature and proximity of dams and other man-related hazards (10 CFR 100.20(b)) and the physical characteristics of the site, including the hydrology (10 CFR 100.21(d)).

2.2 Enclosure 2 to the 50.54(f) Letter

The 50.54(f) letter requests all power reactor licensees and construction permit holders reevaluate all external flooding-causing mechanisms at each site. The reevaluation should apply present-day methods and regulatory guidance that are used by the NRC staff to conduct ESP and COL reviews. This includes current techniques, software, and methods used in present-day standard engineering practice. If the reevaluated flood-causing mechanisms are not bounded by the current plant design-basis flood hazard, an integrated assessment will be necessary.

2.2.1 Flood-Causing Mechanisms

Attachment 1 to Recommendation 2.1, Flooding (Enclosure 2 of the 50.54(f) letter) discusses flood-causing mechanisms for the licensee to address in its FHRR. Table 2.2-1 lists the flood-causing mechanisms the licensee should consider. Table 2.2-1 also lists the corresponding *Standard Review Plan* (NRC, 2007) section(s) and applicable interim staff guidance (ISG) documents containing acceptance criteria and review procedures. The licensee should incorporate in the flooding reevaluation associated effects with each flood-causing mechanism per Japan-Lessons Learned Directorate (JLD) JLD-ISG-2012-05 (NRC, 2012c) in addition to the maximum water level associated.

2.2.2 Associated Effects

In reevaluating the flood-causing mechanisms, the "flood height and associated effects" should be considered. JLD-ISG-2012-05 (NRC, 2012c), defines "flood height and associated effects" as the maximum stillwater surface elevation plus:

- wind waves and runup effects
- hydrodynamic loading, including debris
- effects caused by sediment deposition and erosion
- concurrent site conditions, including adverse weather conditions
- groundwater ingress
- other pertinent factors

2.2.3 Combined Effects Flood

The worst flooding at a site that may result from a reasonable combination of individual flooding mechanisms is sometimes referred to as a "Combined Effects Flood." Even if some or all of these individual flood-causing mechanisms are less severe than their worst-case occurrence, their combination may still exceed the most severe flooding effects from the worst-case occurrence of any single mechanism described in the 50.54(f) letter. (See the Standard Review Plan, Section 2.4.2, Area of Review 9 (NRC, 2007)). Attachment 1 to Recommendation 2.1, Flooding (Enclosure 2 of the 50.54(f) letter) describes the "Combined Effect Flood"[2] which are defined in American National Standards Institute/American Nuclear Society (ANSI/ANS) 2.8-1992 (ANSI/ANS, 1992) as follows:

> For flood hazard associated with combined events, American Nuclear Society (ANS) 2.8-1992 provides guidance for combination of flood causing mechanisms for flood hazard at nuclear power reactor sites. In addition to those listed in the ANS guidance, additional plausible combined events should be considered on a site-specific basis and should be based on the impacts of other flood causing mechanisms and the location of the site.

If two less severe mechanisms are plausibly combined (per ANS-2.8-1992 (ANSI/ANS, 1992) and Standard Review Plan, Section 2.4.2, Areas of Review 9 (NRC, 2007)), then the staff will document and report the result as part of one of the hazard sections. An example of a situation where this may occur is flooding at a riverine site located where the river enters the ocean. For this site, storm surge and river flooding should be plausibly combined.

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[2] For the purposes of this Staff Assessment, the terms "combined effects" and "combined events" are synonyms.

2.2.4 Flood Event Duration

Flood event duration was defined in the ISG for the integrated assessment for external flooding, JLD-ISG-2012-05 (NRC, 2012c), as the length of time during which the flood event affects the site. It begins when conditions are met for entry into a flood procedure, or with notification of an impending flood (e.g., a flood forecast or notification of dam failure), and includes preparation for the flood. It continues during the period of inundation, and ends when water recedes from the site and the plant reaches a safe and stable state that can be maintained indefinitely. Figure 2.2-1 illustrates flood event duration.

2.2.5 Actions Following the FHRR

For the sites where the reevaluated flood probable maximum flood (PMF) elevation is not bounded by the current design-basis flood PMF elevation for all flood-causing mechanisms, the 50.54(f) letter requests licensees and construction permit holders to:

- Submit an Interim Action Plan with the FHRR documenting actions planned or already taken to address the reevaluated hazard
- Perform an integrated assessment subsequent to the FHRR to (a) evaluate the effectiveness of the current design basis (i.e., flood protection and mitigation systems), (b) identify plant-specific vulnerabilities, and (c) assess the effectiveness of existing or planned systems and procedures for protecting against and mitigating consequences of flooding for the flood event duration.

If the reevaluated flood hazard is bounded by the current design-basis flood hazard for all flood-causing mechanisms at the site, licensees are not required to perform an integrated assessment at this time.

3.0 TECHNICAL EVALUATION

The NRC staff has reviewed the information provided for the Calvert Cliffs, Units 1 and 2 flood hazard reevaluation. The licensee conducted the hazard reevaluation using present-day methodologies and regulatory guidance used by the NRC staff in connection with ESP and COL reviews. The staff's review and evaluation is provided below.

The licensee's flood hazard reevaluation studies were conducted using customary units of measure. In this report, customary measurements are followed by the equivalent measurement in metric units. Because the conversion to metric units may involve loss of precision, the measurement in conventional units is definitive. loss of precision, the measurement in primary units is definitive.

To provide additional information in support of the summaries and conclusions in the FHRR, the licensee made calculation packages available to the staff via an electronic reading room. When the staff relied directly on some of these calculation packages in its review; these calculation packages are docketed, and are cited, as appropriate, in the discussion below. Certain other calculation packages were found only to expand upon and clarify the information provided on the docket, and so are not docketed or cited.

The staff requested additional information from the licensee to supplement the FHRR by letter dated January 9, 2014 (NRC, 2014a). The licensee provided this additional information by letters dated February 10, 2014 (CENG, 2014b), and March 7, 2014 (CENG, 2014c). The licensee's responses are discussed in the appropriate sections(s) below.

The site grade at the powerblock is 45.0 ft (13.7 m) National Geodetic Vertical Datum of 1929 (NGVD29). Unless otherwise stated, elevations in this staff assessment are given with respect to the NGVD29. Table 3.0-1 provides the summary of controlling reevaluated flood-causing mechanisms, including associated effects, the licensee computed to be higher than the powerblock elevation.

3.1 Site Information

The 50.54(f) letter includes the SSCs important to safety, and the ultimate heat sink, in the scope of the hazard reevaluation. Per the 50.54(f) letter, Enclosure 2, Requested Information, Hazard Reevaluation Report, Item a, the licensee included pertinent data concerning these SSCs in the FHRR.

The 50.54(f) letter, Enclosure 2 (Recommendation 2.1: Flooding), Requested Information, Hazard Reevaluation Report, Item a, describes site information to be contained in the FHRR. The staff reviewed and summarized this information as follows.

3.1.1 Detailed Site Information

The FHRR describes the Calvert Cliffs site, which is summarized bellow. The Calvert Cliffs site is located in Calvert County, Maryland, approximately 10.5 mi (16.9 km) southeast of Prince Frederick, Maryland, and on the western shore of the Chesapeake Bay, approximately 110 mi (180 km) north from the Chesapeake Bay entrance. The Calvert Cliffs site covers approximately 2,057 acres (8,324,000 m^2). Calvert Cliffs, Units 1 and 2 and ancillary facilities are located on approximately 932 acres (3,770,000 m^2).

The topography at the Calvert Cliffs, Units 1 and 2 site is gently rolling with steeper slopes along stream banks. Local relief ranges from sea level up to an approximate elevation of 130 ft (40 m), with an average elevation of approximately 100 ft (30 m). Along the northeastern perimeter of the Calvert Cliffs site, the Chesapeake Bay shoreline consists mostly of steep cliffs with a narrow beach area.

The Calvert Cliffs site is well drained by short, ephemeral streams. A drainage divide, which is roughly parallel to the shoreline, extends across the Calvert Cliffs site, as shown in Figure 3.1-1. The area to the northeast of the divide, which lies within the Maryland Western Shore Watershed, comprises about 20 percent of the Calvert Cliffs site property, includes Calvert Cliffs, Units 1 and 2, and drains into the Chesapeake Bay. The area southwest of the divide, which lies within the Patuxent River Watershed, is drained by tributaries of Johns Creek. These 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 km) to the southeast from the mouth of St. Leonard Creek. All streams that drain the Calvert Cliffs site that are located east of Maryland State Highway 2/4 are non-tidal.

Calvert Cliffs State Park is southeast of Calvert Cliffs, Units 1 and 2. Flag Ponds Nature Park is northwest of the Calvert Cliffs site on the western shore of the Chesapeake Bay. The Calvert Cliffs, Units 1 and 2 powerblock is located within the 1,000 ft (305 m) critical area along the shoreline, as defined by Maryland's Critical Area Commission law.

Calvert Cliffs, Units 1 and 2 safety-related and important-to-safety SSCs were constructed across three nearly level terraces, as described in the Flooding Walkdown Report ((CENG, 2012b). The safety-related intake structure is located at a deck elevation of 10.0 ft (3.05 m), safety-related and important-to-safety SSCs in the main plant area are located at a grade elevation of about 45.0 ft (13.7 m), and plant substation (switchyard) and administrative buildings are located at a grade elevation of about 70.0 ft (21.3 m).

Critical elevations for the safety-related and important-to-safety SSCs are summarized in Table 3.1-1, and locations of these SSCs are shown in Figure 3.1-2.

The safety-related facilities for Calvert Cliffs, Units 1 and 2, including the Containment Buildings, Auxiliary Building, Emergency Diesel Generator Building, and Station Blackout Diesel Generator Building, are located in the main plant area where the grade elevation is about 45.0 ft (13.7 m). The Turbine Building is located east of the Auxiliary Building, which is classified as Seismic Category II, and houses the safety-related Auxiliary Feedwater System pumps located at a sub-grade elevation of 12.0 ft (3.66 m). The Turbine Building also provides access to Calvert Cliffs, Units 1 and 2 Control Rooms.

The safety-related Calvert Cliffs, Units 1 and 2 Intake Structure is located on the Chesapeake Bay shoreline, as shown in Figure 3.1-2, and is classified as safety-related as it houses the saltwater pumps that are essential for safe shutdown. The intake structure includes a 50.0 ft (15.2 m) wide, open deck and has openings for the trash rakes and racks, stop logs, and traveling screens. The roof of the pump building is at elevation 28.5 ft (8.69 m) and has watertight hatches to provide access to the pumps for maintenance.

3.1.2 Design-Basis Flood Hazards

The current design-basis flood levels are summarized by flood-causing mechanism in Table 3.1-2. For Calvert Cliffs, these mechanisms are described in the Calvert Cliffs, Units 1 and 2 Updated FSAR(UFSAR) (CENG, 2011).

3.1.3 Flood-related Changes to the Licensing Basis

The FHRR stated that there have been no changes to design-basis flooding elevations or flooding protection designs beyond what is described in the Calvert Cliffs, Units 1 and 2 UFSAR, Revision 43 (CENG, 2011). The licensee stated that the flooding walkdown also did not identify any deficiency in the current flooding protection measures.

The FHRR stated that vertical vehicle barriers were built on the landward sides of Calvert Cliffs Units 1 and 2. These vehicle barriers could potentially change the drainage flow paths near the plant and divert runoff which otherwise would flow toward the plant. These vehicle barriers are included in the site drainage analysis due to a local intense precipitation (LIP) event as described in the FHRR.

3.1.4 Changes to the Watershed and Local Area

The FHRR stated that 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 (Maryland Department of Natural Resources (MDNR), n.d.-a). In addition, 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 (MDNR, n.d.-a).

The FHRR stated that while watershed changes have taken place in the Maryland Lower Western Shore or Lower Patuxent River watersheds since Calvert Cliffs, Units 1 and 2 were built, these changes did not affect the site, which is located at the headwater areas in these subwatersheds. The licensee also applied for a combined license for a new unit within the Calvert Cliffs site. The new unit, Calvert Cliffs, Unit 3, would be located south-southeast of the Calvert Cliffs, Units 1 and 2 plant area (UniStar, 2012). 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 Calvert Cliffs, Units 1 and 2, as described in FHRR Section 2.1.

3.1.5 Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

The FHRR stated that the maximum storm surge elevation in the Chesapeake Bay from a probable maximum hurricane (PMH) constitutes the design basis flood elevation for the Calvert Cliffs, Units 1 and 2 Intake Structure. According to the Calvert Cliffs, Units 1 and 2 UFSAR, Revision 43, Section 2.8.3.6 (CENG, 2011), 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 Calvert Cliffs, Units 1 and 2 (Constellation Energy, n.d.).

According to the FHRR, the maximum storm surge elevation, including wind-wave runup, at the intake structure is 27.1 ft (8.3 m) with a flood protection elevation of 28.5 ft (8.7 m). 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.

Calvert Cliffs, Units 1 and 2 drainage facilities are designed to drain runoff from intense precipitation events away from the plant. Local probable maximum precipitation (PMP) results in a flood elevation of 44.8 ft (13.7 m) at the Emergency Diesel Generator Building. Floor elevation of this building is 45.5 ft (13.9 m), which prevents flooding of the building from 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 45.0 ft (13.7 m).

3.1.6 Additional Site Details to Assess the Flood Hazard

The licensee provided the following information for models used to assess the flooding hazard:

- HMR 52 computer program (Hansen, Schreiner, and Miller, 1982) data input and output used for PMP determination.
- Topography and elevation data used in a geographic information system to develop site-specific topography and contours.
- HEC-1, HEC-2, HEC-HMS, and HEC-RAS model input and output used for flooding analysis.[3]
- Tabulated streamflow information used for flooding analyses.

3.1.7 Results of Plant Walkdown Activities

The 50.54(f) letter was sent to licensees on March 12, 2012. Enclosure 4 of the 50.54(f) letter requested that licensees plan and perform plant walkdown activities to verify that current flood protection systems are available, functional, and implementable. Other parts of the 50.54(f) letter (Requested Information Item 1.c, and Step 6 of Attachment 1 to Recommendation 2.1, Flooding (Enclosure 2)) asked the licensee to report any relevant information from the results of the plant walkdown activities.

By letter dated November 27, 2012, the licensee provided the flood walkdown report for Calvert Cliffs, Units 1 and 2 (CENG, 2012b). By Letter dated dated June 26, 2014 (NRC, 2014b), the staff prepared a staff assessment to document its review of the walkdown report.

3.2 <u>LIP and Associated Site Drainage</u>

The licensee's FHRR includes a reevaluation of the flood hazard, including associated effects, from LIP.

The reevaluated flood hazard elevation for LIP is 45.1 ft (13.7 m) to 47.0 ft (14.3 m), depending on the location at the site. The reevaluated LIP flood depth near the entrance of the Auxiliary Building, the South Service Building and the Turbine Building varies between 0.1 ft (0.03 m) to 2.0 ft (0.61 m). Other SSCs are not affected by these flood water-surface elevations.

This flood-causing mechanism is described in the licensee's current design-basis. The FHRR states that the current design-basis hazard for the LIP and associated site drainage hazards is 44.8 ft (13.7 m) as described in Sections 2.5, 2.6, and 2.8 of the Calvert Cliffs, Units 1 and 2 UFSAR, Revision 43 (CENG, 2011).

The licensee provided estimates of flood hazards from LIP and described the capacity of the site to drain the flood water away from the site. However, the licensee did not evaluate the PMF for LIP associated with the proposed Haul Road and the Branch 1 and 2 drainage areas, which discharge to the Chesapeake Bay southwest of the site. The licensee stated that stormwater drainage associated with the proposed Haul Road area would not affect CCNPP, Units 1 and 2. The Branch 1 and 2 drainage areas are farther from CCNPP, Units 1 and 2, than the Haul Road drainage area, which abuts the area analyzed by the licensee. The staff confirms that stormwater drainage associated with the proposed Haul Road and Branch 1 and 2 drainage areas would not affect Calvert Cliffs, Units 1 and 2.

3.2.1 LIP-Depth Analysis

Tables 3.2-1 and 3.2-2 provide PMP inputs and depths supplied by the licensee in the FHRR. Section 2.1.2 of the FHRR stated that a review of historical precipitation records for Maryland and Virginia since the publication of National Oceanic and Atmospheric Administration (NOAA) Hydrometeorological Reports Nos. 51 (Schreiner and Riedel, 1978) and 52 (Hansen, Schreiner, and Miller, 1982) identified no events approaching or exceeding the PMP provided therein. The staff reviewed that information and found that the licensee's conclusions are reasonable.

3.2.2 Runoff Analyses

The licensee used the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) software to evaluate runoff and hydrologic routing for each of the six subbasins. Figure 3.2-1 shows the subbasins, and Figure 3.2-2 shows the node-link (subbasin-junction) schematic of the subbasins, as represented throughout the HEC-HMS analysis. The subbasins were assumed to be nearly impervious, with a runoff curve number of 98. The times of concentration for the subbasins were estimated using Natural Resources Conservation Service (NRCS) methodologies. The subbasin peak discharges were estimated using the HEC-HMS NRCS dimensionless unit hydrograph option. Table 3.2-1 presents the drainage area and time of concentration for each subbasin.

The PMP depths shown in Table 3.2-2 were temporally distributed using the HEC-HMS Meteorological Model module (USACE, 2010). The FHRR referenced PMP depths for all durations, excluding the 15-min duration, and the HEC-HMS frequency storm option of the Meteorological Model to develop a 6-hour PMP storm event (CENG, 2013a, Section 2.1.2). The FHRR stated that the 2- and 3-hour duration PMP depths were 21.82 in (55.42 cm) and 23.96 in (60.86 cm), respectively.

The FHRR stated that the HEC-HMS model incorporates topographic information used to support the FSAR for the Combined License Application (COLA FSAR) for Calvert Cliffs, Unit 3 (Unistar, 2012). The stormwater runoff from Subbasins 1, 2, and 3 combine at Junction J-2 near the southern corner of the powerblock, where the flow is diverted into two downstream flows paths, Reaches R-2 and R-3, which direct flow around the powerblock. Reaches R-1 and R-2 also are identified as Downstream 1 and 2 reaches, which flow to the southeast and southwest of the powerblock, respectively (see Figure 3.2-2). These downstream reaches discharge into the Chesapeake Bay at Outlet 1 and 2. Runoff from Subbasin 5 is routed through Outlet 1. Runoff from Subbasins 4 and 6 are routed through Outlet 2. Small storm drainage ditches and culverts are assumed to not function for the purposes of the PMF calculations.

The licensee's results from the HEC-HMS analysis are summarized in Table 3.2-3. The staff's review confirms that the licensee-estimated peak discharges correspond with the contributing area. The staff's review also confirms that the reevaluated hazard is not bounded by the current design-basis and thus requires an integrated assessment to be performed. The staff notes that additional information is needed to determine the adequacy of the LIP analysis, as indicated in Integrated Assessment Open Item 1 in Section 3.2.4 of this report.

The staff requested through an RAI, electronic versions of the HEC-HMS model input files used in the LIP analyses (NRC, 2014a). In its March 7, 2014, response, the licensee provided the requested files for staff's review (CENG, 2014c). After reviewing the information provided, the staff determines that the effect of uncertainty in the subbasin slope had not been sufficiently addressed by the licensee. The staff determines that this uncertainty could have a potentially significant impact on the PMP runoff lag time and the peak local intense precipitation flow rates. The staff further determines that the sheet flow characteristic length, blockage and conservatism of vehicle barriers, and roof drainage partitioning were not discussed. The staff conducted additional sensitivity analyses, and also reviewed the LIP characterization, as well as model boundary and initial conditions.

The staff conducted limited sensitivity analyses of these parameters, and also reviewed the LIP characterization, as well as model boundary and initial conditions. The staff altered the site parameters that were used to estimate the timing of the LIP runoff and evaluated that sensitivity of the model results. The staff determines that no significant change in the licensee's conclusion would be likely given reasonable additional conservatisms in parameter selections.

3.2.3 Water Level Determination

The FHRR referenced the Hydrologic Engineering Center's River Analysis System (HEC-RAS) software (USACE, 2008b) to estimate water-surface elevations near the powerblock area for LIP events, with the primary input being the peak flows simulated by HEC-HMS.

The FHRR stated that a hydraulic model using HEC-RAS was developed. This model was configured with an upstream reach and two downstream reaches with intermediate cross sections to incorporate the effects of site structures by representing them as obstructions to flow. The two downstream reaches receive flow from the upstream reach and flow around the powerblock. The division of flow at the junction of the upstream and downstream reaches is computed using a momentum balance method. A normal depth boundary condition was used at the upstream boundary of the upstream reach. Critical depth boundary conditions were used at the downstream boundary of the downstream reaches. Overflows in the two HEC-RAS downstream reaches were computed using lateral weir equations. Channel and overbank area roughness coefficients were set to values consistent with rough impervious pavement. The site grade is shown in Figure 3.1-1, and Figures 3.2-3 and 3.2-4 show the reach and cross section configuration.

For selected cross sections, Table 3.2-4 provides HEC-HMS results developed by the licensee for the LIP analysis, including the water-surface elevations, depths, and durations of flooding at SSCs located at the South Service Building, Turbine Building, Auxiliary Building and Diesel Generator Building. Table 3.2-4 shows a comparison of building entrance and PMF water-surface elevations, as well as flood duration.

In order to review the formulation of complex spatially and temporally distributed HEC-RAS model input, the staff requested through an RAI electronic versions of the HEC-RAS input files used for local intense precipitation analysis (NRC, 2014a). In its March 7, 2014 response, the licensee provided the requested input files (CENG, 2014c). Through an alternative HEC-RAS simulation, the staff determined that an alternative roof drainage could minimally increase the peak water-surface elevation resulting from the LIP event in the plant area above the licensee's base model. The staff specifically found that, if the turbine and reactor building drained precipitation off the west building edge rather than to the northwest, higher water-surface elevations would occur. The staff determined that while the alternative roof drainage could raise the peak water-surface elevations, the licensee's implementation of the LIP conceptual model was adequately represented by the HEC-RAS simulations.

In order to review and determine the appropriateness of the process followed to estimate slopes and flowpaths, the staff requested a description of the methods used to incorporate elevation measurements into the HEC-RAS and HEC-HMS analyses (NRC, 2014a). The licensee provided a description of its methods and the likely magnitude of the errors associated with these elevations (CENG, 2014c), stating that, near the site, 90 percent of the elevations were within 0.5 ft (0.2 m) of the 1-ft (0.3-m) contour intervals developed for the site and that the remaining 10 percent were within 1.0 ft (0.30 m). The licensee also indicated that 90 percent of spot elevations were within 0.25 ft (0.076 m) of the contour interval and the remaining were within 0.5 ft (0.2 m) of the contour interval. The licensee also stated that uncertainty associated with the elevation measurements was small when the other conservatisms in the local intense precipitation analysis (e.g., Manning's n, reduced times of concentration, impervious surfaces, non-operational storm drains, etc.) are considered.

The staff conducted a sensitivity analysis to evaluate the effect of elevation uncertainty on the estimated maximum water-surface elevations beyond the elevation uncertainty analysis described in Section 3.2.2. The staff altered channel cross section elevations in the HEC-RAS model including decreasing the elevations at the upstream ends of the flowpaths that occur along the west and south edges of the plant area, varying the elevation profile for each flow path was altered independently, and lowering the elevation of the upper cross section by 0.5 ft (0.2 m). The staff observed that the peak water-surface elevations were not significantly different than those found by the licensee.

After reviewing the HEC-HMS and HEC-RAS models, the staff confirms that the site-specific formulations of the HEC-HMS and HEC-RAS models are appropriate. The staff determines that assumptions made by the licensee are not likely to alter the licensee's conclusions regarding triggering an integrated assessment.

3.2.4 Staff Conclusion

The staff confirms the licensee's conclusion that the reevaluated flood hazard for LIP and associated site drainage is not bounded by the current design-basis flood hazard; therefore, the licensee must include LIP and associated site drainage within the scope of the integrated assessment. The information on flooding from LIP and associated site drainage that is specific to the data needs of the integrated assessment is described in Section 4 of this staff assessment.

The staff notes that longer duration LIP events may result in higher flood elevations and longer periods of inundation than the six-hour event. The staff notes that PMP events having relatively short durations may result in limiting warning time and may result in consequential LIP flood elevation (e.g., flood elevations above the openings to plant structures). Therefore, the staff determines that, as part of the integrated assessment report, the licensee should consider a range of rainfall durations associated with the LIP hazard events (e.g., 1-, 6-, 12-, 24-, 48-, 72-hour PMPs) to determine the controlling scenario(s) for evaluation as part of the integrated assessment (see NRC, 2012c). This should include a sensitivity analysis to identify potentially limiting scenarios with respect to plant response when considering flood height, relevant associated effects, and flood event duration parameters for LIP events. This is **Integrated Assessment Open Item No. 1.**

3.3 Streams and Rivers

The FHRR reported that the reevaluated hazard, including associated effects, for streams and rivers does not inundate the plant site, but did not report a probable maximum flood elevation for the plant site. This flood-causing mechanism is not described in the licensee's current design-basis.

The licensee identified three streams that could flood areas within the Calvert Cliffs property boundary: Johns Creek, to the southwest of the site, and two unnamed streams (Branch 1 and 2), to the southeast of the site. These streams are shown in Figure 3.1-1.

Johns Creek flows west into St. Leonard Creek, a tributary of the Patuxent River, and is part of the Lower Patuxent River Watershed discharging into the Chesapeake Bay. Johns Creek drainage is bounded on the east by the drainage boundary at an elevation of 98.0 ft (29.9 m) (Unistar, 2012). The FHRR stated that based on the dominant tidal influence of the water levels in the Patuxent River and St. Leonard Creek, flood water-surface elevations in the two would not significantly affect PMF elevations near the Calvert Cliffs site.

The drainage divide between the Maryland Lower Western Shore Watershed and the Lower Patuxent River Watershed is at an elevation of about 100 ft (30 m) near the site (see Figure 3.1-1). A culvert passing under Maryland State Highway 2/4 (MD 2/4) is a controlling feature of the Johns Creek flow as shown on Figure 3.1-1. The MD 2/4 roadway is at an elevation of about 45 ft (14 m). Assuming that the MD 2/4 culvert is blocked, the roadway becomes the controlling downstream land feature for backwater effects toward the Calvert Cliffs site. Based on the approximately 55 ft (17 m) elevation difference between the MD 2/4 roadway and the drainage divide, and the small drainage areas contributing to runoff upstream of the MD 2/4 crossing, flooding in Johns Creek would be unlikely to cause an overtopping of the drainage divide resulting in flood flow directed toward the Calvert Cliffs site. The staff confirms the licensee's determination that flooding from Johns Creek would not result in inundation of the Calvert Cliffs site (NRC, 2013b).

The other two streams which drain near the site (Branch 1 and 2) discharge to the east into the Chesapeake Bay, but drain very small areas relative to the total drainage area of the Chesapeake Bay. The FHRR stated that it is implausible that these streams would have a significant and bounding impact on the water-surface elevations on the Calvert Cliffs site (see Figure 3.1-1). Based on information provided in the FHRR and additional information obtained through RAIs, the staff confirms the licensee's conclusion that the reevaluated hazard for flooding from streams and rivers alone does not inundate the plant site.

3.4 Failure of Dams and Onsite Water Control/Storage Structures

The FHRR reported that the reevaluated hazard, including associated effects, for site flooding from failure of dams and onsite water control or storage structures does not inundate the plant site. This flood-causing mechanism is not described in the licensee's current design-basis.

The FHRR estimated the potential effects of a flood caused by dam breaches on the tributaries of Patuxent River, including the St. Leonard Creek and Johns Creek drainages.

The FHRR noted that there are no dams on Johns Creek or St. Leonard Creek, but there are two dams on the Patuxent River upstream of the mouth of St. Leonard Creek: Rocky Gorge Dam and Brighton Dam. These dams are located about 65 mi (100 km) and 78 mi (130 km), respectively, upstream from the mouth of St. Leonard Creek and have a combined maximum storage volume of about 49,000 acre-ft (60,000,000 m^3).

The FHRR stated that the analysis assumed that the combined volumes of water stored behind the dams were instantaneously introduced to the tidally affected region (with an area of 40.9 mi^2 (106 km^2)) near the mouth of the Patuxent River. Further, the analysis assumed no discharge from this region to the Chesapeake Bay to maximize the water-surface elevation. The licensee estimated that the water-surface elevation in the tidal region would increase about 2 ft (0.6 m) but this increase would not alter its earlier estimate of the PMF water-surface elevation in Johns Creek.

The staff examined the locations and storage volumes of dams within the Patuxent River Watershed based on the data from the National Inventory of Dams (USACE, n.d.-a) and confirms that Rocky Gorge Dam and Brighton Dam are the only dams that could potentially affect the Calvert Cliffs site. The staff estimated the combined maximum storage volume of the two dams is less than that reported by the licensee. The staff used a bounding calculation to estimate the incremental increase in water-surface elevation resulting from upstream dam failures from a method described by the Washington State Department of Ecology (WDOE, 2007). The staff selected a flow velocity equal to 1.4 ft/s (0.4 m/s), which is associated with rough, mildly sloped channels and conservative because slower velocities generate higher water-surface elevations.

The staff also examined the topography near the confluence of St. Leonard Creek and the Patuxent River using the U.S. Geological Survey National Map Viewer (USGS, 2014) and estimated that the width of the Patuxent River at this location is 2 mi (3 km). The staff estimated the peak discharge from failures of the Rocky Gorge and Brighton dams using the Froehlich equation (Wahl, 1998; Froehlich, 1995), and the maximum dam storage volumes and dam heights for the two dams as reported in the National Inventory of Dams database in the Froehlich equation. The estimated incremental water-surface elevation rises resulting from the postulated Brighton and Rocky Gorge dam failures are of about 11.8 ft (3.60 m) and 22.5 ft (6.86 m) respectively. The staff added these two incremental water-surface elevation rises, assuming that the peak discharges produced by the two dam failures would occur simultaneously and translate downstream unattenuated and result in higher water-surface elevations in the Patuxent River. At the confluence of St. Leonard Creek and the Patuxent River, the staff estimated the total incremental water-surface elevation increase is 34.3 ft (10.5 m).

The staff added 4.4 ft (1.3 m), the highest observed tide water-surface elevation at the Solomons Island NOAA station (ID# 8577330), to this water-surface elevation increase for an estimated dam failure peak water-surface elevation of 38.7 ft (11.8 m) at the confluence of St. Leonard Creek and the Patuxent River. The staff noted that this estimate of peak water-surface elevation is substantially below the elevation of 45.5 ft (13.9 m) of MD 2/4, the downstream boundary condition of PMF analysis. Therefore, the staff confirms that an upstream dam failure event would not cause a higher water-surface elevation at the downstream boundary of the PMF analysis. The staff confirms that the PMF analysis would result in higher water-surface elevations near the Calvert Cliffs site than those expected to result from an upstream dam failure. The conservatism used in the staff analysis exceeds that used by the licensee and therefore does not alter the licensee's conclusion.

The staff notes that there are no onsite water control or storage structures at Calvert Cliffs whose failure could result in flooding.

The staff confirms the licensee's conclusion that the reevaluated flood hazard for failure of dams and onsite water control/storage structures alone does not inundate the plant site.

3.5 Storm Surge

The FHRR reported that the reevaluated probable maximum flood elevation, including associated effects, for storm surge is 31.3 ft (9.54 m). This flood-causing mechanism is described in the licensee's current design basis.

In the following discussion, probable maximum storm surge (PMSS) is the water elevation representing the sum of the antecedent water level – the water level before the storm surge - and the additional water elevation caused by storm surge. The reevaluated probable maximum flood elevation, including associated effects, is the sum of PMSS and wave runup. In the discussion it is referred to as PMSS + wave runup.

3.5.1 Summary of Previous Evaluations

The Calvert Cliff. Units 1 and 2 flood walkdown report (CENG, 2012b, Attachment 4, Response 3.a) identified multiple values of the probable maximum hurricane-induced storm surge: "Section 2.8.3.6 of the UFSAR [CENG, 2012a] states that the calculated maximum wave runup[4] is to elevation 27.1 feet [8.3 m] MSL. Section 2.8.3.5 states that the maximum wave runup at the intake structure is 27.5 feet [8.4 m] MSL." The flood walkdown report (CENG, 2012b) indicates that elevation 27.5 ft (8.38 m) MSL[5] was used as the design-basis flood level for performance of the walkdowns. To clarify this contradiction, staff issued this request for additional information (NRC, 2014a):

> RAI 6: Storm Surge Flooding: The licensee's walkdown report submitted as part of Enclosure 4 of the March 12, 2012, 50.54(f) letter states the design basis flooding elevation is 27.5 ft MSL (although the walkdown report also notes that both 27.1 ft MSL and 27.5 ft MSL appears in the UFSAR). The FHRR states the design basis is 27.1 ft NGVD 29, however FHRR Table 2.4.3 states the PMSS + Wave Runup is 28.14 ft NGVD. Please describe the apparent contradiction of the site's design basis storm surge height.

The licensee responded to the RAI by letter dated February 10, 2014 (CENG, 2014b). The licensee stated that FHRR Table 2.4-3 (relevant portions shown in Table 3.5-1 in this Staff Assessment) identifies a value of 28.14 ft (8.58 m) as the PMSS + wave runup, which is a revised calculated value from the UFSAR (CEGG, 2007). It is the sum of (1) the PMSS level of 16.24 ft (4.950 m) (which includes the antecedent water level of 2.7 ft (0.82 m) MLW and the additional effect of storm surge), and (2) wave runup height of 11.9 ft (3.63 m).

The licensee stated that the original supporting vendor's Storm Surge Report used values directly taken from UFSAR Sections 3.8.3.5 and 3.8.5.6. The final submitted vendor report edited some of the values to show the makeup of the PMSS + wave runup by explicitly showing that it included (1) the antecedent water level and (2) the newly calculated wave runup of 11.9 ft (3.63 m).

Together these yield the 28.14 ft (8.58 m) value. The licensee stated in its RAI response that the purpose of the edited values was to provide a better comparison to data from the Calvert Cliffs Unit 3 COLA (Unistar, 2012). The licensee stated that the water-surface elevation for wave runup including storm surge used for comparison is 27.1 ft (8.26 m), which is consistent with the UFSAR value.

The FHRR stated that the storm surge level in the Calvert Cliffs, Units 1 and 2 UFSAR, Revision 39 (CEGG, 2007) was determined by using a computer program developed by the Jacksonville District Corps of Engineers (USACE, 1959). The PMH parameters were based on the Technical

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[4] Here "maximum wave run-up" is the maximum elevation reached by waves under storm-surge conditions. It is the same as the probable maximum flood elevation, including associated effects, for storm surge.

[5] Different terminology and vertical datum are used in the walkdown report (CENG, 2012b) and the hazard reevaluation report (CENG, 2013a).

Memorandum No. 120 (USACE, 1960) and HUR 7-97 (Weather Bureau, 1968). The licensee developed the PMH track in the UFSAR for Calvert Cliffs, Units 1 and 2 assuming a central pressure of 912 mb, peripheral pressure of 1047 mb, radius of maximum wind of 30 nmi (56 km) and forward speed of 23 mi/h (37 km/h). The licensee's surge level reported in the Calvert Cliffs, Units 1 and 2 UFSAR is 16.24 ft (4.95 m), with a total peak wave runup elevation of 28.14 ft (8.58 m). In the walkdown report (CENG, 2012b)[4], the licensee used 27.5 ft (8.38 m) as the design value. As stated, both 27.5 ft (8.38 m) and 27.1 ft (8.26 m) are contained in the UFSAR, Revision 39 (CEGG, 2007). Section 2.8.3.6 of the UFSAR explains that the physical model calculation of 27.1 ft (8.26 m) was performed as a method to validate the calculated value of 27.5 ft (8.38 m), taking into account the physical attributes of the intake structure. This section was added to the UFSAR in response to questions from the NRC in 1971, related to the calculated value of 27.5 ft (8.38 m). Therefore, the licensee concluded the use of 27.5 ft (8.38 m) is conservative for penetrations in the intake structure as it relates to available physical margin determination.

3.5.2 FHRR Probable Maximum Storm Surge Evaluations

The licensee performed the FHRR storm surge reevaluation for Calvert Cliffs, Units 1 and 2 using the hierarchical hazard assessment (HHA) approach in accordance with NUREG/CR-7046 (NRC, 2011e). The HHA is a progressively refined, stepwise estimation of site-specific hazards that evaluates the flooding hazard with the most conservative input parameters, methodology, and assumptions.

The licensee used the updated PMH parameters from National Weather Service (NWS) 23 (NOAA, 1979). For storm surge modeling, the licensee used the SLOSH program (NOAA, 1992; NOAA, 2009).

The licensee determined PMSS + wave runup for Calvert Cliffs, Units 1 and 2 reevaluation in two phases. The licensee's Phase I analysis is based on the results of the PMSS analysis provided in Section 2.4.5 of the FSAR for Calvert Cliffs, Unit 3 (UniStar, 2012). Specifically, the licensee applied the surge level and wind speed from the Calvert Cliffs, Unit 3 FSAR analyses to compute the wind-wave runup at the Calvert Cliffs, Units 1 and 2 Intake Structure. The licensee indicated that the results of the Phase I evaluation yielded higher water levels than both the design-basis water levels and the elevation of safety-related structures at Calvert Cliffs, Units 1 and 2. Therefore, the licensee 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 licensee compared the resultant Phase II peak PMSS + wave runup against the critical elevation of safety-related structures, the Calvert Cliffs, Units 1 and 2 Intake Structure roof at elevation 28.5 ft (8.69 m), and the design basis flood level to determine if the PMSS + wave runup could affect the site.

The FHRR stated that the application of the results of the storm surge analyses performed for Calvert Cliffs, Unit 3 is valid for Calvert Cliffs, Units 1 and 2 because the sites are adjacent. Specifically, the licensee noted that the distance separating the sites has no effect on calculated wind speed or water-level values in the Chesapeake Bay. The grid cell which contains Calvert Cliffs, Unit 3 in the SLOSH computer software used for the surge analysis also contains Calvert Cliffs, Units 1 and 2. However, while the sources of PMH parameters are the same as Calvert Cliffs, Units 3 (NWS 23), the exact values chosen and track direction are somewhat different in order to create the SLOSH Maximum of MEOWs (MOM)[6], which is the most critical combination of PMH parameters and storm track, at the Cavert Cliffs, Units 1 and 2 site. Also, the FHRR indicated that the calculation of the wave runup changes between Calvert Cliffs, Unit 3 and Calvert Cliffs, Units 1 and 2 because of differing site geometry.

The FHRR reported an initial water level which included the 10 percent exceedance high spring tide of 1.53 ft (0.466 m) MSL, initial rise of 1.1 ft (0.34 m), long-term sea level rise of 1.07 ft (0.326 m), and mean sea level to NGVD29 conversion height of 0.64 ft (0.20 m). This provided the FHR's initial stillwater level of 4.4 ft (1.34 m), which was used as the antecedent water level (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 Clavert Cliffs, Unit 3 PMSS analysis as stated in the FHRR.

The FHRR stated that in the Phase I evaluation, the wave runup was calculated for the Calvert Cliffs site using the equivalent-slope method described in the USACE Coastal Engineering Manual (CEM) (USACE, 2008a). 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. The licensee noted that this approach is consistent with the HHA approach outlined in NUREG/CR-7046 (NRC, 2011e).

The FHRR stated that the wave runup was calculated for the Phase II evaluation using the equivalent-slope method described in the USACE CEM (USACE, 2008a), which was also used for the Phase I wave runup analysis. The calculation used the deep-water wave parameters and surge water level calculated for Calvert Cliffs, Unit 3 as inputs into this analysis as stated in the FHRR. Specifically, a significant wave height of 10.9 ft (3.32 m) and a wave period of 5.6 seconds was used for the wave runup analysis. The geometry used by in the runup analysis was taken from as-built drawings. Thus, the methodology and set of inputs used for the Phase II evaluation is identical to those used for the Phase I evaluation, with the exception of the surge level which varied based on the iteration of the HHA (e.g., no storm decay or storm decays with higher latitudes) for the recomputed storm surge analysis.

3.5.3 FHRR Storm Surge Results

Table 3.5-1 summarizes and compares the results of the reevaluated storm surge at Calvert Cliffs, Units 1 and 2 to the design-basis storm surge and wave runup parameters and results listed in Section 2.8.3 of the Calvert Cliffs, Units 1 and 2 UFSAR, Revision 39 (CEGG, 2007). Table 3.5-2 summarizes the resulting flood levels from all iterations of the HHA for the storm surge flood hazard, including wave runup effects. The reevaluated probable maximum flood elevation, including associated effects, for storm surge is 31.3 ft (9.54 m).

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[6] MEOW is the "maximum envelope of water," which represents the highest water due to a family of parallel hurricane tracks with the same direction, speed and intensity. MOM represents the "maximum of MEOWS," or the overall worst-case high-water scenario.

The staff reviewed the licensee's description of the historical observations of hurricane storm surge in the Chesapeake Bay caused by hurricanes. The staff independently reviewed historical information about storm surges resulting from both hurricanes and northeasters (Siebers, 2010). The staff found that hurricanes were the cause of the two highest surges reported at Sewells Point, Virginia (in the lower portion of the Chesapeake Bay).

The staff confirms the licensee's conclusion that both hurricanes and norrtheasters could produce storm surges at the Calvert Cliffs site and that the historical data presented by Siebers (2010) suggest that maximum surges produced by hurricanes are likely to exceed those produced by northeasters. The staff further concludes that consideration of PMH is the more conservative phenomenon to consider in a determination of the PMSS + wave runup near the Calvert Cliffs site. Based on this review, the staff confirms the licensee's conclusion that a hurricane moving north along the west edge of the Chesapeake Bay would produce the largest positive surge near the Calvert Cliffs, Unit 1 and 2 site because of the combination of the primary surge moving up the Chesapeake Bay and the wind setup caused by the hurricane's counter-clockwise cross winds. The staff independently determined from NWS 23 (NOAA, 1979) that the direction of approach for a PMH near the Calvert Cliffs, Unit 1 and 2 site could range from 69° to 153° clockwise from north. The staff reviewed the licensee's chosen PMH track for Calvert Cliffs, Units 1 and 2 with respect to the range of direction of approach given in NWS 23 (NOAA, 1979) and confirms that the licensee's chosen PMH track conforms to existing guidance. The staff also confirms that the licensee's selected PMH parameter values follow NWS 23 (NOAA, 1979) guidance.

3.5.4 Staff Conclusion

The staff confirms the licensee's conclusion that the reevaluated hazard for flooding from storm surge is not bounded by the current design basis flood hazard; therefore, the licensee must include flooding from storm surge within the scope of the integrated assessment. Information on flooding from storm surge that is specific to the data needs of the integrated assessment is described in section 4 of this staff assessment.

3.6 Seiche

The FHRR reported that the reevaluated hazard, including associated effects, for site flooding from seiche does not inundate the plant site. This flood-causing mechanism is not described in the licensee's current design basis.

The FHRR stated that no significant oscillations within the Chesapeake Bay have been observed in the storm surge records. The FHRR stated that sustained wind speed along the north-south axis of the Chesapeake Bay may cause a seiche, but the period of these oscillations is reported to be 2 to 3 days. The licensee stated that any existing seiche oscillations in the Chesapeake Bay prior to the arrival of PMH would be eliminated by the strong and changing PMH wind field. Therefore, the licensee concluded that resonance (seiche oscillations) in conjunction with the PMSS would not occur.

The staff reviewed the licensee's conclusions that although a PMH that made landfall south of the mouth of the Chesapeake Bay and then moved north along the west edge of the Chesapeake Bay could result in transverse winds, the lateral winds reversing at the natural period of oscillation in the Chesapeake Bay would not occur during PMH conditions.

The staff confirms the licensee's conclusion that the reevaluated hazard for flooding from seiche alone does not inundate the plant site.

3.7 Tsunami

The FHRR reported that the reevaluated PMF elevation, including associated effects, for site flooding due to tsunami is 11.5 ft (3.51 m). This flood-causing mechanism is not described in the licensee's current design-basis.

3.7.1 Tsunami Sources

The FHRR evaluation of tsunami flooding hazard was based on the evaluation conducted for the COLA for Calvert Cliffs, Unit 3 (Unistar, 2012). The evaluation was supplemented with information from recent literature and databases on tsunamis, and followed the guidance of NUREG/CR-6966 (NRC, 2009)

The licensee evaluated three different distant tsunami sources for establishing the probable maximum tsunami (PMT):

- Norfolk Canyon (or Currituck/Norfolk) submarine landslide
- La Palma (Canary Islands) volcanic flank failure
- Greater Antilles (Puerto Rico) submarine earthquake

The staff identified the same three primary tsunami source regions for determining the PMT at the Calvert Cliffs site. In addition, the staff identified and evaluated the Azores-Gibraltar oceanic convergence boundary as an earthquake source. The staff's independent confirmatory analysis confirms that this was not a significant potential tsunami source, so its effects were not evaluated in detail.

3.7.2 Historical Tsunami Record

The FHRR stated that a review of the same primary sources of information as were used for estimating PMT were used to establish the historical record of tsunamis affecting the U.S. Atlantic coast. Five potential distant tsunami source regions were identified: (1) submarine landslide along the U.S. Atlantic continental slope; (2) sources in the eastern Atlantic Ocean, including submarine earthquakes near Portugal and volcanos in the Canary Islands; (3) plate-boundary earthquakes in the Caribbean; (4) submarine earthquakes in the northern Atlantic Ocean offshore of Newfoundland, Canada; and (5) subduction zone earthquakes along the South Sandwich Islands in the southern Atlantic.

The FHRR described three historic tsunamis for which there are measured or computed tsunami amplitudes along the U.S. East Coast: (1) based on published numerical computations (Mader, 2001), the 1755 Lisbon seismogenic tsunami associated with the eastern Atlantic source region is estimated to have had a maximum amplitude of 3 m along the U.S. East Coast, (2) the 1918

Puerto Rico earthquake in the Caribbean source region resulted in a tsunami amplitude measurement of 0.2 ft (0.06 m) at Atlantic City, NJ, and the 1929 Grand Banks earthquake and associated landslide from the northern Atlantic source region resulted in a tsunami amplitude measurement of 2.2 ft (0.7 m) at Atlantic City, NJ.

3.7.3 Source Generator Characteristics

The FHRR stated that the analysis estimated maximum tsunami amplitude and dominant period offshore of the Chesapeake Bay entrance from published journal articles (Norfolk Canyon and Canary Islands sources) and NUREG/CR-1106 (Greater Antilles source) (NRC, 1979). These values were used as a boundary forcing function for the Chesapeake Bay propagation model described in the Calvert Cliffs, Unit 3 COLA FSAR (Unistar, 2012). Conditions at the Chesapeake Bay entrance were as follows:

For the Norfolk Canyon landslide scenario, the maximum amplitude is 13 ft (4.0 m) and the dominant period is 60 min.

For the La Palma (Canary Islands) volcanic flank failure, the maximum amplitude is 9.8 ft (3.0 m) and the dominant period is 60 min. For the Greater Antilles earthquake, the maximum amplitude is 2.95 ft (0.899 m) and the dominant period is 86.7 min.

3.7.4 Tsunami Analysis

The FHRR analysis computed tsunami propagation within the Chesapeake Bay and estimated tsunami water levels at the Calvert Cliffs site for each of the three tsunami source scenarios . Tsunami water levels at the Calvert Cliffs site were determined from tsunami propagation models of the Chesapeake Bay based on the shallow-water wave equation. Both the Non-Linear Shallow-Water Wave Equation (NLSWE) and the TSUNAMI (TSU) models were used.
The FHRR analysis derived the bathymetric grid from the NOAA digital elevation model, which was derived from a variety of data sources – primarily depth soundings between the years 1859 and 1993. The analysis used an adjusted MSL at the Chesapeake Bay Bridge Tunnel as the reference water level used for the simulations. The optimal grid size for the bathymetry and model calculations, in terms of model accuracy and computational time requirements, was determined by the licensee to be 1,181 ft by 1,181 ft (360 m by 360 m) based on sensitivity analysis. The analysis used a constant Manning's roughness coefficient of 0.025 for the bottom friction term and approximated frequency dispersion by using the numerical dispersion available from finite-differencing, and using a "hidden grid" technique to emulate physical dispersion. The FHRR stated that the numerical models were verified against analytical solutions from a Gaussian hump.

3.7.5 Tsunami Water Levels

The FHRR stated that four propagation simulations were performed to establish the PMT water levels relative to MSL at the Calvert Cliffs, Units 1 and 2 site:

- Norfolk Canyon landslide source scenario using the NLSWE model
- Norfolk Canyon landslide source scenario using the TSU model

- Canary Islands volcanic flank failure source scenario using the NLSWE model

Tsunami Amplitude, Initial Estimate The FHRR noted that the results from the NLSWE model indicate that the maximum amplitude water level is associated with the Norfolk Canyon landslide, whereas the maximum drawdown is associated with the Greater Antilles earthquake. Because the TSU model does not have energy dissipation from bottom friction and non-linear terms, the FHRR analysis used the water levels from this model with the Norfolk Canyon source to establish the PMT water levels at the site. The maximum tsunami amplitude at the Calvert Cliffs, Units 1 and 2 site reported in the FHRR is 1.07 ft (0.326 m) MSL.

Tsunami Amplitude from Sensitivity Simulations 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, the FHRR analysis cited the performance of a series of model sensitivity simulations wherein the minimum allowable water depth (cutoff depth) was varied in the model. The scenarios using a cutoff depth of 6.6 ft (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 (Norfolk Canyon) linear simulation (TSU) was increased by 60 percent, or 0.66 ft (0.20 m) to obtain the PMT water level at the Calvert Cliffs, Units 1 and 2 site. Therefore, the FHRR stated that the maximum tsunami amplitude at the Calvert Cliffs, Units 1 and 2 site is 1.71 ft (0.521 m).

PMT With Antecedent Water Level The FHRR reported that the PMT water level 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 4.36 ft (1.33 m), which accounts for the 10 percent exceedance high spring tide of 2.16 ft (0.658 m), a sea level anomaly of 1.12 ft (0.341 m), and long-term sea level rise of 1.08 ft (0.329 m). The PMT water level is therefore 6.07 ft (1.85 m).

PMT with All Effects The FHRR stated that Mader (2001) indicates that tsunami runup is about 2 to 3 times the deep-water tsunami amplitude. Madsen and Fuhrman (2007) 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 Calvert Cliffs, Units 1 and 2 site. Therefore, the FHRR analysis used a tsunami runup of 3 times the maximum tsunami amplitude in the Chesapeake Bay near the site to provide a conservative estimate of runup. The runup height therefore was estimated as 5.13 ft (1.56 m). The PMT high-water level, considering the maximum tsunami amplitude, antecedent conditions, and runup, was therefore estimated as 11.18 ft (3.41 m), rounded up to 11.5 ft (3.51 m).

3.7.6 Staff Analysis and Conclusion

The staff conducted an independent confirmatory analysis to determine the PMT at the Calvert Cliffs, Units 1 and 2 site that is described in detail in the sections that follow. The staff considered both far-field seismogenic (Puerto Rico subduction zone) and far-field (Canary Islands) and near-field (Currituck) landslide sources as potential generators for the PMT. Initial staff analysis indicated that the near-field submarine landslide is the likely source that determines the PMT maximum water level. The PMT minimum water level is determined by a far-field earthquake source along the Puerto Rico subduction zone.

The staff noted that the models the licensee used to determine tsunami wave height and periods at the site, the NLSWE and TSU models, do not include the effects of dispersion and turbulence. The water-level analysis portions of the staff's confirmatory analysis use COULWAVE to account for these effects. The staff performed numerical modeling simulations of three different tsunami sources to determine their impact on the Calvert Cliffs site. The three sources are a near field landslide source along the continental shelf break (the Currituck source), a far-field landslide source with extremely large local waves (the Canary Islands source), and a far-field earthquake source (the Puerto Rico Subduction Zone source). For all conditions, the most conservative parameters were employed to provide an absolute upper bounding limit on the possible tsunami effects at the Calvert Cliffs site.

The staff determined that the local submarine landslide source (Currituck landslide) has the largest impact at the Calvert Cliffs site, with the tsunami generating maximum water-surface elevations of 4.6 ft (1.4 m) and maximum fluid speeds of 4.3 ft/s (1.3 m/s), and is therefore the PMT.

The staff conducted an independent analysis of the 10 percent exceedance high tide of NOAA National Ocean Service Center for Operational Oceanographic Product Services (NOS-CO-OPS) data at the Solomons Island, MD tide gauge station (from 1996 to 2009) (NOAA, n.d.-a). The 10 percent exceedance high tide was determined to be 2.23 ft (0.680 m) NAVD88 for these years. The long-term sea-level rise at the Solomons Island, MD station is 3.41±0.29 mm/yr according to NOAA NOS-CO-OPS data. Therefore, the estimated antecedent water level is 4.56 ft (1.39 m). Therefore, the staff's analysis determines a PMT high-water level of 9.15 ft (2.79 m), less than the 11.5 ft (3.51 m) estimated by the licensee. The staff notes that the difference in high-water level was attributable to the licensee including the maximum tsunami amplitude in the antecedent water level.

The licensee's analysis indicates that the maximum water level from tsunamis is less than the grade elevation for the plant intake structure. Therefore, the licensee concluded that there will be no tsunami waves affecting safety-related facilities on the nuclear powerblock.

The staff confirms the licensee's conclusion that the reevaluated hazard for flooding from tsunami alone does not inundate the plant site.

3.8 Ice-Induced Flooding

The FHRR reported that the reevaluated hazard, including associated effects, for ice-induced flooding does not inundate the plant site. This flood-causing mechanism is not described in the licensee's current design-basis.

The licensee evaluated the historical air temperatures at the Calvert Cliffs, Unit 1 and 2 site using data from the nearby Patuxent River Naval Air Station meteorological tower for the period from 1945 through 2006.

The FHRR stated that the following mechanisms for ice-induced flooding at a nuclear power plant site were evaluated:

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

The FHRR stated that historical data characterizing ice conditions at the Calvert Cliffs site have been collected and their effects evaluated. The FHRR estimated the maximum Accumulated Freezing Degree Days (AFDD) to be 265.3 °F-days (129.4 °C-days) occurring on February 9, 1977, and the corresponding ice thickness to be approximately 13 in (33 cm). The FHRR stated that if ice forms at maximum thickness on the surface of the Chesapeake Bay, it will not cause any adverse flood risk to the safety-related or important-to-safety facilities as they are located at a minimum elevation of 28.5 ft (8.69 m). In addition, the formation of frazil or anchor ice is considered highly unlikely based on the historical climate records. Furthermore, the FHRR stated that formation of frazil ice at the existing intake could be precluded because of the potential recirculation of the heated cooling-water discharge from Calvert Cliffs, Units 1 and 2 back to the makeup water intake structure (MWIS) forebay.

The staff examined daily air temperature records for 1945 to 2002 reported by the NOAA National Climatic Data Center for the Patuxent River Naval Air Station (WBANID 13721) near the Calvert Cliffs site (NOAA, n.d.-b). The staff estimated the maximum AFDD to be 136.7 °C-days (246 °F-days). The staff estimated the maximum possible thickness of an ice sheet in the Chesapeake Bay near the CCNPP site to be consistent with the ice thickness observed in 1977 that was noted in the FHRR. The staff confirms that its estimate of ice sheet thickness is conservative because (1) the calculation was based on the most conservative reasonable assumed conditions, and (2) a correction for later initiation of freezing in the brackish waters of the Chesapeake Bay was not made.

According to NOAA's Chesapeake Bay water temperature map (NOAA, n.d.-c), in the winter the temperature drops to 1 to 4°C (34 to 39°F). Also, there are no public records of frazil or anchor ice obstructing MWIS intakes in the Chesapeake Bay. Based on the historical climate records, the staff confirms that frazil ice or anchor ice is unlikely to occur and that if it did occur it would not affect the function of the MWIS intakes.

The staff confirmed the results of the FHRR reevaluation of the ice hazard by analysis of historical temperature data in the vicinity of Calvert Cliffs, Units 1 and 2, and a search of the USACE Ice Jam Database (USACE, n.d.-b).

The staff confirms the licensee's conclusion that the reevaluated hazard for ice-induced flooding does not inundate the plant site.

3.9 Channel Migrations or Diversions

The FHRR reported that the reevaluated hazard, including associated effects, for site flooding from channel migrations or diversions does not inundate the plant site. This flood-causing mechanism is not described in the licensee's current design-basis.

The FHRR stated that slow rise in sea level is the primary long-term process causing the shoreline to recede. The FHRR indicated that the waves and surges due to occasional hurricanes may considerably change coastal morphology by reaching the high upland banks out of the range of normal tides and waves.

The Maryland Department of Natural Resources (MDNR, n.d.-b) estimated rate of shoreline erosion in the area near Calvert Cliffs, Units 1 and 2 to be between 2 ft (0.6 m) and 4.0 ft (1.2 m) per year. Because the main plant area of Calvert Cliffs, Units 1 and 2 is located at a grade elevation of about 45 ft (14 m), and is set back approximately 300 ft (90 m) from the Chesapeake Bay shoreline, the FHRR stated that it is unlikely that the shoreline will retreat due to erosion to the site boundary.

Although slope failures appear to be caused by shoreline erosion undercutting a portion of the cliff, the FHRR concluded that any slope failure 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. Due to the approximately 300 ft (90 m) setback from the shoreline, the FHRR stated that it is unlikely that shoreline erosion south of the barge jetty would impact Calvert Cliffs, Units 1 and 2.

The FHRR stated that 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. Because there is no major stream outfall to the Chesapeake Bay near the site and most streams within the Calvert Cliffs property area drain toward Johns Creek, the FHRR concluded that it is unlikely that the shoreline would be affected by fluvial processes near the site.

The staff reviewed the topography and the absence of evidence of large-scale hill-slope failures within the watershed. The staff noted that the shoreline near the Calvert Clifss, Units 1 and 2 plant is protected against shoreline erosion. The staff also noted that the Chesapeake Bay is a broad estuary, and is not subject to meander erosion and deposition, or to the formation of channel cutoffs or diversions.

The staff confirms the licensee's conclusion that the reevaluated hazard for flooding from channel migrations or diversions is bounded by the current design-basis flood hazard.

4.0 INTEGRATED ASSESSMENT AND ASSOCIATED HAZARD DATA

The staff confirms that the reevaluated hazard results for all reevaluated hazard mechanisms are not bounded by the current design-basis flood hazard. Therefore, the staff concludes that an integrated assessment is necessary and must consider the following flood-causing mechanisms:

- Local Intense Precipitation (LIP)
- Probable Maximum Storm Surge (PMSS)

Section 5 of JLD-ISG-2012-05 describes the flood hazard parameters needed to complete the integrated assessment. The staff reviewed the following subset of these flood hazard parameters to conclude that the flood hazard information is appropriate input to the integrated assessment:

- Flood event duration, as shown in Table 4.0-1, including warning time and intermediate water-surface elevations that trigger actions by plant personnel, as defined in JLD-ISG-2012-05.
- Flood height and associated effects, as defined in JLD-ISG-2012-05 are shown in Table 4.0-2. Inputs to the integrated assessment for the associated effects are shown in Table 4.0-3.

The staff requested in RAI 7 (NRC, 2014a) that the licensee provide flood event duration parameters and the basis for these parameters:

- RAI 7: The March 12, 2012, 50.54(f) letter, Enclosure 2, requests the licensee to perform an integrated assessment of the plant's response to the reevaluated hazard if the reevaluated flood hazard is not bounded by the current design basis. The licensee is requested to provide the applicable flood event duration parameters (see definition and Figure 6 of the Guidance for Performing an Integrated Assessment, JLD-ISG-2012-05) associated with mechanisms that trigger an integrated assessment. This includes (as applicable) the warning time the site will have to prepare for the event, the period of time the site is inundated, and the period of time necessary for water to recede off the site for the mechanisms that are not bounded by the current design basis. The licensee is also requested to provide a basis for the flood event duration parameters. The basis for warning time may include information from relevant forecasting methods (e.g., products from local, regional, or national weather forecasting centers).

In a February 10, 2014, response (CENG, 2014b), the licensee summarized the flood duration parameters for LIP and PMSS, as shown in Table 4.0-1. The licensee provided discussions, diagrams, figures and tables for the PMP and PMSS events including flood duration, hydrodynamic loading, sediment deposition/erosion, debris, adverse weather, groundwater ingress and other pertinent factors. The staff notes that longer duration LIP events may result in higher flood elevations and longer periods of inundation than the six-hour event. The staff notes that PMP events having relatively short durations may result in limiting warning time and may result in consequential LIP flood elevation (e.g., flood elevations above the openings to plant structures). Therefore, the staff determines that, as part of the integrated assessment report, the licensee should consider a range of rainfall durations associated with the LIP hazard events (e.g., 1-, 6-, 12-, 24-, 48-, 72-hour PMPs) to determine the controlling scenario(s) for evaluation as part of the integrated assessment (see NRC, 2012c). This is **Integrated Assessment Open Item No. 1.**

Based upon the preceding analysis, staff confirms that the reevaluated flood hazard information defined in the sections above, with the exception of the identified open item, is appropriate input to the integrated assessment. As described in the 50.54(f) letter, the licensee must submit the integrated assessment no later than two years from the date of the FHRR. Subsequent to the issuance of the 50.54(f) letter the NRC issued a letter (NRC, 2014c) revising the requirement to submit an integrated assessment for FHRR's submitted before June 2013, which includes Calvert

Cliffs, Units 1 and 2. The revised requirement extended the request for an integrated assessment by 6 months. Thus, the licensee's integrated assessment submittal is due to the NRC by September 12, 2015. The staff notes that this action item, as well as the bases for flood duration parameters (e.g., warning time based on existing agreements), may be further evaluated as part of the integrated assessment.

5.0 CONCLUSION

The NRC staff has reviewed the information provided for the reevaluated flood-causing mechanisms of Calvert Cliffs, Unit 1 and 2. Based on its review, the staff concludes that the licensee conducted the hazard reevaluation using present-day methodologies and regulatory guidance used by the NRC staff in connection with ESP and COL reviews.

Based upon the preceding analysis, the NRC staff confirms that the licensee responded appropriately to Enclosure 2, Required Response 2, of the 50.54(f) letter, dated March 12, 2012. In reaching this determination, staff confirms the licensee's conclusions that (a) the reevaluated flood hazard results LIP and storm surge are not bounded by the current design-basis flood hazard, (b) an integrated assessment including LIP and storm surge is expected to be submitted by the licensee, and (c) the reevaluated flood-causing mechanism information is appropriate input to the integrated assessment as described in JLD-ISG-2012-05 (NRC, 2012c).

The NRC staff identified an Integrated Assessment Open Item related to rainfall durations associated with the LIP hazard event. The Integrated Assessment Open Item is summarized in Table 5.0-1. Therefore, the NRC is not providing finality on the flood parameters related to the LIP hazard events as part of this Staff Assessment.

REFERENCES

Notes: (1) ADAMS Accession Nos. refers to documents available through NRC's Agencywide Document Access and Management System (ADAMS). Publicly-available ADAMS documents may be accessed through http://www.nrc.gov/reading-rm/adams.html. (2) "n.d." indicates no date is available or relevant, for example for sources that are updated by parts; "n.d.-a", "n.d.-b" indicate multiple undated references from the same source.

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Table 2.2-1. Flood-Causing Mechanisms and Corresponding Guidance

Flood-Causing Mechanism	SRP Section(s) and JLD-ISG
Local Intense Precipitation and Associated Drainage	SRP 2.4.2 SRP 2.4.3
Streams and Rivers	SRP 2.4.2 SRP 2.4.3
Failure of Dams and Onsite Water Control/Storage Structures	SRP 2.4.4 JLD-ISG-2013-01
Storm Surge	SRP 2.4.5 JLD-ISG-2012-06
Seiche	SRP 2.4.5 JLD-ISG-2012-06
Tsunami	SRP 2.4.6 JLD-ISG-2012-06
Ice-Induced	SRP 2.4.7
Channel Migrations or Diversions	SRP 2.4.9

Notes:

SRP is the Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition (NRC, 2007).

JLD-ISG-2012-06 is the "Guidance for Performing a Tsunami, Surge, or Seiche Hazard Assessment" (NRC, 2012c).

JLD-ISFG-2013-01 is the "Guidance for Assessment of Flooding Hazards Due to Dam Failure" (NRC, 2013a).

Table 3.0-1. Summary of Controlling Flood-Causing Mechanisms

Reevaluated Flood-Causing Mechanisms and Associated Effects that May Exceed the Powerblock Elevation (45.0 ft (13.7 m) NGVD29**)**[1]	**Elevation NGVD29**
Local Intense Precipitation and Associated Drainage	45.1 ft to 47.0 ft (13.7 m to 14.3 m)

Notes:
[1]Flood Height and Associated Effects as defined in JLD-ISG-2012-05 (NRC, 2012c).

Table 3.1-1 Safety-Related and Important-to-Safety SSCs and Associated Critical Elevations

Safety-Related and Important-to-Safety SSCs	Surrounding Grade Elevations,[1] ft (m) NGVD29	Entrance (Roof) Elevation,[2] ft (m) NGVD29	UFSAR Section[3]
Containment Buildings	45.0 (13.7)	45.0 (13.7)[4]	1.2.2
Auxiliary Buildings	45.0 (13.7)	45.0 ft (13.7)	1.2.2
Emergency Diesel Generator Building	42.0 - 45.0 (12.8 -13.7)	45.5 ft (13.9)	1.2.2
Intake Structure[5]	10.0 (3.05)	Entrance: 10.0 ft (3.05) Roof: 28.5 ft (8.69)	1.2.2 2.8.3
Turbine Building[6]	45.0 (13.7)	45.0 ft (13.7)	1.2.2
Station Blackout Diesel Generator Building[7]	42.0 - 45.0 (12.8 - 13.7)	45.5 ft (13.9)	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 and 2 UFSAR (CENG, 2011) 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 NGVD29 elevation is watertight.

[6]Turbine Building is a Seismic Category II structure.

[7]Station Blackout Diesel Generator Building is an augmented quality structure.

Source: FHRR Table 1.1-1 (CENG, 2013a)

Table 3.1-2. Current Design-Basis Flood Levels

Flooding Mechanism	DB Stillwater Level, ft (m) NGVD29	DB Associated Effects	Current DB Flood Level, ft (m) NGVD29	Reference
Local Intense Precipitation and Associated Drainage	44.8 (13.6)	Not Discussed in CDB	44.8 (13.6)	SRP Section 2.1
Streams and Rivers	Not Discussed in CDB	Not Discussed in CDB	Not Discussed in CDB	SRP Section 2.2
Failure of Dams and Onsite Water Control/Storage Structures	Not Discussed in CDB	Not Discussed in CDB	Not Discussed in CDB	SRP Section 2.3
Storm Surge	17.6 (5.4)	9.5 (2.9) due to wave runup at intake	27.1 (8.3) at Intake[1]	SRP Section 2.4
Seiche	Not Discussed in CDB	Not Discussed in CDB	Not Discussed in CDB	SRP Section 2.5
Tsunami	Not Discussed in CDB	Not Discussed in CDB	Not Discussed in CDB	SRP Section 2.6
Ice-Induced	Not Discussed in CDB	Not Discussed in CDB	Not Discussed in CDB	SRP Section 2.7
Channel Migrations or Diversions	Not Discussed in CDB	Not Discussed in CDB	Not Discussed in CDB	SRP Section 2.8

[1]This value is based on a physical model (CEGG, 2007). In the walkdown report (CENG, 2012b), the licensee used the calculated value of 27.5 ft (8.38 m) as the design basis. See discussion in FHRR Section 3.5.1.

Table 3.2-1. Subbasin, Drainage Area, and Time of Concentration

Subbasin	Description	Drainage Area, mi^2 (km^2)	Time of Concentration, h
1	West edge of site	0.0256 (0.0663)	0.508
2	West of plant	0.0225 (0.0583)	0.325
3	Southwest of plant	0.0068 (0.0176)	0.187
4	West side of units	0.0087 (0.0225)	0.223
5	Southeast of units	0.0202 (0.0523)	0.349
6	Northwest of units	0.0219 (0.0567)	0.235

Source: FHRR Tables 2.1-2 and 2.1-3 (CENG, 2013a)

Table 3.2-2. Probable Maximum Precipitation Depths

Time (min)	PMP Depth, in (cm)
360	28.00 (71.12)
180	23.96 (60.86)
120	21.82 (55.42)
60	18.48 (46.94)
30	13.86 (35.20)
15	9.70 (24.64)
5	6.15 (15.62)

Source: FHRR Table 2.1-1 (CENG, 2013a)

Table 3.2-3. HEC-HMS Element Identification, Drainage Area, and HEC-HMS Computed Peak Discharges

HEC-HMS Element	Description	Drainage Area, mi^2 (km^2)	Peak Discharge, ft^3/s (m^3/s)
Sub-1	Subbasin west edge of site	0.0256 (0.0663)	544.5 (15.4)
Sub-2	Subbasin west of plant	0.0225 (0.0583)	598.2 (16.9)
Sub-3	Subbasin southwest of plant	0.0068 (0.0176)	236.0 (6.7)
Sub-4	Subbasin west side of units	0.0087 (0.0225)	278.7 (7.9)
Sub-5	Subbasin southeast of units	0.0202 (0.0523)	521.4 (14.8)
Sub-6	Subbasin northwest of units	0.0219 (0.0567)	686.2 (19.4)
R-1	Reach receives Sub-1 flow	0.0256 (0.0663)	544.5 (15.4)
R-2	Reach receives flow from J-1	0.0481 (0.1246)	1,094.3 (31.0)
R-3	Reach receives flow from J-3	0.0087 (0.0225)	928.0 (26.3)
J-1	Junction receives flow from R-1 and S-2	0.0481 (0.1246)	1,094.3 (31.0)
J-2	Junction receives flow from R-2	0.0549 (0.1422)	1,240.9 (35.1)
J-3	Junction receives flow from Diversion and Sub-4	0.0087 (0.0225)	928.0 (26.3)
Diversion	Junction receives flow from J-2 and provides flow to Outlet-1	0.0549 (0.1422)	567.4 (16.1)
Outlet-1	Outlet receives flow from Diversion and Sub-5 and discharges to Chesapeake Bay	0.0751 (0.1945)	1,088.8 (30.8)
Outlet-2	Outlet receives flow from R-3 and Sub-6 and discharges to Chesapeake Bay	0.0306 (0.0793)	1,602.0 (45.4)

Source: FHRR (CENG, 2013a,Tables 2.1-2 and 2.1-3)

Table 3.2-4. Comparison of Building Entrance and LIP Water-Surface Elevations, and Duration of Flooding

Safety-Related Facility	Opening/ Floor Elevation, ft (m)	Associated Reach and Cross Section	PMF Water-Surface Elevation, ft (m)	PMF Maximum Inundation Depths, ft (m)	Duration of Flooding, h
South Service Building	45.0 (13.7)	Downstream-1, 489	46.8 (14.3)	1.8 (0.5)	1.5
Turbine Building	45.0 (13.7)	Downstream-1, 382	45.9 (14.0)	0.9 (0.3)	0.8
Auxiliary Building	45.0 (13.7)	Downstream-2, 1722	47.0 (14.3)	2.0 (0.6)	1.5
Auxiliary Building	45.0 (13.7)	Downstream-2, 1509	46.9 (14.3)	1.9 (0.3)	1.5
Auxiliary Building	45.0 (13.7)	Downstream-2, 1412	46.9 (14.3)	1.9 (0.3)	1.3
Auxiliary Building	45.0 (13.7)	Downstream-2, 1336	46.9 (14.3)	1.9 (0.3)	1.0
Turbine Building	45.0 (13.7)	Downstream-2, 1075	45.1 (13.7)	0.1 (0.03)	0.3
Diesel Generator Buildings	45.5 (13.9)	Downstream-2, 1075	45.1 (13.7)	Not Flooded	

Source: FHRR (CENG, 2013a, Table 2.1-5)

Table 3.5-1. Comparison of Parameters and Results for Storm Surge and Wave Runup Analyses

Parameter or Result	UFSAR for CCNPP 1 & 2 (CENG, 2011)[2]	CCNPP 1 & 2 Values Used or Computed in the FHRR[1] (CENG, 2013a)
PMH Parameters and Results		
Central Pressure Deficit, mb	135	118 to 124
Radius of Maximum Winds, nmi (km)	29 (54)	32 to 46 (59 to 85)
Forward Speed, mi/h (km/h)	23 (37)	15.7 to 30.5 (25.3 to 49.1)
Maximum Wind Speed, mi/h, (km/h)	124.7 (200.7)	111 (179)
Wave Runup Parameters and Results		
Antecedent Water Level, ft (m) NGVD29	2.82 (0.860)	4.4 (1.3)
PMSS Level, ft (m) NGVD29	16.24 (4.950)	17.5 (5.33)
Wave Runup, ft (m)	11.9 (3.63)	13.8 (4.21)
PMSS + Wave Runup, ft (m) NGVD29	28.14 (8.577)	31.3 (9.54)

Notes:
[1]Phase II Iteration No. 4
[2] In the walkdown report (CENG, 2012b), the licensee used the UFSAR calculated value of 27.5 ft (8.38 m) NGVD29 as the design basis. The FHRR uses the physically modeled value of 27.1 ft (8.26 m) NGVD29 (FHRR Table 2.4-3). See discussion in FHRR Section 3.5.1.

Table 3.5-2. Summary of HHA Results

Phase	Wave Runup Iteration	Key Storm Surge Assumptions	PMSS Water Level, ft (m) NGVD29	Key Wave Runup Assumptions	Calculated Equivalent Slope	Wave Runup Height, ft (m)	PMSS+ Wave Runup Water Level, ft (m) NGVD29
I	1	Use CCNPP 3 results	18.1 (5.52)	Does not take into account effect of Intake Structure lower deck	29.2°	32.7 (9.97)	50.8 (15.5)
	2	Use CCNPP 3 results	18.1 (5.52)	Reduced breaking wave height and equivalent slope due to effect of Intake Structure lower deck	18.0°	15.5 (4.72)	33.6 (10.2)
II	1	No decay of storm; 20 percent SLOSH accuracy applied directly to SLOSH results	19.9 (6.07)	Does not take into account effect of Intake Structure lower deck	29.2°	32.7 (9.97)	52.6 (16.0)
	2	Storm decays with higher latitudes; 20 percent SLOSH accuracy applied directly to SLOSH results	18.4 (5.61)	Does not take into account effect of Intake Structure lower deck	29.2°	32.7 (9.97)	51.1 (15.6)
	3	Storm decays with higher latitudes; 20 percent SLOSH accuracy applied directly to SLOSH results	18.4 (5.61)	Reduced breaking wave height and equivalent slope due to effect of Intake Structure lower deck	18.5°	16.2 (4.94)	34.6 (10.5)
	4	Storm decays with higher latitudes; 20 percent SLOSH accuracy applied only to storm surge rise	17.5 (5.33)	Reduced breaking wave height and equivalent slope due to effect of Intake Structure lower deck	16.7°	13.8 (4.21)	31.3 (9.54)

Table 4.0-1. Flood Event Duration for Flood-Causing Mechanisms to be Examined in the Integrated Assessment[1]

Flood-Causing Mechanism	Site Preparation for Flood Event	Period of Site Inundation	Recession of Water from Site
Local Intense Precipitation and Associated Drainage	More than 24 h	1.5 h	2 to 3 h
Storm Surge	48 h	No specific duration for the PMSS event was defined in the RAI 7 response (CENG, 2014a). This duration will be reviewed as part of the integrated assessment.	

[1] The bases for flood duration parameters (e.g., warning time based on existing agreements) may be further evaluated as part of the integrated assessment.

Table 4.0-2. Reevaluated Flood Hazards for Flood-Causing Mechanisms to be Examined in the Integrated Assessment[2]

Flood-Causing Mechanism	Stillwater Elevation m (ft) NGVD29	Associated Effects ft (m)	Reevaluated Flood Hazard ft (m) NGVD29	FHRR Section[1]
Local Intense Precipitation and Associated Drainage	45.1 to 47.0 (13.7 to 14.3)	NA	45.1 to 47.0 (13.7 to 14.3)	2.1
Storm Surge	17.5 (5.33)	13.8 (4.2) due to wave runup	31.3 (9.54)	2.4

[1]Flood Hazard Remediation Report (CENG, 2013a)
[2] The bases for flood elevation parameters may be further evaluated as part of the integrated assessment.

Table 4.0-3. Integrated Assessment Associated Effects Inputs

	Flooding Mechanism	
Associated Effects Factor	**Local Intense Precipitation**	**Storm Surge**
Hydrodynamic loading at plant grade	Determined by licensee to be minimal based on site conditions.	Determined by licensee to be minimal based on site conditions. See Figure 4 in RAI 7 response, Attachment 1 (CENG, 2014a) and UFSAR (CENG, 2011), Section 2.8.3.6, Structural Analysis of the Intake Structure and Conclusions.
Debris loading at plant grade	Determined by licensee to be minimal based on site conditions.	No specific debris load identified by licensee. Existing UFSAR states intake structure can withstand impact of baffle wall plate without damage to the intake structure (FHRR Section 3).
Sediment loading at plant grade	Determined by licensee to be minimal based on site conditions.	Determined by licensee to be minimal based on site conditions.
Sediment deposition and erosion	Sediment, erosion and scour determined by licensee to be minimal based on site conditions and impermeable surfaces.	Scour is not expected by licensee as stated in UFSAR (CENG, 2011), Section 2.8.3.6.
Concurrent conditions, including adverse weather	No specific concurrent condition evaluated by licensee. Interim actions to be performed prior to the expected PMP event.	No specific concurrent condition evaluated by licensee. Interim actions[1] to be performed prior to expected PMSS storm.

	Flooding Mechanism	
Associated Effects Factor	**Local Intense Precipitation**	**Storm Surge**
Groundwater ingress	Interim actions to Auxiliary Building to preclude ingress. Intake Structure and 1A Diesel Generator not susceptible. Turbine Building evaluated for Ingress during PMP with all ingress paths open and it was determined by licensee that no safety significant SSCs would be affected.	None determined by licensee.
Other pertinent factors (e.g., waterborne projectiles)	None noted by licensee.	Intake Structure roof ventilation louvers - Wind and hydrodynamic loading and wind driven missiles. No specific debris loading identified by licensee.

[1] The licensee stated in FHRR Section 4.3 that interim actions and procedures exist and that these interim actions and procedures will be reevaluated and updated as determined by the integrated assessment.

Table 5.0-1: Integrated Assessment Open Item

Integrated Assessment Open Item: The Integrated Assessment Open Item set forth in the Staff Assessment and summarized in the table below identifies certain matters that will be addressed in the integrated assessment submitted by the Licensee. This item constitutes information requirements but does not form the only acceptable set of information. A licensee may depart from or omit this item, provided that the departure or omission is identified and justified in the integrated assessment. In addition, this item does not relieve a licensee from any requested information described in Part 2, Integrated Assessment, of the March 12, 2012, 10 CFR 50.54(f) letter, Enclosure 2.

Open Item No.	SA Section No.	Subject to be Addressed
1	3.2	The licensee is requested to consider a range of rainfall durations associated with the LIP hazard events (e.g., 1-, 6-, 12-, 24-, 48-, 72-hour PMPs) to determine the controlling scenario(s) for evaluation as part of the integrated assessment. The evaluation should include a sensitivity analysis to identify potentially limiting scenarios with respect to plant response when considering flood height, relevant associated effects, and flood event duration parameters for LIP events.

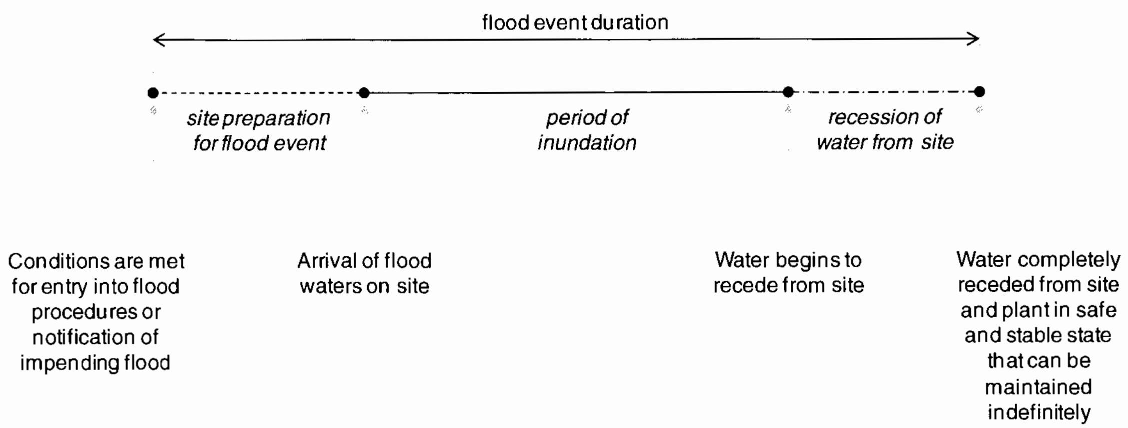


Figure 2.2-1. Flood event Duration

Figure 3.1-1 Site Area Topography, Drainage, Drainage Divide, River and Streams, Site Boundary and MD 2/4 Roadway (Source: CENG, 2013a, Figure 1.1-2)

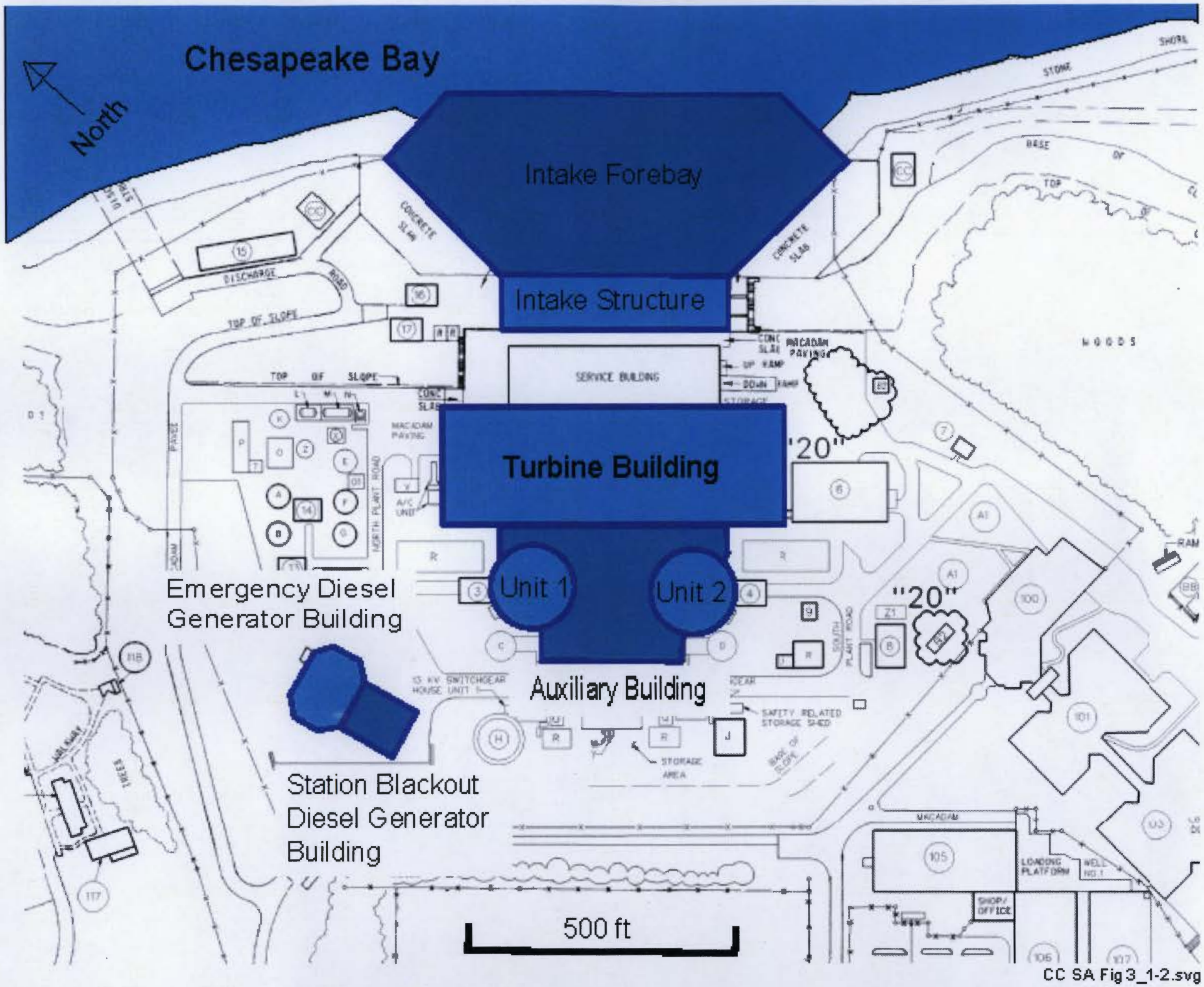
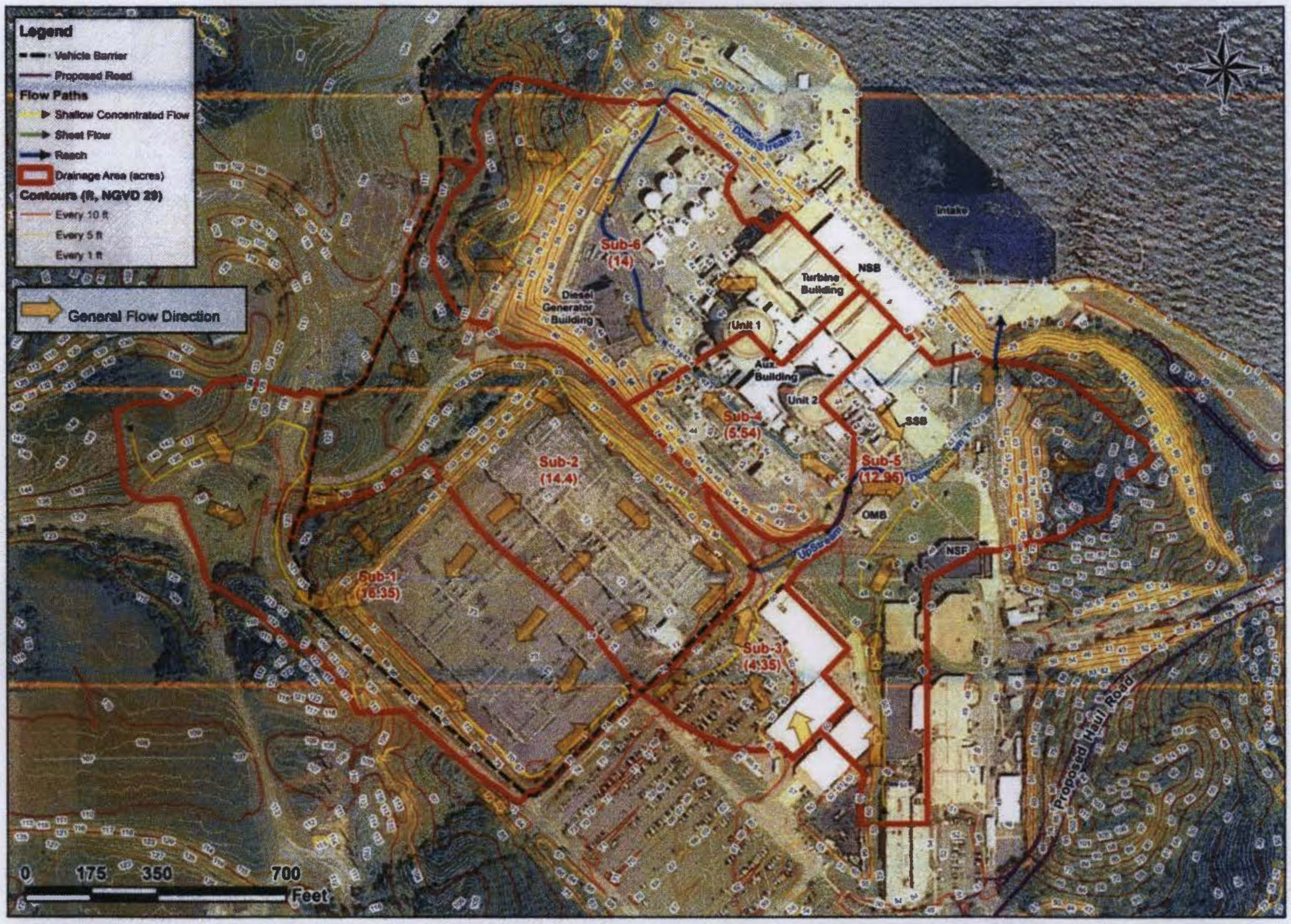


Figure 3.1-2 CCNPP Units 1 and 2 Plant Property and Buildings (Modified from CENG, 2013a, Figure 1.1-3)



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

Figure 3.2-1 Subbasin Drainage Area Map
(Source: CENG, 2013a, Figure 2.1-3)

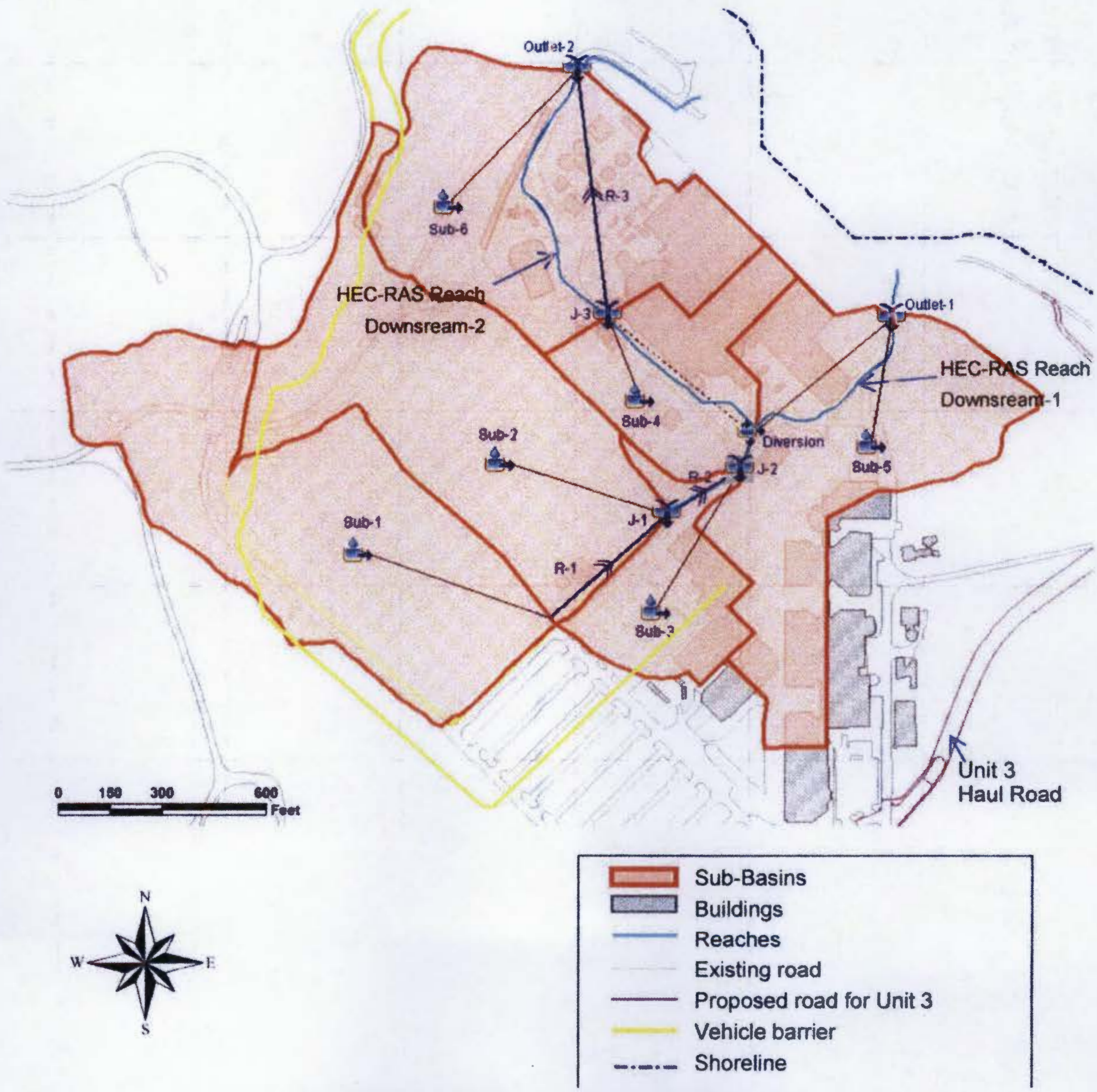


Figure 3.2.-2 Schematic of HEC-HMS Model
(Source: CENG, 2013a, Figure 2.1-5)

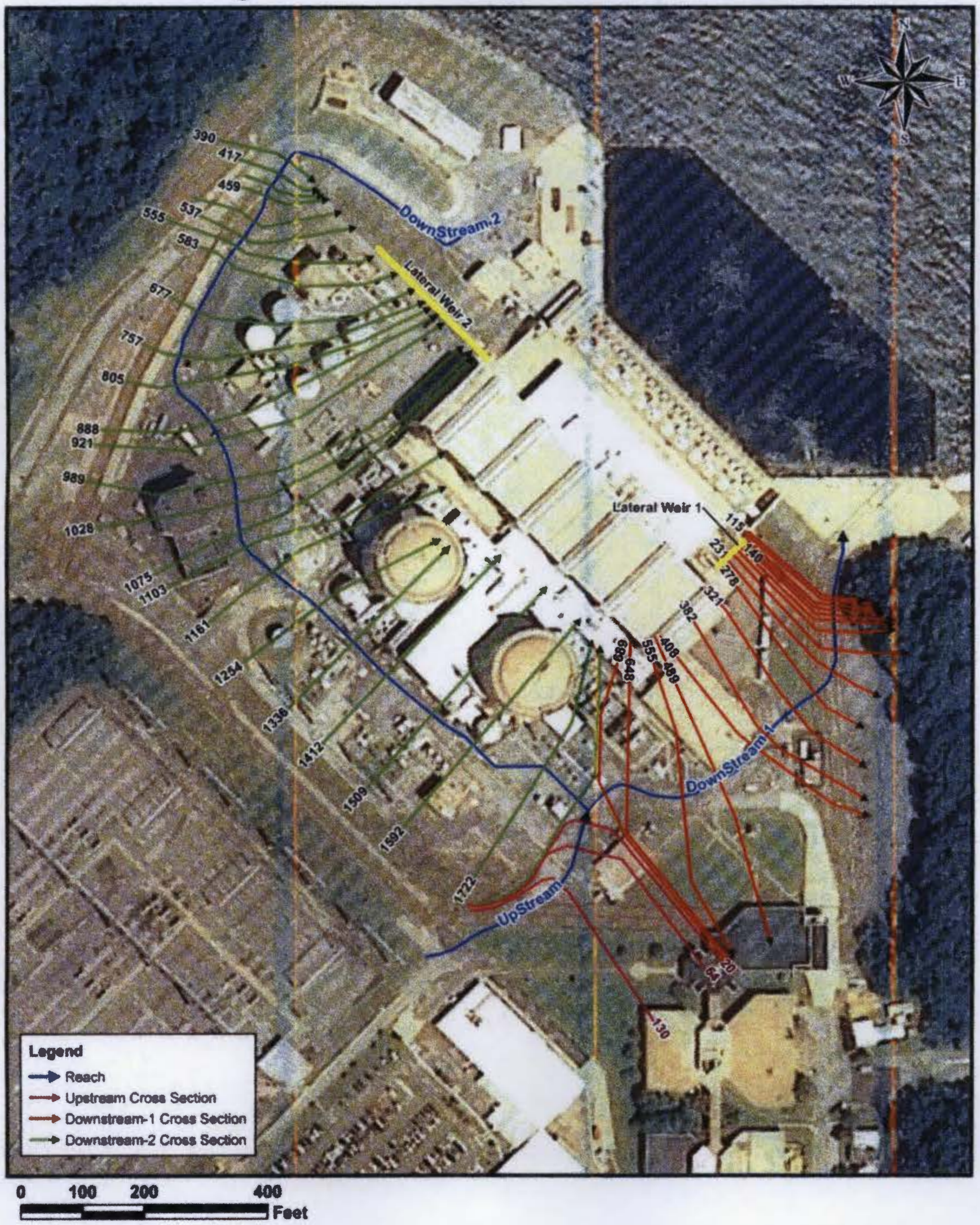


Figure 3.2-3 HEC-RAS Model Cross Section Plan
(Source: CENG, 2013a, Figure 2.1-7a)

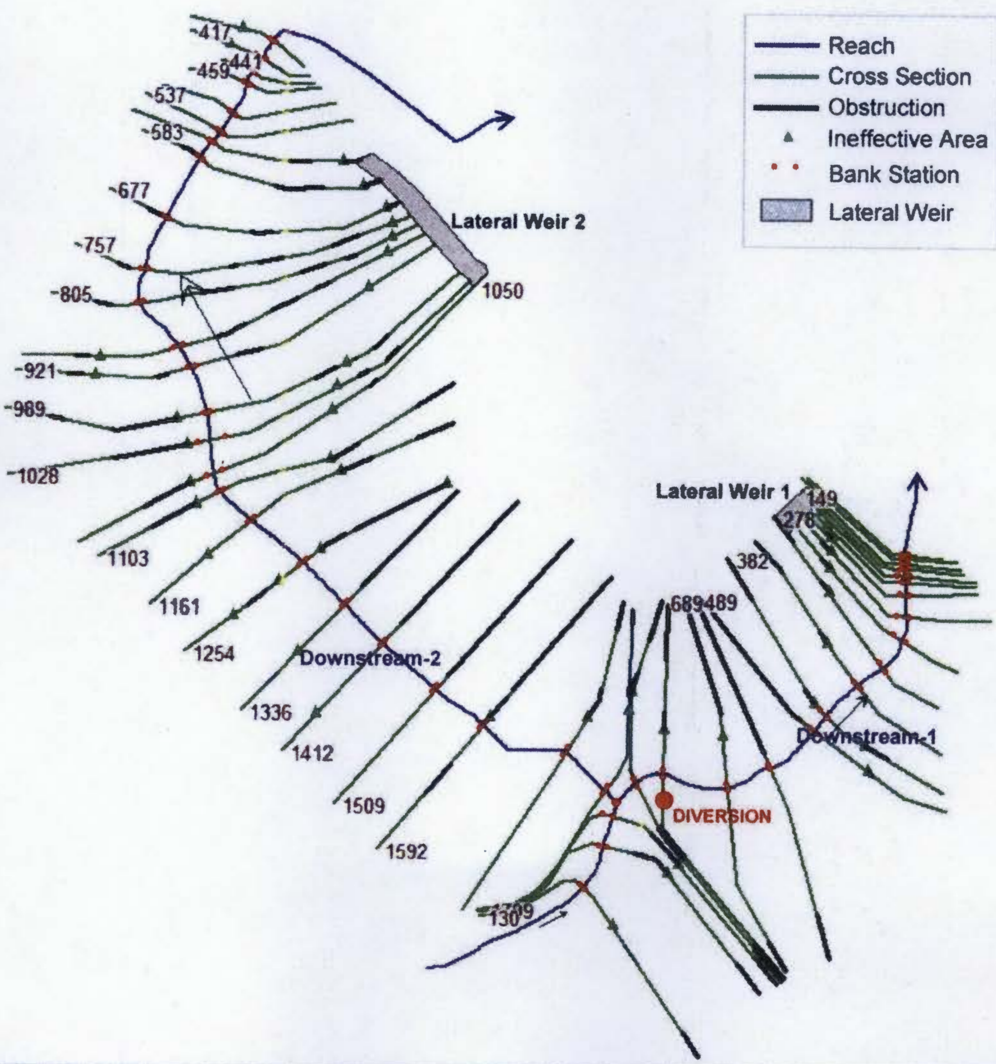


Figure 3.2-4 Schematic of HEC-RAS Model (Source: CENG, 2013a, Figure 2.1-8)

If you have any questions, please contact me at (301) 415-3733 or email at Robert.Kuntz@nrc.gov.

Sincerely,

/RA/

Robert F. Kuntz, Senior Project Manager
Hazards Management Branch
Japan Lessons-Learned Division
Office of Nuclear Reactor Regulation

Docket Nos. 50-317 and 50-318

Enclosure:
Staff Assessment of Flood Hazard
Reevaluation Report

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