2 SITE CHARACTERISTICS

2.4 Hydrologic Engineering

To ensure that a nuclear power plant or plants can be designed, constructed, and safely operated on the Combined License (COL) applicant's site and in accordance with U.S. Nuclear Regulatory Commission (NRC) regulations, the staff evaluated the hydrologic characteristics of the site and surrounding vicinity that may affect the safety of the proposed nuclear power plant at the site. These site characteristics describe the potential for flooding due to precipitation, riverine processes (runoff, dam breach discharge, channel blockage or diversion), coastal effects (storm surges and tsunamis), and combined events (e.g., from coincident wind waves). In addition, the staff reviewed the maximum elevation of surface water during floods and combined events, associated static and dynamic characteristics, minimum water-surface elevation during low-water events, the maximum elevation of groundwater, and the characteristic ability of the site to attenuate a postulated accidental release of radiological material into surface water and groundwater. The surface-water hydrologic site characteristics determine the design-basis flood for the proposed nuclear power plant (Calvert Cliffs Nuclear Power Plant (CCNPP Unit 3) and provide the basis for determining whether flood protection will be required. The groundwater hydrologic site characteristics determine the design-basis groundwater loadings and provide the basis for radiological dose analysis for a potential receptor from the postulated accidental release of radioactive liquid effluents in surface and ground waters.

The staff prepared Sections 2.4.1 through 2.4.14 of this report in accordance with the review procedures described in NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," Sections 2.4.1 through 2.4.14, using information presented in Calvert Cliffs COL Final Safety Analysis Report (FSAR), Revision 9, Section 2.4, "Hydrologic Engineering," (UniStar Nuclear Operating Services 2008, referred to as the COL FSAR in the rest of this report), which references U.S. EPR design certification FSAR, Revision 4, responses to staff requests for additional information (RAIs), and generally available reference materials (e.g., those cited in applicable sections of NUREG-0800).

The nominal proposed site grade for the CCNPP Unit 3 nuclear power block is 25.9 m (85.0 ft), National Geodetic Vertical Datum 1929 (NGVD29[1]). The elevations of the entrances to safety-related structures, systems, and components (SSCs) and openings within the protected area are located at or above 25.8 m (84.6 ft) (COL FSAR). CCNPP Unit 3, with its proposed U.S. EPR reactor design, relies on a water supply from the Chesapeake Bay, MD, for its safety-related ultimate heat sink (UHS). This external safety-related intake system (including intake pipes, stilling basin, pumps, and pipes leading up to the nuclear island) is not described or evaluated in U.S. EPR FSAR Revision 4 as described in COL FSAR Chapter 9. The COL applicant proposes to obtain UHS water through the makeup water intake structure (MWIS) located along the shoreline of the Chesapeake Bay. Therefore, the staff's review focused on the site characteristics of two locations: (1) The nuclear island and (2) the UHS

[1] Unless otherwise noted, NGVD29 is the reference for elevations throughout Section 2.4 of this report. The elevation of NGVD29 is 0.23 m (0.75 ft) lower than mean sea level at the Solomons Island National Oceanic and Atmospheric Administration (NOAA) station near the CCNPP Unit 3 site (NOAA 2003).

MWIS, including the safety-related pipe that will deliver water needed for the UHS from the MWIS to the nuclear island.
The site grade near the CCNPP Unit 3 is at elevation 25.8 m (84.6 ft). At this location, the following flooding hazard mechanisms, including associated effects, were computed and reported in the COL FSAR Revision 9.

CALCULATED FLOODING HAZARDS AND ASSOCIATED EFFECTS AS EVALUATED IN FSAR REV. 9	WATER-SURFACE ELEVATION	
	ft	m
Local Intense Precipitation and Associated Drainage	81.5	24.84
Flooding from Streams and Rivers	65	19.81
Failure of Dams and Onsite Water Control/Storage Structures*	+2	0.61
Flooding from Storm Surge With Wave Runup	33.2	10.12
Flooding from Seiche	NA	NA
Flooding from Tsunami	3.8	1.16
Ice-Induced Flooding	~0.2	0.05
Flooding from Channel Migrations or Diversions	NA	NA

*Rise in the elevation of St. Leonard Creek.

2.4.1 Hydrologic Description

2.4.1.1 *Introduction*

COL FSAR Section 2.4.1 describes the site and all safety-related elevations, structures, and systems from the standpoint of hydrologic considerations and provides a topographic map showing any proposed changes to grading and to natural drainage features.

Section 2.4.1 of this report provides a review of the following specific areas: (1) Interface of the plant with the hydrosphere including descriptions of the site location, major hydrologic features in the site vicinity, surface water- and groundwater-related characteristics, and the proposed water supply to the plant; (2) hydrologic causal mechanisms that may require special plant design bases or operating limitations with regard to floods and water-supply requirements; (3) current and likely future surface-water and groundwater uses by the plant and water users in the vicinity of the site that may affect the safety of the plant; (4) available spatial and temporal data relevant for the site review; (5) alternate conceptual models of the hydrology of the site that reasonably bound hydrologic conditions at the site; (6) potential effects of seismic and non-seismic data on the postulated design bases and how they relate to the hydrology in the vicinity of the site and the site region; and (7) additional information to satisfy requirements prescribed within the "Contents of Application" sections of the applicable subparts of Title 10 of the *Code of Federal Regulations* (10 CFR) Part 52.

2.4.1.2 *Summary of Application*

This section of the COL FSAR describes the site and all safety-related elevations, structures, and systems from the standpoint of hydrologic considerations. A topographic map included in the COL FSAR shows the proposed changes to existing grade and to natural drainage features (see Figure 2.4.1-1 below). COL FSAR Section 2.4.1 incorporates by reference U.S. EPR FSAR Tier 2, Revision 2, Section 2.4.1. The COL applicant included information related to

significant tributaries, estuaries, and the Chesapeake Bay. In addition, the COL applicant described the location of the UHS and its relationship to the MWIS.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-1

> A COL applicant that references the U.S. EPR design certification will provide a site-specific description of the hydrologic characteristics of the plant site.

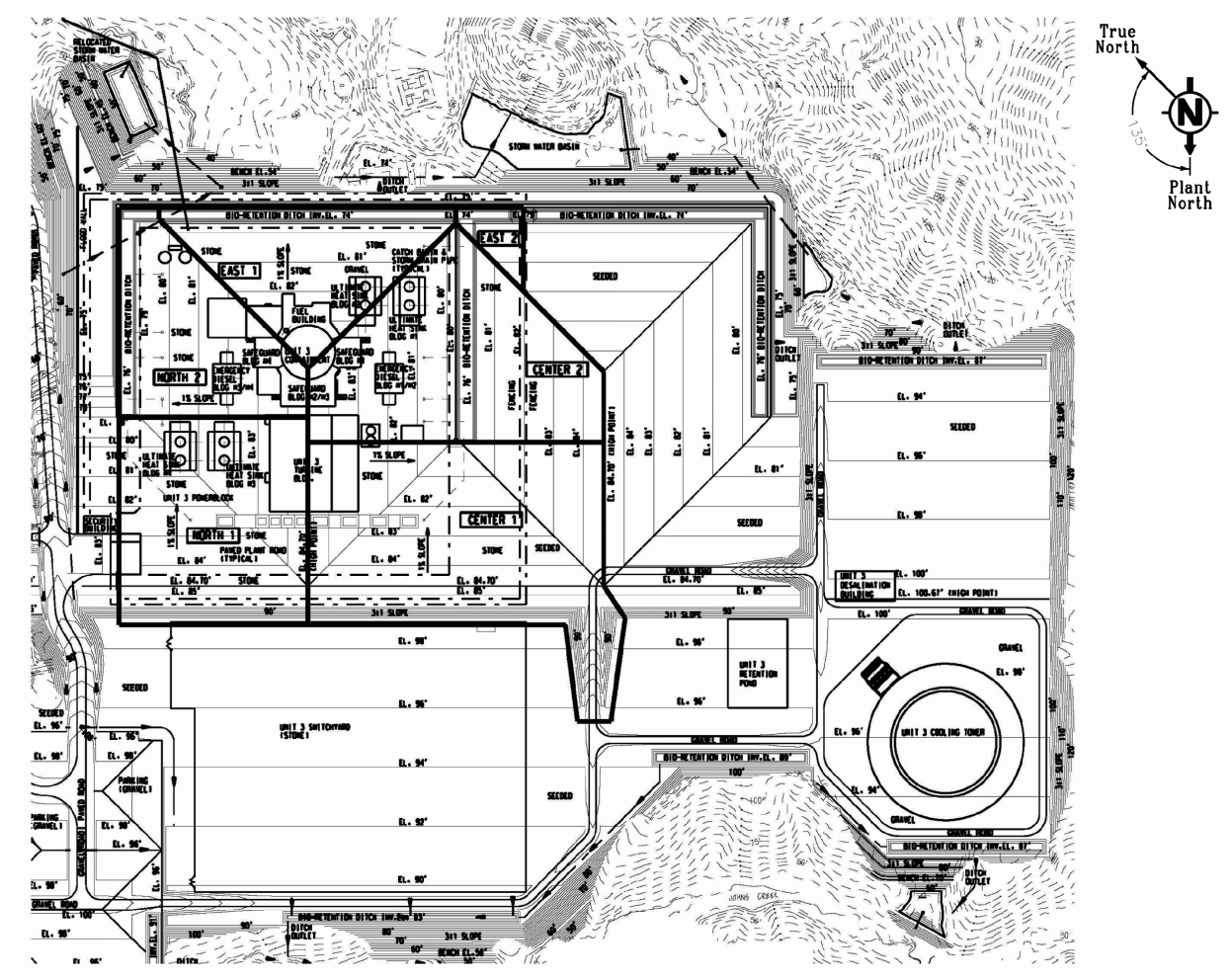

Figure 2.4.1-1. CCNPP Unit 3 Subbasin Drainage Boundaries with Proposed Changes in Topography

(Source: COL FSAR Figure 2.4-7)

2.4.1.3 *Regulatory Basis*

NRC regulations for the hydrologic description, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.1.

The applicable regulatory requirements for identifying the site location and describing the site hydrosphere are set forth in the following:

- 10 CFR 52.79(a)(1)(iii), "Contents of applications; technical information in final safety analysis report," as it relates to the hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated
- 10 CFR Part 100, "Reactor Site Criteria," as it relates to identifying and evaluating hydrologic features of the site
- 10 CFR 100.20(c), "Factors to be considered when evaluating sites," as it relates to requirements to consider physical site characteristics in site evaluations

The staff also used the following Regulatory Guides (RGs) for the acceptance criteria identified in NUREG-0800, Section 2.4.1:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding or those associated with the MWIS
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site

The regulatory basis for the information incorporated by reference will be addressed in the staff's Final Safety Evaluation Report (FSER) related to the U.S. EPR design certification application.

2.4.1.4 *Technical Evaluation*

The staff reviewed COL FSAR Section 2.4.1 and checked the referenced U.S. EPR design certification FSAR, Revision 4, to ensure that the combination of the information in the U.S. EPR design certification FSAR, Revision 4, and the COL FSAR represents the complete scope of required information related to this review topic. On the basis of its review, the staff confirmed that the information contained in the COL application, or information incorporated by reference, addresses the required information related to this section. U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.1 is being reviewed by the staff under Docket No. 52-020. The staff's technical evaluation of the information incorporated by reference will be documented in the staff's FSER for the U.S. EPR design certification application.

The staff reviewed the information in the COL FSAR including information related to the following information item:

COL Information Item 2.4-1

A COL applicant that references the U.S. EPR design certification will provide a site-specific description of the hydrologic characteristics of the plant site.

2.4.1.4.1 Site and Facilities

Information Supplied by the COL Applicant

The COL applicant described two locations where safety-related systems are located. The nuclear power block and the UHS MWIS. In addition, the COL applicant described safety-related water-supply pipes that would run from the UHS MWIS to the nuclear power block. The COL applicant described the use of a non-safety-related desalination plant that would draw water from the Chesapeake Bay to provide the makeup water for the essential service water system (ESWS) cooling towers.

The COL applicant stated that the CCNPP Unit 3 site containing the nuclear power block is located in the vicinity of the existing watershed divide. The COL applicant stated that after final site grading, a portion of the CCNPP Unit 3 site would drain abruptly toward the Chesapeake Bay (on one side of the watershed divide) and the other part would drain more gradually toward the Patuxent River via Johns Creek and St. Leonard Creek (on the other side of the watershed divide). The second location relevant to the staff's review of the hydrologic site characteristics is that of the UHS MWIS near the western shore of the Chesapeake Bay. Since the UHS MWIS is the source of safety-related water supply for CCNPP Unit 3, the COL applicant stated that both high-water and low-water-surface elevations are relevant for this location. The COL applicant stated that the invert water-surface elevation of the UHS sump is -6.9 m (-22.5 ft).

In addition, the COL applicant stated that four safety-related buried supply pipes would deliver water needed for the UHS from the UHS MWIS to the UHS cooling-tower basins located on the nuclear island, and four UHS makeup water pipes would deliver water from the UHS MWIS to the UHS cooling-tower basins on the nuclear island. The COL applicant stated that the supply pipes would be located in a utility corridor and would be buried below the final site grade at a depth that is generally sufficient to protect from frost formation. The COL applicant stated that structural fill would be placed below the supply pipes as bedding material and soil surrounding the pipes would be compacted structural fill.

The Staff's Technical Evaluation

Based on a June 24-26, 2008, site audit, an independent review of published U.S. Geological Survey (USGS) topographic maps of this area (Figure 2.4.1-2 of this report), and the COL applicant's site drainage and grading map (Figure 2.4.1-3 of this report), the staff confirmed the hydrologic characterization of the site provided by the COL applicant in COL FSAR Section 2.4.1.1. The staff relied on USGS topographic maps to independently delineate the Johns Creek subwatersheds upstream from the culvert beneath the Maryland State Highway (hereafter referred to as MD 2–4) roadway. For analyses related to the COL FSAR, the COL applicant assumed that this culvert would completely fail, (i.e., become completely blocked). Therefore, the staff did not attempt to characterize any hydraulic discharge characteristics of the culvert but assumed that the MD 2–4 roadway would make a suitable broad-crested hydraulic weir as floodwaters flow over the roadway. This weir could influence flooding scenarios at the site. Consequently, the staff evaluated (1) the influence of floods on Johns Creek from probable maximum precipitation (PMP) events (see Section 2.4.3 of this report); and (2) floods from dam

failures on the Patuxent River (see Section 2.4.4 of this report). Due to the elevation of the nuclear power block itself above the Chesapeake Bay as well as the elevation of the terrain contiguous with the area occupied by the nuclear power block, the staff determined that the only plausible flood mechanisms for the nuclear power block area would be local intense precipitation exceeding the design conveyance capacity of the engineered onsite drainage system.

Figure 2.4.1-2. Site Area Topography and Drainage

Note that the CCNPP Unit 3 site boundary shown on this figure differs from those shown on other figures of Section 2.4 of this report. This difference exists in the source COL FSAR figures as well. In RAI 400, Question 02.04-1, the staff requested that the COL applicant clarify

the site boundary associated with this figure. RAI 400, Question 02.04-1**is being tracked as an open item.** (Source: COL FSAR Figure 2.4-1)

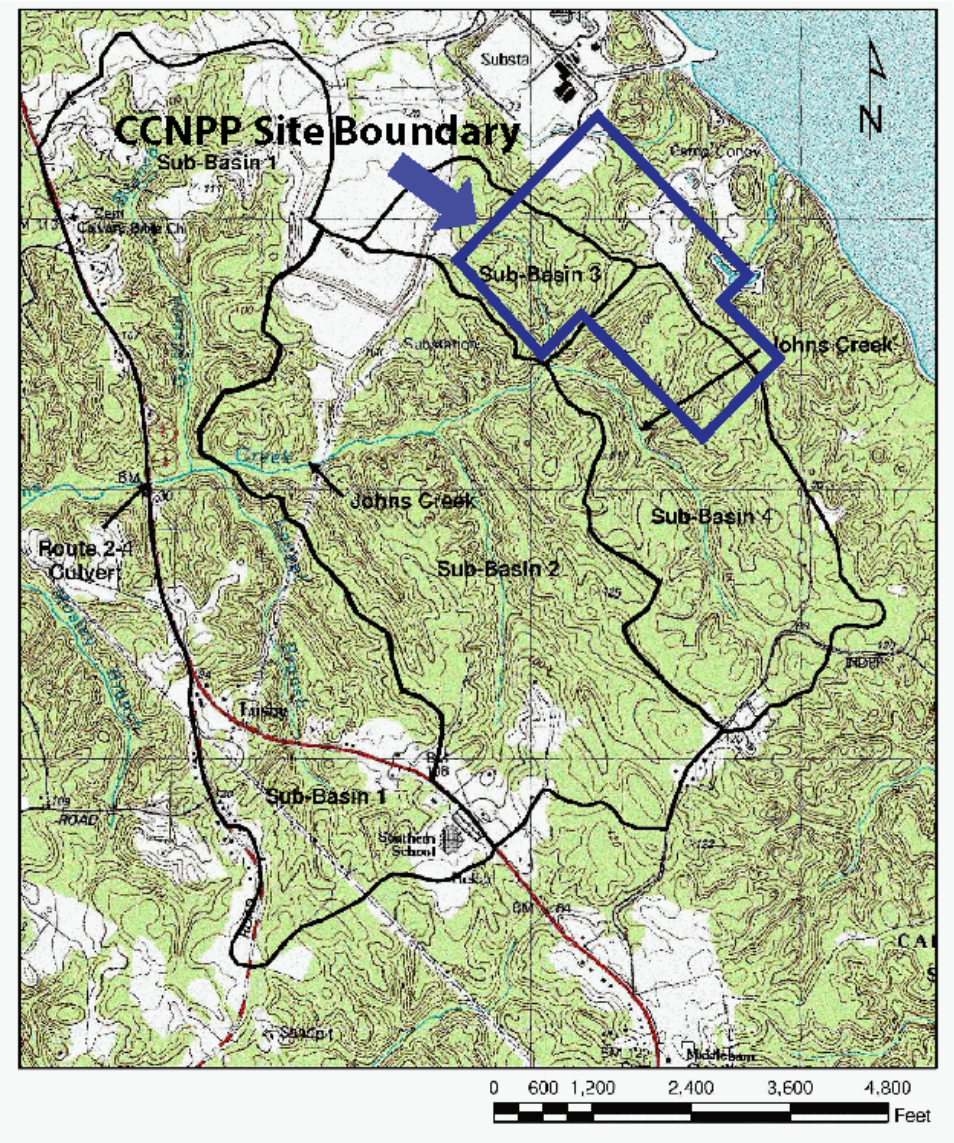

Figure 2.4.1-3. Johns Creek Watershed and Subbasin Delineation

Note that the staff added the approximate CCNPP site boundary to show the relationship between the subbasins and the site. (Source: COL FSAR Revision 9)

Based on a review of USGS topographic maps and observations made during the June 24-26, 2008, site audit, the staff determined that, because of the character of the surrounding terrain and the proximity of the site to the UHS MWIS, any design-basis flooding event related to the UHS MWIS would not be from extreme precipitation events, such as the PMP. Rather, a design-basis flooding event for the CCNPP Unit 3 site would be defined either by storm surge

(see Section 2.4.5 of this report) or by tsunami runup (see Section 2.4.6 of this report). Moreover, due to a very small catchment area situated upstream of the MWIS location, any runoff from a precipitation event would be small. Thus, any increase in water-surface elevation near the UHS MWIS associated with runoff from its catchment area would rapidly dissipate into the Chesapeake Bay.

Based on a review of the material presented by the COL applicant in COL FSAR Section 2.4.1 and the staff's observations made during the CCNPP Unit 3 site audit, and based on the reasons given above, the staff finds that the COL applicant adequately considered the hydrologic characteristics of the CCNPP Unit 3 site as required by 10 CFR 100.20(c), as it relates to requirements to consider physical site characteristics in site evaluations. Based on its review, the staff also finds that the COL applicant met the other requirements of 10 CFR Part 100, as they relate to identifying and evaluating hydrologic features of the site.

2.4.1.4.2 Hydrosphere

Information Supplied by the COL Applicant

In COL FSAR Section 2.4.1.2, the COL applicant provided information and summarized some observed data on the hydrologic characteristics of the Maryland Western Shore Watershed, Patuxent River Watershed, and Chesapeake Bay Estuary. In subsequent sections of the COL FSAR, the COL applicant provided information related to dams and reservoirs on the Patuxent River and its tributaries, surface-water users, and groundwater users.

The Staff's Technical Evaluation

The staff determined that there is no physical (hydrologic) nexus between surface-water use or groundwater use and the safety-related hydrologic site characteristics for this site. The Chesapeake Bay water supply (the principal source of surface water for CCNPP Unit 3) is not influenced by consumptive water use, because the bay is connected tidally to the Atlantic Ocean; it is also not influenced by tributary flows. The CCNPP Unit 3 design does not rely on groundwater as a source of safety-related water supply; therefore, the absence or loss of a groundwater supply has no relevance to this review.

Based on a review of the material presented by the COL applicant in COL FSAR Section 2.4.1 and the staff's observations of the CCNPP Unit 3 site during the June 24-26, 2008, site audit, and based on the reasons given above, the staff finds that the COL applicant has adequately considered the hydrosphere near the CCNPP Unit 3 site. Therefore, based on the reason given above, the staff finds that the COL applicant has met the requirements of 10 CFR 52.79(a)(1)(iii), as they relate to identifying the hydrologic characteristics of the proposed site with appropriate consideration of the most severe natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2.4.1.5 *Post Combined License Activities*

There are no post COL activities related to this section.

2.4.1.6 *Conclusions*

The staff reviewed the COL application and the referenced design certification FSAR Revision 4. As a result of the open item in RAI 400, Question 02.04-1, the staff was unable to finalize the conclusions, in accordance with NRC requirements. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to the characteristics of the site, incorporated by reference in the COL FSAR, will be documented in the staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will update Section 2.4.1 of this report to reflect the final disposition of the design certification application. **RAI 222, Question 01-5 is being tracked as an open item as part of this chapter.**

2.4.2 Floods

2.4.2.1 *Introduction*

COL FSAR Section 2.4.2 discusses the historical flooding at the proposed site or in the region contiguous with the site. The information identifies and summarizes the individual types of flood-producing phenomena, and combinations of flood-producing phenomena considered in establishing the flood design bases for safety-related plant features. The discussion also covers the potential effects of local intense precipitation.

2.4.2.2 *Summary of Application*

This section of the COL FSAR addresses site-specific flooding.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-2

> A COL applicant that references the U.S. EPR design certification will identify site-specific information related to flood history, flood design considerations, and effects of local intense precipitation.

In addition, this section addressed the following COL-specific information identified in U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.2:

The COL applicant provided information for flooding based on information from gage stations along the nearby St. Leonard Creek and the Patuxent River. The COL applicant also provided tide-elevation data recorded at Baltimore and Annapolis, MD. The COL applicant provided estimates for probable maximum flood (PMF), probable maximum hurricane (PMH), and probable maximum tsunami (PMT). The COL applicant also provided detailed information for the development of PMF. Later sections of the COL FSAR provide additional detailed discussion of the other extreme events, including probable maximum storm surge (PMSS) and PMT.

The COL applicant provided an estimate for the local PMP at the site and the local PMP's effects on the drainage systems throughout the site.

2.4.2.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the identification of floods and flood design considerations, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.2.

The applicable regulatory requirements for identifying probable maximum flooding on streams and rivers are set forth in the following:

- 10 CFR 52.79(a)(1)(iii), as it relates to the hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.
- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

The staff also used the appropriate sections of the following Regulatory Guides for the acceptance criteria identified in NUREG-0800, Section 2.4.2:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding or those associated with the MWIS
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site

The regulatory basis for the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.2.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.2. The staff confirmed that the information in the COL application addresses the required information related to site floods. The staff's technical review of this COL application included an independent review of the COL applicant's information in the COL FSAR and in the COL applicant's responses to staff RAIs. The staff supplemented this information with other publicly available sources of data.

COL FSAR Section 2.4.2 presents information about flood history, flood design considerations, and local intense precipitation. The staff's review of the information contained in the COL FSAR is discussed below.

2.4.2.4.1 Flood History

This section describes the historical floods at and in the vicinity of the proposed site.

Information Submitted by the COL Applicant

In COL FSAR Section 2.4.2.1, "Flood History," the COL applicant presented information about historical flooding based on records from the USGS streamflow stations at St. Leonard Creek and the Patuxent River. St. Leonard Creek is a tributary that connects Johns Creek to the Patuxent River. The COL applicant stated that flood records at Johns Creek and two small streams near the CCNPP Unit 3 site are not available. The COL applicant also stated that records for the two stations show no significant influence of tidal behavior on average stream flow. The maximum peak flow at St. Leonard Creek (USGS station ID# 01594800) is 8.16 cubic meters per second (m^3/s) (288 cubic feet per second (cfs)), which was recorded in 1960, as discussed in COL FSAR Section 2.4.1. Flood potentials for the Johns Creek and the two small streams are discussed in COL FSAR Sections 2.4.3, "Probable Maximum Flood (PMF) on Streams and Rivers," and 2.4.2.3, "Effects of Local Intense Precipitation."

COL FSAR Section 2.4.2.1 also presents tidal data recorded at Baltimore, MD from 1902 to the present and at Annapolis, MD from 1928 to the present. The COL applicant stated that the maximum recorded water level is 2.1 m (6.8 ft) above mean higher high water (MHHW) (2.4 m (8.0 ft)) due to storm surge. The COL applicant stated that since the construction of CCNPP Units 1 and 2, there has been no flooding that exceeds the grade elevation near the UHS pump intake area, which is located at an elevation of 3.5 m (11.69 ft).

The COL applicant stated that other flood-inducing phenomena such as ice jams and landslide-generated tsunamis are not important for determination of the design-basis flood. The tsunami-induced flooding event is discussed in COL FSAR Section 2.4.6, "Probable Maximum Tsunami Flooding."

The Staff's Technical Evaluation

The staff reviewed the flood history information provided by the COL applicant in COL FSAR Section 2.4.2.1. For the review, the staff examined steamflow records at two USGS gage stations located on St. Leonard Creek (USGS ID# 01594800) and the Patuxent River at Bowie (USGS ID# 01594440), MD. Flow recording at St. Leonard Creek has been discontinued (although records are available for two periods: 1957–1968 and 2000–2004). The USGS steamflow station closest to the site is located about 97 km (60 mi) upstream from the mouth of the Patuxent River at Bowie, MD. The maximum annual peak flood discharge recorded during the period from 1978 to 2009 was 360 m^3/s (12,700 cfs) in 2006, while the maximum annual peak stage was recorded as 9.85 m (32.3 ft) in 2006.

The staff examined water-elevation records observed at the Annapolis, MD tide station (National NOAA station ID# 8575512), and the Solomons Island tide station (NOAA station ID# 8577330) near the mouth of the Patuxent River. These stations are approximately 97 km (60 mi) and 12.3 km (7.7 mi), respectively, from the CCNPP Unit 3 site. Although the Annapolis NOAA station has a longer period of continuous monitoring records (hourly water levels are available from 1928 to the present) than the Solomons Island station, monitoring records from the Solomons Island NOAA station were used because of its closer proximity to the CCNPP Unit 3 site. However, the monitoring records for this site extend for a shorter period of time (verified

hourly water levels are available from 1979 to the present). The highest observed water-surface elevations at the Annapolis and Solomons Island NOAA stations are 1.9 m (6.3 ft) and 1.3 m (4.2 ft), respectively. The staff computed the cross-correlation coefficient between the daily maximum tidal elevations from hourly records for both stations for the period April 2012 through April 2013. Based on that calculation, the staff notes that the tidal records are highly correlated with a cross-correlation coefficient of 0.91. Therefore, the staff concluded that tidal elevation at the two stations is reasonably similar on daily time scales.

The staff reviewed the flood history information provided by the COL applicant in COL FSAR Section 2.4.2 and concluded that the information provided is sufficient to establish the history of flooding at and near the CCNPP Unit 3 site. Therefore, based on the reasons given above, the staff finds that the COL applicant has met the requirements of 10 CFR Part 100.20(c), as they relate to identifying and evaluating hydrological features of the site. Since the flood history provided by the COL applicant can be used as baseline information to compare with the estimated design bases for safety-related SSCs, the staff also finds that the requirements of 10 CFR 52.79(a)(1)(iii), as they relate to the determination of hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been recorded, are met.

2.4.2.4.2 Flood Design Considerations

This section describes the scenarios used to determine the design-basis flood at the CCNPP Unit 3 site.

Information Submitted by the COL Applicant

COL FSAR Section 2.4.2.2, “Flood Design Considerations,” discusses flood design requirements, including PMF, PMH, PMT, and local PMP. COL FSAR Sections 2.4.3 through 2.4.7 discuss the details concerning these extreme natural events. The COL applicant stated that the Seismic Category I SSCs are designed to withstand external flooding caused by natural phenomena. The COL applicant stated that the non-safety-related SSCs do not have protection requirements. The COL applicant stated that the design grade elevation for the CCNPP Unit 3 nuclear power block is 25.8 m (84.6 ft).

The COL applicant’s estimate of PMF stillwater-surface elevation in Johns Creek is 19.8 m (65.0 ft). This estimated water-surface elevation is lower than the lowest reported elevation of the existing (natural) drainage divide – given as 29.8 m (98.0 ft) – that would serve to block flood flow from Johns Creek to the CCNPP Unit 3 area.

The COL applicant stated that upstream dam failures could increase the water-surface elevation of the Patuxent River by about 0.6 m (2 ft), which would pose a potential flood hazard to the CCNPP Unit 3 site. The COL applicant’s analysis of potential dam failures is discussed in COL FSAR Section 2.4.4.

The COL applicant’s estimate of PMSS water-surface elevation in the Chesapeake Bay, including wind-wave runup associated with the storm is 10.1 m (33.2 ft) along the shore near the CCNPP Unit 3 site. This water-surface elevation would result in flooding of the UHS MWIS located at an elevation of 3.5 m (11.69 ft). Therefore, the UHS MWIS would require

flood-protection measures as discussed in COL FSAR Section 2.4.10, “Flooding Protection Requirements.”

The estimate of highest water-surface elevation caused by a PMT is 1.2 m (3.8 ft) as described in COL FSAR Section 2.4.6.

The estimate of the maximum flood water-surface elevation resulting from local intense precipitation is 24.8 m (81.5 ft), which is discussed in COL FSAR Section 2.4.2.3. This is the design-basis flood water-surface elevation for safety-related SSCs in the CCNPP Unit 3 nuclear power block area. The elevations of all safety-related building entrances in the nuclear power block area are above this design-basis flood water-surface elevation.

The Staff’s Technical Evaluation

The staff reviewed flood design considerations provided by the COL applicant in COL FSAR Section 2.4.2.2. Seismic Category I SSCs within the plant site are designed to withstand the effects of flooding caused by natural phenomena. The staff reviewed the COL applicant’s combinations of the most severe natural flood-producing phenomena that have been historically reported for the site and surrounding region, and determined that the COL applicant has appropriately considered flood-producing phenomena for the CCNPP Unit 3 site. The staff’s detailed reviews of PMF and PMH are discussed in Sections 2.4.3 and 2.4.5, respectively, of this report. The staff described its estimate of the PMF water-surface elevation, 21.4 m (70.3 ft), in Section 2.4.3.4.5 of this report. The COL applicant stated in COL FSAR, Revision 9, Section 2.4.1.1, “Site and Facilities,” that access to safety-related SSCs in the protected area boundary will be located at an elevation of 25.8 m (84.6 ft) or higher.

In RAI 328, Question 02.04.10-1, the staff requested that the COL applicant provide quantified information in the COL FSAR demonstrating that the UHS system and associated safety-related pipe system would be protected from adverse effects of natural phenomena, including flooding from local intense precipitation, storm surge, tsunami, ice formation, and groundwater. The staff reviewed the COL applicant’s December 20, 2012, response to RAI 328, Question 02.04.10-1.

In that response, the COL applicant described the application of the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) and Hydrologic Engineering Center River Analysis System (HEC-RAS) to the area drainage swale, utility corridor, and the haul road that runs along the northern edge of the CCNPP Unit 3 nuclear power block and toward the UHS MWIS before draining into Chesapeake Bay. The COL applicant simulated the runoff resulting from the local intense precipitation in this area to develop and evaluate the potential for erosion of the cover material over the pipes buried in this area. The COL applicant modeled the runoff resulting from the local intense precipitation in this area and estimated channel discharges by characterizing the haul road and drainage swales as a single channel having roughness coefficients consistent with concrete surfaces.

The staff reviewed the general site layout presented in the COL applicant’s December 20, 2012, response to RAI 328, Question 02.04.10-1, and concluded that this characterization is reasonable. The staff verified that the COL applicant appropriately selected the value for the Manning’s roughness coefficient. In the RAI response, the COL applicant described the use of site topography to configure the HEC-RAS model and the spacing of the model cross sections, and concluded that the incorporation of site information and model configuration is appropriate for the local intense precipitation analysis. The COL applicant estimated that the maximum

discharge velocities resulting from the local intense precipitation range from 1.7 to 7.4 m/s (5.5 to 24.2 ft/s). The COL applicant concluded that the drainage swales and the haul road need to be concrete-lined to mitigate against potential damage related to the local intense precipitation-generated flows.

The staff performed a simple independent evaluation of the expected peak water velocity at the cross section that the COL applicant determined to yield the largest peak velocity. The staff based its calculation on information presented in the COL applicant's December 20, 2012, response to RAI 328, Question 02.04.10-1. The staff performed the calculation for cross-section location 15, which overlies the UHS makeup water pipes. The staff used Manning's equation by characterizing the drainage swales and haul road as a 76.2-m (250-ft) wide trapezoidal channel; the staff estimated the channel width based on figures presented in the RAI response. The staff used a roughness value and bed slopes consistent with those described by the COL applicant. The peak local intense precipitation-generated discharge at this cross section, as estimated by the COL applicant, is 132.7 m^3/s (4,685 cfs). The staff adjusted the cross-sectional average depth of flow in its use of Manning's equation to match the COL applicant's value. The staff then computed the cross-sectional average water velocity to be 6.1 m/s (20.1 ft/s), which is lower than the value determined by the COL applicant, 7.4 m/s (24.2 ft/s). The staff concluded that the COL applicant's estimate is based on conservative assumptions and adequate incorporation of site-specific information.

The staff reviewed the velocities of the local intense precipitation-generated flood, evaluated the December 20, 2012, COL applicant's response, agreed with the COL applicant's conclusions, and closed the open item associated with RAI 328, Question 02.04.10-1. Based on the reasons given above, the staff finds that the COL applicant committed to the selection of a site-characteristic maximum permissible water velocity of 7.4 m/s (24.2 ft/s) and that this site-characteristic value would be used to protect buried UHS water pipes from erosion generated by local intense precipitation.

Based on a review of the COL applicant's information contained in the COL FSAR, the staff concluded that the COL applicant appropriately considered flood-causing phenomena and their combinations that are relevant for the CCNPP Unit 3 site. Based on the reasons given above, the staff finds that the requirements of 10 CFR Part 100.20(c), as they relate to identifying and evaluating hydrological features of the site, are met. The staff agreed that the combinations of flood-causing phenomena considered by the COL applicant are appropriate for the CCNPP Unit 3 site. Based on the reasons given above, the staff finds that the requirements of 10 CFR 52.79(a)(1)(iii) are met, as they relate to the determination of hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2.4.2.4.3 Effects of Local Intense Precipitation

This section describes the estimation of local intense precipitation and its effects on the safety-related SSCs of CCNPP Unit 3.

Information Submitted by the COL Applicant

COL FSAR Section 2.4.2.3 discusses the analysis of local intense precipitation at the site. The analysis includes runoff from the site grade and facility roofing and the local intense precipitation flood water-surface elevation associated with safety-related facilities. The lowest entrance elevation for safety-related facilities is 25.8 m (84.6 ft).

In COL FSAR Section 2.4.2.3, the COL applicant divided the local drainage area into six subbasins. Four ditches drain floodwater from the nuclear power block and Turbine Building areas, which are graded at a one percent slope. The East ditch collects flows provided by the North and Center ditches. The southern half of the East bioretention ditch is not considered in the hydraulic analysis, because it does not affect the local intense precipitation flood water-surface- elevation in the CCNPP Unit 3 area. During the PMP event, the overflow pipes and culverts are assumed not to function. The runoff collected in the North and East ditches would overflow the berm at elevation 24.1 m (79 ft), spilling out onto the bluff toward the Chesapeake Bay.

In the local intense precipitation analysis for the site, the COL applicant derived the 1-hour, 2.6-km^2 (1-mi^2) PMP from NOAA Hydrometeorological Report No. 52 (HMR-52), "Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian" (NOAA 1982) and converted the precipitation to runoff using HEC-HMS computer software (U.S. Army Corps of Engineers (USACE) 2006) with the input precipitation intensity based on the time of concentration. The COL applicant estimated the time of concentration values using the methodology described in U.S. Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS), Technical Release 55 (TR-55) Manual (USDA 1986). The COL applicant considered the nonlinear effects of extreme flood conditions during the local intense precipitation by reducing the estimated time of concentration as recommended by USACE EM-1110-2-1417 (USACE 1994). The COL applicant used NRCS unit hydrograph option in the runoff model. The COL applicant set the NRCS runoff curve number to 98, which implies a mostly impervious surface (USDA 1986).

The COL applicant conducted a steady-state analysis of the local intense precipitation-generated flood using the USACE HEC-RAS computer software (USACE 2008). The COL applicant used the lateral weir option in HEC-RAS computer software to represent overflows from the North and East retention ditches. The COL applicant selected a Manning's roughness coefficient value of 0.035 was selected, consistent with the materials comprising the channel bed and the overbank. The COL applicant assumed a water-surface elevation of 24.3 m (79.7 ft) at the downstream end of North and East ditches using a trial-and-error method. In estimating the overflow from the East and North ditches, the COL applicant assumed that the outer edge of the ditches act as weirs.

The COL applicant estimated the maximum water-surface elevation in the nuclear power block area to be 24.8 m (81.5 ft), which is 1.0 m (3.1 ft) below the design grade elevation for the CCNPP Unit 3 nuclear power block and 1.1 m (3.5 ft) below the nominal site grade. Therefore, the COL applicant concluded that the safety-related structures in the nuclear power block area for CCNPP Unit 3 would be safe from potential flooding during the local intense precipitation event, even if the entire drainage system does not function as intended.

The Staff's Technical Evaluation

The staff reviewed the COL applicant's analysis of the flood caused by local intense precipitation and its effect on safety-related facilities. The staff's review consisted of verifying the COL applicant's estimate of local intense precipitation, the runoff analysis, and the hydraulic backwater analysis for the onsite drainage channels.

The staff developed an independent HEC-RAS hydraulic model based on information obtained from the COL application. The staff confirmed that the overall flow paths and division of subbasins are appropriate based on the examination of the site layout plan provided by the COL applicant. The staff also confirmed the location and elevation of the cross sections used in the HEC-RAS analysis (see COL FSAR, Revision 9, Figure 2.4.1-3). The staff independently estimated the local intense precipitation for the CCNPP Unit 3 site as the 1-hour, 2.6 km^2 (1-mi^2) PMP using HMR-52. The staff independently confirmed that the COL applicant's estimate of 1-hour, 2.6 km^2 (1-mi^2) PMP depth of 46.9 cm (18.5 in.) is appropriate. The design certification FSAR states that the design parameters for maximum rain intensity are 49.3 cm/hr (19.4 in./hr) for the U.S. EPR reactor design. Therefore, the staff concluded that the 1-hour, 2.6 km^2 (1 mi^2) PMP values are within the design parameters approved for this particular reactor design. The staff estimated the peak runoff from each subbasin using the HEC-HMS model based on information provided by the COL applicant. The staff's independent estimates of travel time (channel length divided by average velocity) based on a HEC-RAS analysis indicates that the COL applicant's estimates are conservative. The staff conservatively assumed that all subbasins were impervious surfaces to maximize runoff from the PMP event. The staff's HEC-HMS model used the NRCS (previously Soil Conservation Service (SCS)) unit hydrograph method to transform PMP to runoff. The staff's independent analysis estimated peak runoffs that were reasonably consistent with those estimated by the COL applicant. Therefore, the staff concluded that the COL applicant's estimates of peak runoff are appropriate and conservative.

The staff's review then focused on the HEC-RAS analysis and whether the COL applicant used appropriate methodology and the correct input parameter values. The staff confirmed that the COL applicant distributed the peak discharge computed by HEC-HMS appropriately to each cross section of the HEC-RAS model. The staff estimated the water-surface elevation at the downstream boundaries assuming the site berms functioned as weirs. Using the independently configured model and information provided by the COL applicant, the staff estimated the maximum flood elevation of 24.8 m (81.5 ft), which does not exceed safety-related plant grade (25.8 m (84.6 ft)).

The staff conducted a sensitivity analysis using a higher Manning's roughness coefficient of 0.075 corresponding to a brush-covered floodplain for the nuclear power block area. The staff's estimate of maximum water-surface elevation is 25.1 m (82.4 ft) at the upstream of the Center ditch, which is 0.7 m (2.2 ft) lower than the safety-related plant grade. The staff notes that the combination of the higher value for Manning's roughness coefficient and the impervious surface area assumption in the estimation of peak runoff is conservative. Therefore, the staff concluded that the flood resulting from the local intense precipitation would not flood the safety-related facilities in the nuclear power block area.

In RAI 100, Question 02.04.02-1, the staff requested that the COL applicant clearly identify locations where supercritical flows are likely to occur and identify locations where hydraulic jumps are likely to form during the flooding event. This request included a description of fortification measures to ensure that hydraulic forces induced by the jumps do not erode or

degrade the ditch conveyance and a detailed description of the lateral-structure flow simulated in the numerical model. In a May 19, 2009, response to RAI 100, Question 02.04.02-1, the COL applicant revised the COL FSAR (Revision 9) to clarify the description of the lateral-structure flow simulated in HEC-RAS hydraulic model. The COL applicant stated that supercritical flows did not occur in any of the drainage ditches. The staff's independent analysis also confirmed that supercritical flow did not occur. Riprap channel linings are sized to withstand the estimated peak velocity of 1.7 m/s (5.5 ft/s). The staff also agreed that the fill slopes of the northern and eastern areas of the nuclear power block are protected with riprap, because the 1-hour, 2.6-km^2 (1-mi^2) PMP-generated flood peak flow velocity is 3.1 m/s (10 ft/s); the staff confirmed the PMP-generated flood peak flow velocity in its analysis. The staff finds the COL applicant's response acceptable. Accordingly, the staff considers RAI 100, Question 02.04.02-1 resolved.

In RAI 328, Question 02.04.10-1, the staff requested that the COL applicant provide quantified information in the COL FSAR to demonstrate that the UHS system and associated safety-related pipe system would be protected from the adverse effects of natural phenomena, including the following: Flooding from local intense precipitation; storm surge; tsunami; ice formation; and groundwater. The staff reviewed the COL applicant's December 20, 2012, response to RAI 328, Question 02.04.10-1. The staff documented its evaluation of the adverse effects of the natural phenomena and finds the COL applicant's response acceptable. Accordingly, the staff considers RAI 328, Question 02.04.10-1 resolved.

Based on a review of the COL applicant's information in COL FSAR Revision 9, the staff concluded that the COL applicant has appropriately considered flood-causing phenomena related to local intense precipitation for the CCNPP Unit 3 site. Therefore, based on the reasons given above, the staff finds that the COL applicant has met the requirements of 10 CFR 52.79(a)(1)(iii), as they relate to identifying and evaluating hydrological features of the site. The staff agreed that the flood-causing phenomena associated with local intense precipitation considered by the COL applicant are appropriate for the CCNPP Unit 3 site. Therefore, based on the reasons given above, the staff finds that the COL applicant has met the requirements of 10 CFR 52.79(a)(1)(iii), as they relate to the determination of hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2.4.2.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.2.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR Revision 4. On the basis of its review, the staff confirmed that the COL applicant addressed the required information related to the flood history, flood causal mechanisms, local intense precipitation, and the estimation of the local PMF. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to the flood history, flood causal mechanisms, local intense precipitation, and the estimation of the local PMF incorporated by reference in the COL FSAR will be documented in the staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the

U.S. EPR is not yet complete. The staff will update Section 2.4.2 of this report to reflect the final disposition of the design certification application.

2.4.3 Probable Maximum Flood on Streams and Rivers

2.4.3.1 *Introduction*

COL FSAR Section 2.4.3, "Probable Maximum Flood (PMF) on Streams and Rivers," describes the hydrologic site characteristics affecting any potential hazard to the plant's safety-related facilities as a result of the effects of PMF on streams and rivers.

Section 2.4.3 of this report provides a review of the following specific areas: (1) Design basis for flooding in streams and rivers; (2) design basis for site drainage; (3) consideration of other site-related evaluation criteria; and (4) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.3.2 *Summary of Application*

This section of the COL FSAR provides information about site-specific PMFs on streams and rivers. COL FSAR Section 2.4.3 incorporates by reference U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.3, "Probable Maximum Flood (PMF) on Streams and Rivers." In this section, the COL applicant provided site-specific supplemental information to address the COL-specific information identified in U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-3

> A COL applicant that references the U.S. EPR design certification will provide site-specific information to describe the probable maximum flood of streams and rivers and the effect of flooding on the design.

To address this information item, the COL applicant provided the PMP estimated in COL FSAR Section 2.4.3.1, "Probable Maximum Precipitation," following the procedures of HMR-51 (Schreiner and Riedel 1978) and HMR-52 (Hansen et al. 1982).

The COL applicant discussed precipitation losses from infiltration and provided a runoff analysis using the HEC-HMS hydrologic model. The analysis included transformation of PMP to runoff from subbasins and routing of runoff hydrographs across MD 2–4 as it passes over Johns Creek at the downstream boundary of the watershed. The COL applicant estimated the runoff using the NRCS unit hydrograph method.

To estimate the flood water-surface elevations in Johns Creek, the COL applicant provided steady-state backwater calculation results developed using the HEC-RAS hydraulic model for six different scenarios. The scenarios are based on discharges generated by the runoff model for each subbasin at the specified time relative to the time of the peak discharge. In COL FSAR Section 2.4.3.4.5, the COL applicant used the HEC-RAS model to simulate water-surface elevations based on hydraulic characteristics of the watershed geometry and boundary conditions.

2.4.3.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the identification of floods and flood design considerations, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.3.

The applicable regulatory requirements for identifying probable maximum flooding on streams and rivers are as follows:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.
- 10 CFR Part 100, as it relates to identifying and evaluating hydrologic features of the site. The requirements to consider physical site characteristics in site evaluations are specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used appropriate sections of the following Regulatory Guides for the acceptance criteria identified in NUREG-0800, Section 2.4.3:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding or those associated with the MWIS
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site

The regulatory basis of the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification.

2.4.3.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.3. The staff confirmed that the information in the COL application addresses the relevant information to PMF. The staff's technical review included an independent review of the information in the COL FSAR. The staff supplemented this information with other publicly available sources of data.

The staff reviewed COL Information Item 2.4-3 from U.S. EPR FSAR Tier 2, Revision 4, Table 1.8-2, included under COL FSAR Section 2.4.3.

COL Information Item 2.4-3

> A COL applicant that references the U.S. EPR design certification will provide site-specific information to describe the probable maximum flood of streams and rivers and the effect of flooding on the design.

As mentioned above, the COL applicant estimated PMP in COL FSAR Section 2.4.3.1, "Probable Maximum Precipitation," following the procedures described in HMR-51 (Schreiner and Riedel 1978) and HMR-52 (Hansen et al. 1982); discussed precipitation losses from infiltration and presented the runoff analysis using the HEC-HMS hydrologic model; and presented the steady-state backwater calculation using the HEC-RAS hydraulic model for six different scenarios. These analyses address the requirements of COL Information Item 2.4-3.

2.4.3.4.1 Probable Maximum Precipitation

Information Submitted by COL Applicant

The COL applicant estimated PMP in COL FSAR Section 2.4.3.1, "Probable Maximum Precipitation," following the procedures of HMR-51 (NOAA 1978) and HMR-52 (NOAA 1982). The COL applicant used the 1-hour, 2.6-km^2 (1-mi^2) PMP depth in the PMF analysis, because the Johns Creek drainage area (5.9 km^2 (2.3 mi^2)) is smaller than the next larger storm included in HMR-51, which is 26 km^2 (10 mi^2). The COL applicant stated that the centroid of Johns Creek watershed is close to the location of the CCNPP Unit 3 site. Therefore, the COL applicant's estimates are the same as those used for the local intense precipitation estimates in COL FSAR Section 2.4.2.3. Figure 2.4.3-1 shows water features near the CCNPP Unit 3 site

In the COL applicant's PMF analysis, the downstream boundary of Johns Creek watershed is at the MD 2–4 culvert crossing (see Figure 2.4.3-1). The COL applicant verified, through a parametric sensitivity analysis, that the tailwater at this downstream location does not affect the flood elevation at the CCNPP Unit 3 site. The COL applicant stated that the drainage area is divided into four subbasins with topography varying from about 3 m (10 ft) to 37 m (120 ft). The COL applicant stated that the lowest elevation point of the drainage divide between Johns Creek and CCNPP Unit 3 site is 29.9 m (98.0 ft). The subbasins of Johns Creek watershed are shown in Figure 2.4.1-3. Dense woody vegetation covers most of the subbasins.

The COL applicant specified the PMP temporal distribution using guidelines described by American National Standards Institute/American Nuclear Society (ANSI/ANS) Standard 2.8-1992 (ANS 1992). The duration of the PMP storm is 9 days: 3 days for an antecedent storm; 3 days for a dry period; and 3 days for the full PMP storm. The COL applicant used 40 percent of PMP as the antecedent storm based on ANSI/ANS-2.8-1992 guidelines. The COL applicant stated that it used the PMP depth data to develop the PMP storm rainfall depths. The COL applicant stated that based on the historical data the effect of snowmelt is not an important phenomenon for determination of the design-basis flood.

For the peak runoff analysis, the COL applicant used the HEC-HMS hydrologic model. The COL applicant used the frequency storm method following the procedure in the *HEC-HMS Technical Reference Manual* (USACE 2006).

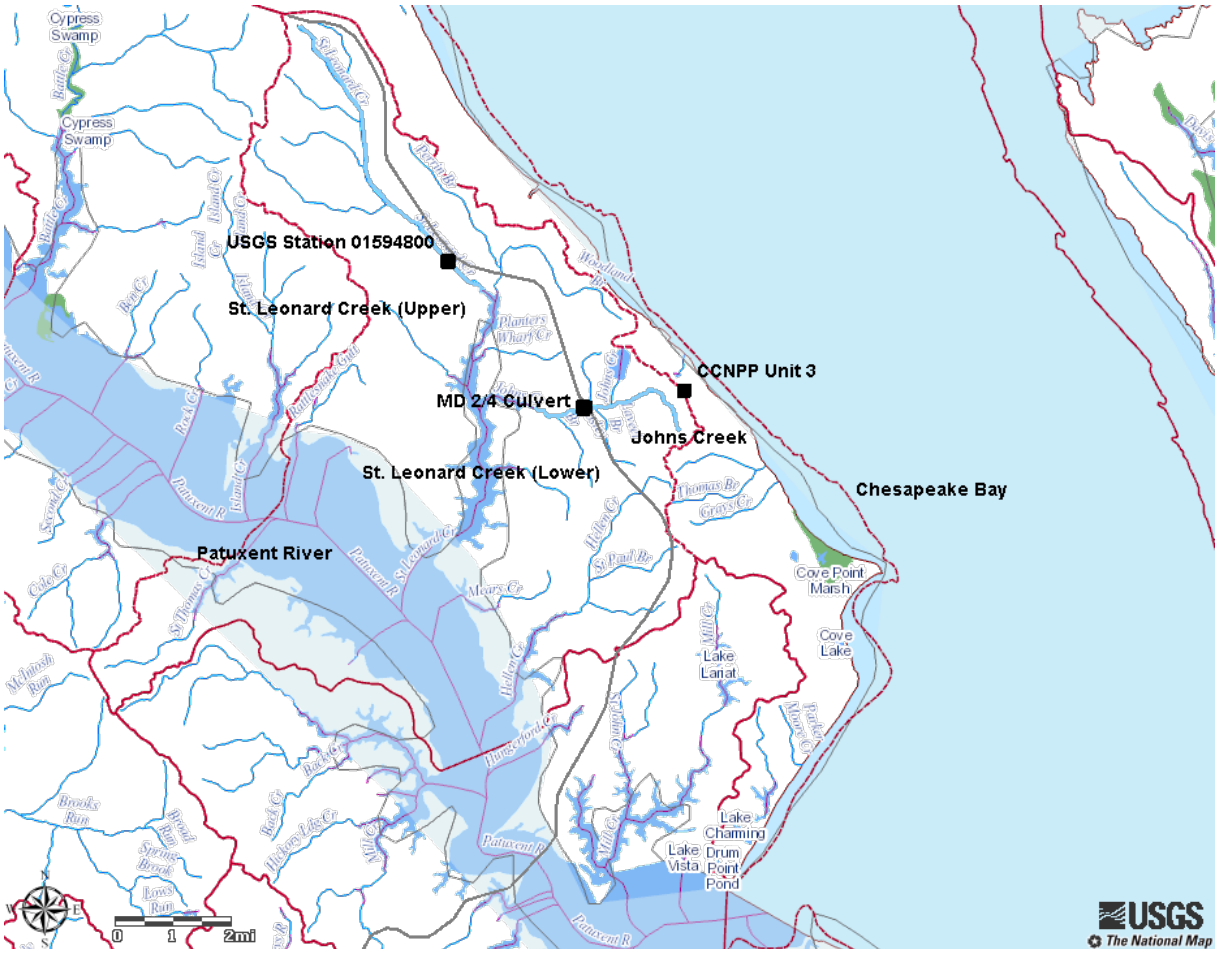

Figure 2.4.3-1. The Major Water Features near the CCNPP Unit 3 Site

Note the location of the USGS Station 01594800 and the MD 2–4 culvert, which passes Johns Creek water flow downstream to St. Leonard Creek. (Base Map Source: (USGS)

The Staff's Technical Evaluation

The staff's review of COL FSAR Section 2.4.3.1 consisted of verification of the COL applicant's analysis and development of the PMP storm. The staff first examined the Johns Creek watershed boundary and topography used for the COL applicant's PMF analysis. The COL applicant used these data to determine the drainage basin area, hydrologic characteristics of the drainage area, and the downstream boundary of the hydrologic model. The COL applicant set the culvert crossing MD 2–4 as the downstream boundary control point based on the conclusion of a sensitivity analysis of tailwater-surface elevations. These features can be seen in Figure 2.4.3-1 of this report. The COL applicant assumed a normal depth boundary condition at this location. The normal depth boundary condition means that the backwater-surface elevation in Johns Creek is not influenced by the downstream floodwater-surface elevation, because the water-surface elevation at the downstream boundary is determined by the discharge and the channel slope. However, the COL applicant did not provide an explanation of how it performed the sensitivity analysis to justify the downstream boundary location. While reviewing the COL FSAR, the staff determined through an independent assessment that the issue would not alter the staff's conclusions. Therefore, the staff did not issue an RAI.

To verify the appropriateness of the COL applicant-selected boundary condition, the staff conducted a flood frequency analysis using the annual peak flow data (1958 to 1968 and 2001 to 2003) at the USGS stream gauging station (ID# 01594800) on St. Leonard Creek. The station is about 6.4 km (4 mi) upstream of the junction between St. Leonard Creek and the Patuxent River and about 4.8 km (3 mi) northwest of the CCNPP Unit 3 site (see Figure 2.4.3-1 of this report). The staff used USGS flood frequency analysis program PeakFQ for this analysis (Flynn et al. 2006). The staff estimated that the water-surface elevation of the 500-year flood event is 4.24 m (13.9 ft), which is 13.9 m (45.5 ft) lower than the low point of MD 2–4 as it passes over Johns Creek; this is the water-surface elevation that would need to be exceeded in St. Leonard Creek for creating backwater effects upstream into Johns Creek when the MD 2–4 culvert is blocked. Due to the relatively large difference between the low point of the MD 2-4 and the 500-year flood water-surface elevation in St. Leonard Creek, the staff concluded that a flood in St. Leonard Creek downstream of Johns Creek would not affect the flood water-surface elevation upstream in Johns Creek watershed at PMP discharges. Therefore, the staff concluded that the COL applicant's assumption of the downstream boundary condition is reasonable.

The staff examined the COL applicant-estimated PMP for Johns Creek watershed. The staff confirmed the watershed area based on the USGS topographic map presented in Figure 2.4.3-1 of this report. The staff-confirmed area (6.0 km^2 (2.3 mi^2)) of the Johns Creek watershed is smaller than 26 km^2 (10 mi^2). The staff finds the COL applicant's use of the 1-hour, 2.6-km^2 (1-mi^2) PMP acceptable. To reach this determination, the staff obtained the 1-hour, 2.6-km^2 (1-mi^2) PMP depth (470 mm (18.5 in.)) using HMR-52, Figure 24. Subsequently, the staff estimated precipitation depths for 5-min, 15-min, and 30-min durations based on 6-hour and 26 km^2 (10 mi^2) precipitation in HMR-51 Figure 18, and the corresponding ratios for each duration to the 1-hour, 2.6-km^2 (1-mi^2) PMP given in HMR-52 Figures 36, 37, and 38. The staff compared its estimates to the COL applicant's PMP depth-duration curve. The staff noted that the COL applicant's PMP estimates were within ± 0.3 mm ($\pm$0.1 in.) of the staff's estimates. Therefore, the staff finds the COL applicant's estimates of 1-hour, 2.6-km^2 (1-mi^2) PMP acceptable.

American National Standards Institute (ANSI)/American Nuclear Society (ANS)-2.8-1992, "Determining Design Basis Flooding at Power Reactor Sites," states that the antecedent snowpack accumulation should be considered for drainage area where snowmelt contributes significantly to the controlling flood situation. The COL applicant did not consider the antecedent snowpack condition, because the COL applicant's assessment of historical data indicates no significant snowmelt contribution to flooding conditions. According to NOAA snow storm event data gathered in Maryland from 1993 through 2010, the maximum recorded daily snow amount is 45.7 cm (18 in.) (Horvitz and Magnus 2010). A conservative conversion to rainfall of this snowfall depth is approximately 9.1 cm (3.6 in.), assuming a 0.2 conversion factor. This amount is less than 20 percent of the PMP rainfall intensity (46.9 cm/hr (18.5 in./hr)) for the 1-hour duration. In Maryland, extreme snowfall occurs from December through February, while heavy precipitation occurs in the summer. Therefore, the staff determined that a heavy precipitation runoff event is not likely to coincide with a heavy snowpack. Therefore, the staff agreed with the COL applicant that the antecedent snowpack accumulation does not contribute significantly to the flood condition in Johns Creek watershed.

2.4.3.4.2 Precipitation Losses

Information Submitted by COL Applicant

In COL FSAR Section 2.4.3.2, "Precipitation Losses," the COL applicant discussed precipitation losses from infiltration. The COL applicant stated that the Johns Creek watershed consists primarily of wooded areas. The COL applicant used the NRCS Curve Number approach described in TR-55 (Natural Resources Conservation Service (NRCS), "Urban Hydrology for Small Watersheds," Conservation Engineering Division, Technical Release 55, June 1986). The COL applicant assumed an impervious surface (runoff curve number (RCN) equal to 98) according to guidance provided in TR-55 (NRCS 1986) for impervious areas.

The Staff's Technical Evaluation

The Johns Creek watershed is wooded except for the main channel area. Therefore, significant infiltration likely occurs, decreasing the amount of precipitation available for runoff. An assumption of impervious surface produces a higher runoff estimate. In the staff's independent runoff analysis using the Rational Method, no infiltration loss occurred, because the runoff coefficient is set to 1. The staff's approach is equivalent to assuming a completely impervious surface and therefore results in the greatest runoff for a given precipitation and drainage area, which is conservative. Therefore, the staff concluded that the COL applicant's assumption of a nearly impervious surface is a conservative approach.

2.4.3.4.3 Runoff Model

Information Submitted by COL Applicant

In COL FSAR Section 2.4.3.3, "Runoff Model," the COL applicant provides its runoff analysis using the HEC-HMS hydrologic model. The COL applicant's analysis included transformation of PMP to runoff for subbasins and routing of runoff hydrographs across the road at the downstream boundary of the watershed. The COL applicant used the NRCS unit hydrograph method. The COL applicant used an RCN of 98, assuming a nearly impervious surface.

For its analysis, the COL applicant computed the time of concentration for each subbasin using the NRCS method (USDA 1986). To consider nonlinear effects during extreme rainfall events, the COL applicant reduced the time of concentration by 25 percent and estimated lag time as 0.6 times the reduced time of concentration (USACE 2006).

The COL applicant's runoff model consisted of a large storage area and four subbasins depicted in the "HEC-HMS Watershed Schematic." The storage area is formed behind the MD 2–4 by assuming the MD 2–4 culvert to be blocked according to the recommendation in ANSI/ANS-2.8-1992 (ANS 1992). The COL applicant estimated the inflow hydrographs by applying the 1-hour, 2.6-km^2 (1-mi^2) PMP. The COL applicant combined the runoff hydrographs from four subbasins to estimate the inflow hydrograph to the storage area. The COL applicant routed the combined inflow hydrograph through the storage area using the level-pool routing method with specified stage-storage and stage-discharge curves. The COL applicant determined the stage-storage relationship using the topographic details from a USGS topographic map (USGS 1987) for the area. The COL applicant determined the stage-discharge relationship by assuming that the road would act as a standard broad-crested weir. The COL applicant provided the outflow hydrographs computed by the COL applicant's

model. The COL applicant-computed peak storage elevation is about 15.8 m (52 ft), which exceeds the MD 2–4 roadway elevation (13.9 m (45.5 ft)).

The COL applicant explained that it did not consider any effect of upstream dam breaching, because there are no dams or reservoirs in Johns Creek. The COL applicant also discussed dam failures in COL FSAR Section 2.4.4.

The Staff's Technical Evaluation

The staff reviewed COL FSAR Section 2.4.3.3 to confirm the COL applicant's approach and estimation of peak runoff in Johns Creek. The staff conducted an independent analysis using the Rational Method to transform the PMP intensity to peak runoff.

The Rational Method is common in engineering runoff analysis and is appropriate for a small drainage area like the Johns Creek watershed. The staff's bounding calculation is based on a conservative estimate of runoff discharge as inputs into the backwater calculation.

The Rational Method is given by the following equation.

$$Q = C \cdot I \cdot A$$

Where:

Q = peak runoff (L^3T^{-1})
C = runoff coefficient
I = rainfall intensity (LT^{-1})
A = drainage area (L^2)

For the first step of the peak runoff analysis, the staff reviewed the Johns Creek drainage layout provided by the COL applicant and determined that it was an appropriate characterization of the Johns Creek subbasins. For the second step, the staff estimated T_c (time of concentration) by the relation L (longest water course length) divided by V (channel velocity). The time of concentration is the sum of overland flow time and channel flow time. The staff conservatively used an overland flow velocity equal to channel velocity. Overland flow velocity in the Johns Creek watershed is expected to be slower than channel velocity because of the presence of woody vegetation in non-channel areas. The staff estimated the longest water course length (L) for each subbasin from the watershed drainage map provided by the COL applicant. The staff estimated the average channel velocity (V) using the staff's HEC-RAS hydraulic model. The staff estimated the rainfall intensity (I) for the duration T_c by interpolating or extrapolating using the second-order polynomial fit to the PMP depth-duration data. For conservatism, the runoff coefficient value (C) is assumed to be 1.0, which is appropriate and conservative for impervious surfaces, because it results in no infiltration losses. Table 2.4.3-1 of this report provides the staff-estimated parameter values of T_c, L, and V. The peak discharge values in Table 2.4.3-1 of this report differ from those presented in COL FSAR Table 2.4-23 by factors ranging from about 0.8 to 1.2 with an average of about 1.04. The significance of this difference is discussed in terms of the water-elevation estimates, which are computed using these differing peak discharge estimates. Based on the staff's independent analysis, the staff concluded that the COL applicant's runoff model is acceptable.

The staff confirmed that there are no upstream dams or reservoirs in the Johns Creek watershed. The staff's independent assessment of dam failures in the Patuxent River basin and

their potential effects on water-surface elevations in Johns Creek watershed is described in Section 2.4.4 of this report.

Table 2.4.3-1. Staff-Estimated Values Used in the PMF Calculation

Subbasin	Drainage Area, km^2 (mi^2)	L, m (ft)	V, m/s (fps)	T_c, min	Q_p, m^3/s (cfs)
1	2.315 (0.894)	1,923.3 (6,310.2)	1.30 (4.27)	24.7	494.29 (17,455.6)
2	2.152 (0.831)	2,252.4 (7,389.8)	0.54 (1.78)	69.2	247.80 (8,750.8)
3	0.357 (0.138)	925.7 (3,037.1)	1.07 (3.51)	14.4	98.74 (3,487.0)
4	1.088 (0.420)	1,533.7 (5,031.8)	1.07 (3.51)	23.9	235.70 (8,323.6)

Notes:
1. Drainage areas are the confirmed values from the COL applicant's estimate.
2. Water course length (L), channel velocity (V), and time of concentration (T_c) are estimated from the USGS topographic map for the Core Point 7.5-minute quadrangle.
3. The peak runoffs (Q_p) are estimated by the Rational Method.

2.4.3.4.4 Probable Maximum Flood Flow

Information Submitted by COL Applicant

The COL applicant conducted a steady-state backwater calculation using the HEC-RAS hydraulic model for six different scenarios. The COL applicant based these scenarios on flow rates generated by the COL applicant's runoff model for each subbasin at the peak flow time (50 min) determined for Subbasin 4, which is the most upstream subbasin in the Johns Creek watershed. The COL applicant subsequently examined five additional water-surface-elevation profiles using estimated flows at 5-min intervals thereafter. The time period examined included the time when the peak flow was determined to pass the MD 2–4 culvert outlet. The COL applicant's most conservative scenario for maximum water-surface elevations was the one that coincided with the peak flow estimated for Subbasin 4.

The Staff's Technical Evaluation

The staff reviewed COL FSAR Section 2.4.3.4, "Probable Maximum Flood Flow," to examine the COL applicant's scenarios and approach for the PMF flow analysis. Upon review of COL FSAR Revision 9, the staff noted inconsistencies between COL FSAR Table 2.4-24 "PMF Flow Rates" and COL FSAR Figures 2.4-18, "Johns Creek Watershed" and 2.4-25, "HEC-RAS Cross-Section Locations," which relate the cross sections to the contributing subbasins. Therefore, in RAI 389, Question 02.04.03-1, the staff requested that the COL applicant update the COL FSAR to ensure that the representation of the Johns Creek watershed in COL FSAR, Revision 9, Figures 2.4-11 and 2.4-18 was consistent with the PMF flow rates presented in COL FSAR, Revision 8, Table 2.4-20. The staff reviewed the COL applicant's May 23, 2013, response to RAI 389, Question 02.04.03-1. The COL applicant's response included an update of the relevant figure and table numbers between COL FSAR Revision 8 and COL FSAR Revision 9. COL FSAR, Revision 8, Figure 2.4-11, Figure 2.4-18, and Table 2.4-20 are renumbered to COL FSAR Figure 2.4-18, Figure 2.4-25, and Table 2.4-24, respectively, in COL FSAR Revision 9. The COL applicant described revisions to the COL FSAR Table 2.4-24 that clearly identify the correspondence between HEC-RAS cross sections and the contributing subbasins.

The COL applicant also identified minor typographical errors in the COL FSAR Revision 8. The staff reviewed these changes and determined that the staff's independent calculations were similar to the COL applicant's in terms of peak discharges and water-surface elevations. Both the COL applicant's and the staff's estimates of peak water-surface elevations were significantly lower than the safety-related plant grade (25.8 m (84.6 ft)). The COL applicant calculated a maximum water-surface elevation of 20.0 m (65.7 ft); the staff estimate for maximum water-surface elevation was 21.2 m (69.5 ft). The difference between the COL applicant's and the staff's PMF water-surface elevation estimates is due to the staff's use of extreme values of Manning's roughness coefficient in its confirmatory analysis. Since the staff-determined PMF water-surface elevation, although higher than the COL applicant's estimate, does not reach the site grade, the staff finds the COL applicant's conclusions acceptable. Therefore, the staff agreed with the COL applicant's modifications to a future revision of the COL FSAR and closed RAI 389, Question 02.04.03-1, because it has been adequately addressed in the COL applicant's response. **RAI 389, Question 02.04.03-1 is being tracked as a confirmatory item** to ensure the next revision of the COL FSAR is updated accordingly**.**

The staff-generated peak runoffs from PMP that were used in the PMF calculations were based on the conservative assumption that no subsurface infiltration occurred; this assumption resulted in the maximum runoff for a given precipitation amount. The staff's estimate of total peak runoff (1,077 m^3/s (38,017 cfs)) is higher than the COL applicant's estimate (1,049 m^3/s (37,055 cfs)), which the staff computed by adding the subbasin PMF flow rate values. The staff-estimated values for drainage parameters and peak runoff are summarized in Table 2.4.3-1 of this report. This comparison suggests that both the staff's and the COL applicant's estimates are conservative. The difference in the two estimates is attributed to the fact that the staff used a different method to estimate peak runoff. The PMF flows in Table 2.4.3-1 of this report are used as inputs to the staff's independent HEC-RAS backwater analysis.

2.4.3.4.5 Water-Elevation Determinations

Information Submitted by COL Applicant

In COL FSAR Section 2.4.3.5, "Water Level Determination," the COL applicant described a steady-state backwater analysis. The COL applicant's backwater analysis relied on a HEC-RAS hydraulic model to estimate water-surface elevations with inputs including the hydraulic characteristics of the watershed geometry and boundary conditions.

The COL applicant did not calibrate the model to historic flood data in Johns Creek but instead used a conservative selection of model parameters. The COL applicant specified the Manning's roughness coefficient values as 0.035 for the main channel and 0.142 for the overbank floodplain areas. The COL applicant extracted the channel cross-section data from the USGS topographic maps (USGS 1987).

The COL applicant set the downstream boundary about 244 m (800 ft) downstream of the MD 2–4 culvert crossing. The MD 2–4 culvert crossing the highway is the downstream control point of the COL applicant's hydraulic model. In its analysis, the COL applicant assumed the culvert to be blocked. Consequently, the COL applicant treated the road crossing as an inline weir in its HEC-RAS hydraulic model; the COL applicant chose a weir coefficient value of 2.6. The COL applicant set the downstream boundary condition to normal depth that requires flow rate and channel slope information in the HEC-RAS model. The COL applicant's assumption

about the downstream boundary condition is that the backwater-surface elevation is not influenced by the downstream floodwater-surface elevation.

The COL applicant used the mixed flow option for the HEC-RAS model setup, allowing the possibility of both subcritical and supercritical flows. The COL applicant's estimate of maximum floodwater-surface elevation is 20.0 m (65.7 ft) 19.8 m (65.0 ft) relative to mean sea level (MSL)) at the most upstream cross section for the first scenario.

The Staff's Technical Evaluation

The staff reviewed the COL applicant's backwater analyses presented in COL FSAR Section 2.4.3.5, in which an HEC-RAS-based hydraulic model is used. Based on its independent confirmatory HEC-RAS analysis, the staff finds the COL applicant's backwater analysis acceptable. The staff's approach was to incorporate conservatism in the model setup and inputs to provide a bounding estimate of the maximum water-surface elevation.

The HEC-RAS cross-section locations for this analysis are plotted in Figure 2.4.3-2 of this report. The staff reviewed the information provided in the COL FSAR related to the COL applicant's HEC-RAS hydraulic modeling, including the selection of the location of the cross sections used for model configuration. Based on its independent review of the site topography and surrounding area, as well as its observations during the site audit, the staff finds the location and number of cross sections used by the COL applicant adequate. In its independent confirmatory analysis, the staff defined the cross sections at the same locations as did the COL applicant. The staff's approach provides a model layout comparable to that used in the COL applicant's analysis by defining similar channel geometry and capacity. To define the cross-section profiles, the staff extracted topographic details from USGS 30-m resolution *National Elevation Dataset*[2] (USGS 2013a).

[2] Available on the Internet at the following location: http://ned.usgs.gov/

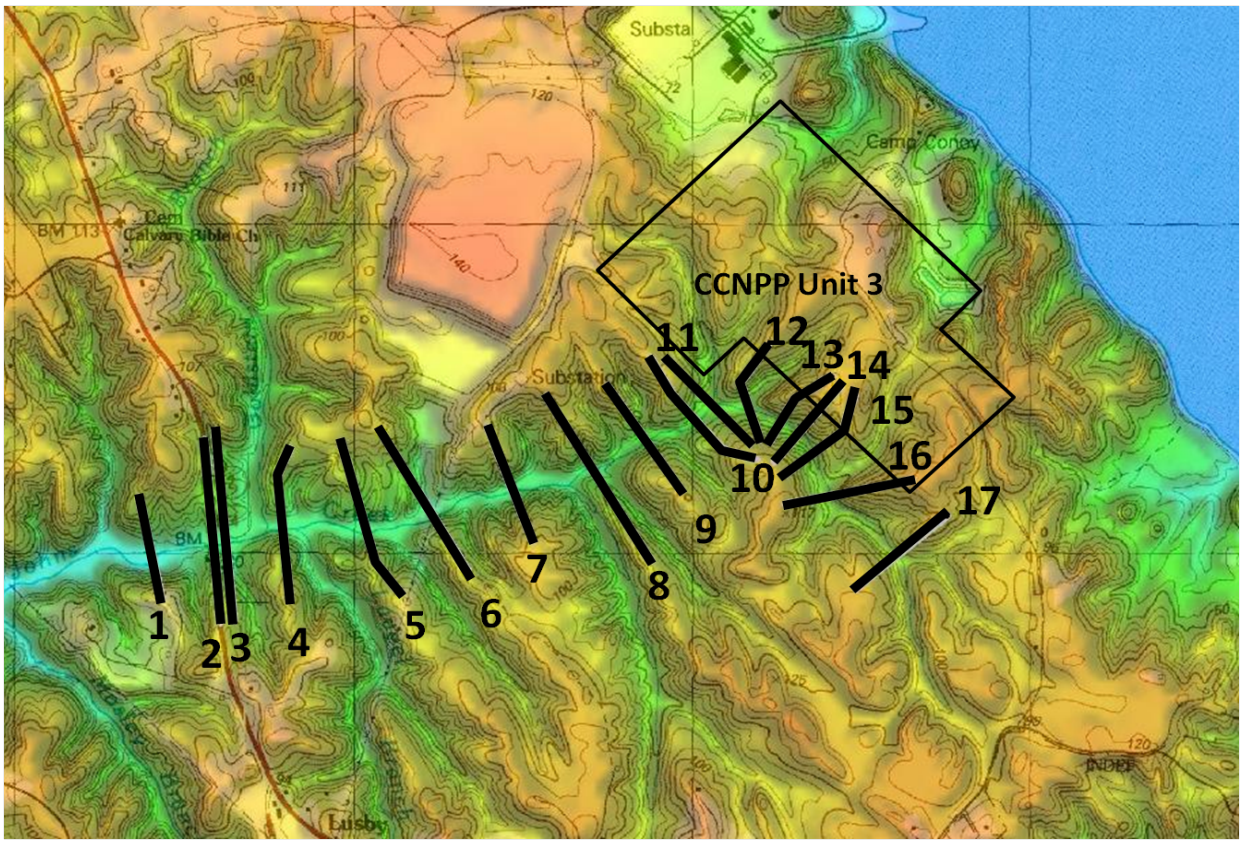

Figure 2.4.3-2. HEC-RAS Cross-Section Locations

(Base Map Source: UGSG)

The staff-estimated inflows used in the HEC-RAS hydraulic model setup are presented in Table 2.4.3-2 of this report. The staff assigned cumulative inflows to the appropriate cross sections (Nos. 17, 11, 7, and 3) in the HEC-RAS hydraulic model. Other inputs to HEC-RAS include the Manning's roughness coefficient and contraction/expansion coefficient. The staff used a conservative Manning's roughness coefficient value of 0.142 for both the main channel and the floodplain. The staff confirmed that the Manning's roughness coefficient is appropriate for a wooded floodplain similar to that currently defining the Johns Creek watershed. The main channel would likely have a much lower roughness coefficient value, like the one used by the COL applicant. Therefore, the staff's estimate is more conservative than the COL applicant's, because it would result in a higher water-surface elevation. The staff set the contraction and expansion coefficient for channels to 0.1 and 0.3, respectively, for all cross sections, except for cross-section locations 12 and 13. For cross-section locations 12 and 13, the staff set the expansion and contraction coefficients to 0.6 and 0.8, respectively, to account for channel meandering at these cross sections near the CCNPP Unit 3 site.

Table 2.4.3-2. Maximum Water-Surface Elevations from the Applicant and Staff Analyses in the Johns Creek Drainage (for HEC-RAS cross-section locations depicted in Figure 2.4.1-3)

Cases	PMF Elevation at Cross-Section Locations, m (ft)					
	17	16	15	14	13	12
Applicant-COL FSAR Table 2.4-21	20.0 (65.7)	18.6 (60.9)	17.3 (56.8)	17.3 (56.7)	16.7 (54.8)	16.6 (54.6)
Staff's Analysis using PMF flow in FSAR Table 2.4-19	21.2 (69.5)	18.5 (60.7)	17.1 (58.2)	17.2 (56.5)	16.7 (54.7)	16.4 (53.9)
Staff's Analysis using PMF flow in Table 2.4.3-1	21.4 (70.2)	18.8 (61.5)	18.0 (59.0)	17.4 (57.1)	16.8 (55.1)	16.5 (54.3)

Notes:
1. The COL applicant model used Manning's roughness coefficient values of 0.035 for the main channel and 0.142 for the floodplain area. The staff's analysis used the coefficient value of 0.142 for both the main channel and the floodplain.
2. The COL applicant estimated the PMF flows using the HEC HMS model, and the staff independently estimated them using the Rational Method.

The staff assumed a normal depth condition at the downstream boundary of the HEC-RAS hydraulic model. In the normal depth condition, the water-surface elevation at the boundary is determined by the outflow and bed slope, assuming a uniform flow. The staff estimated the slope to be 0.01 from the topographic data. As the staff confirmed in Sections 2.4.2 and 2.4.4 of this report, the floodwater-surface elevation in St. Leonard Creek due to extreme precipitation or dam breach, located downstream of Johns Creek watershed, would not influence the water-surface elevations during a PMF in Johns Creek watershed. Therefore, the staff concluded that a normal depth condition at the downstream boundary is appropriate.

In this analysis, the staff also conservatively assumed a blocked culvert at the MD 2–4 roadway crossing in conformance to the ANSI/ANS-2.8-1992 guideline (ANS 1992). A blocked culvert would result in all flow going over the MD 2–4 roadway and therefore would result in a higher water-surface elevation upstream of the MD 2–4 roadway in Johns Creek. While developing its independent HEC-RAS hydraulic model, the staff used two closely spaced cross sections in the model to explicitly represent the road. This treatment allowed the simulation of the backwater effect due to a blocked culvert and road without a parameterization for an inline weir, which was included in the COL applicant's model setup.

Table 2.4.3-2 of this report compares the PMF water-surface elevations from the COL applicant's HEC-RAS hydraulic analysis with those from the staff's analyses. The table presents results from the water-surface profile examined by the COL applicant associated with the one that produced the highest water-surface elevations near the site; the staff adjusted the COL applicant's values to the NGVD29 datum. First, the staff conducted the analysis using PMF discharges estimated by the COL applicant. The staff-estimated highest PMF water-surface elevation is 21.2 m (69.5 ft) at cross-section location 17. The staff's estimate is about 1.15 m (3.8 ft) higher than the COL applicant's estimate. This difference is mainly attributed to the higher values of Manning's roughness coefficient used for the main channel in the staff's analysis. Second, the staff conducted the analysis using the PMF discharges computed independently using the Rational Method. The Rational Method provided a slightly more conservative estimate of total peak discharge. The staff-computed value of the highest PMF water-surface elevation is 21.4 m (70.3 ft) at cross-section location 17. This increased PMF water-surface elevation is 8.5 m (27.7 ft) below the drainage divide elevation of 29.9 m (98.0 ft) in the switchyard of CCNPP Unit 3. The staff concluded that both the COL applicant's

and staff's peak water-surface elevations, while different, are significantly lower than the elevation of the drainage divide.

2.4.3.4.6 Coincident Wind-Wave Activity

Information Submitted by COL Applicant

The COL applicant did not conduct a wind-wave height estimation for the PMF analysis, because the expected top width of the water-surface for the PMF flow in Johns Creek would not be wide enough to generate any significant wave height exceeding the elevation difference between the drainage divide and the PMF water-surface elevation.

The Staff's Technical Evaluation

The staff accepted the COL applicant's reasoning regarding wind-wave effects and also did not perform a wind-wave analysis coincident with a PMF in Johns Creek watershed. The expected width of water surface near the drainage divide is about 60 to 90 m (200 to 300 ft) during the PMF flood. The width is not sufficient to provide an adequate fetch to generate a wave height that would exceed the elevation difference of 8.5 m (27.8 ft) between the PMF water-surface elevation and the elevation of the drainage divide. Therefore, the staff's analysis confirmed that the wind-wave activity in Johns Creek will not adversely affect the CCNPP Unit 3 site.

2.4.3.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.3.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR Revision 4. On the basis of its review, the staff confirmed that the COL applicant addressed the required information related to regional PMF estimates. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to regional PMF estimates incorporated by reference in the COL FSAR will be documented in the staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will update Section 2.4.3 of this report to reflect the final disposition of the design certification application.

2.4.4 Potential Dam Failures

2.4.4.1 *Introduction*

COL FSAR Section 2.4.4 addresses plausible dam failures to ensure that any potential hazard to safety-related structures due to failure of onsite, upstream, and downstream water-control structures is considered in the plant design.

2.4.4.2 *Summary of Application*

This section presents the staff's review of the COL applicant's estimation of the flood water-surface elevation caused by different dam failures. The specific areas of review are as follows: (1) Dam failure permutations; (2) unsteady flow analysis of potential dam failures;

(3) water-surface elevation determination; and (4) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

This section of the COL FSAR also addresses the site-specific information about potential dam failures. COL FSAR Section 2.4.4 incorporates by reference U.S. EPR FSAR Revision 4, Tier 2, Section 2.4.4, "Potential Dam Failures."

The COL applicant addressed the issues as follows:

COL Information Item 2.4-4

> A COL applicant that references the U.S. EPR design certification will verify that the site-specific potential hazards to the safety-related facilities due to the failure of upstream and downstream water control structures are within the hydrogeologic design basis.

The COL applicant provided additional information in COL FSAR Section 2.4.4 to address COL Information Item 2.4-4, "Potential Dam Failures," from U.S. EPR FSAR Tier 2, Revision 4, Table 1.8-2, "U.S. EPR Combined License Information Items." COL Information Item 2.4-4 states that the COL applicant will verify that flood hazards caused by failures of upstream and downstream water control structures are within the hydrologic design bases. In COL FSAR Section 2.4.4, the COL applicant presented the analysis for the flood elevation site characteristic associated with plausible dam failure events. In addition, in COL FSAR Section 2.4.4, the COL applicant presented the analysis of the maximum water-surface elevation at the site associated with a flood caused by dam failures and examined the potential effect of a flood wave due to dam breaches propagating through the tributaries of Patuxent River and upward into the St. Leonard and Johns Creek basins.

2.4.4.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the identification of floods, flood design considerations, and potential dam failures, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.4.

The applicable regulatory requirements for identifying the effects of dam failures are set forth in the following:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.
- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used appropriate sections of the following Regulatory Guides for the acceptance criteria identified in NUREG-0800, Section 2.4.4:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding or those associated with the MWIS
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site

The regulatory basis of the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.4.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.4. The staff confirmed that the COL applicant addressed the relevant information related to the flood elevation site characteristic associated with the most severe plausible dam failure event. The staff's technical review of this COL application included an independent review of the COL applicant's information in the COL FSAR. The staff supplemented this information with other publicly available sources of data.

2.4.4.4.1 Potential Dam Failures

Information Submitted by COL Applicant

In COL FSAR Section 2.4.4, the COL applicant presented the analysis of the maximum water-surface elevation at the site associated with a flood caused by dam failure. The CCNPP Unit 3 site is located on the western shore of the Chesapeake Bay. The Patuxent River runs on the western side of the peninsula on which the proposed site exists. The COL applicant estimated the potential effect of a flood caused by dam breaches on the tributaries of Patuxent River, including the St. Leonard and Johns Creek drainages. Johns Creek drainage is located west of the CCNPP Unit 3 site and is bounded on the east by the drainage boundary at an elevation of 29.9 m (98.0 ft).

The COL applicant noted that there are no dams on Johns Creek or St. Leonard Creek, but there are two dams on the Patuxent River: The Rocky Gorge Dam and the Brighton Dam. These dams are located about 105 km (65 mi) and 126 km (78 mi), respectively, upstream of the mouth of St. Leonard Creek. The combined storage volume of the two dams is about 60×10^6 m^3 (49,000 ac-ft) (USACE 2013).

The COL applicant assumed that the combined volumes of water stored behind the dams were instantaneously introduced to the tidally affected region (with an area of 106 km^2 (40.9 mi^2)) near the Patuxent River mouth. Further, the COL applicant assumed no discharge from this region to the Chesapeake Bay to maximize the water-surface elevation. The COL applicant

estimated that the water-surface elevation in the tidal region would increase about 0.6 m (2 ft) by adding the combined dam storage volume directly to this area. The COL applicant concluded that this increase would not alter its earlier estimate of the PMF water-surface elevation in Johns Creek.

The Staff's Technical Evaluation

The staff examined the locations and storage volumes of dams within the Patuxent River watershed based on the data from *National Inventory of Dams* (NID) (USACE 2013b). The staff identified only two dams (Rocky Gorge Dam and Brighton Dam) that could potentially affect the CCNPP Unit 3 site. The staff estimated the river distance between the two dams and the confluence of the Patuxent River and St. Leonard Creek as about 111 km (69 mi) and 134 km (83 mi), respectively. Using the NID database, the staff determined that the combined maximum storage volume of two dams is 51.3×10^6 m^3 (41,560 ac-ft); this value is less than that reported by the COL applicant. No dams exist on Johns Creek or St. Leonard Creek. The staff concluded that the breaching of several dams in other watersheds would not notably affect the backwater in the Patuxent River, because they discharge into a large body of water – the Chesapeake Bay (with an approximate volume of 1.62×10^{11} m^3 (1.32×10^8 ac-ft)) – which is connected to the Atlantic Ocean, so flood waters discharging to the bay would be rapidly dissipated. Therefore, only the failure of the two upstream dams on the Patuxent River can potentially increase the PMF water-surface elevation in St. Leonard Creek and Johns Creek.

The staff examined the potential route of flood-wave propagation caused by failures of dams on the Patuxent River. After a dam breaks, the flood wave would propagate downstream through the river. Most of the flood-wave energy would be dissipated through the floodplain and the Chesapeake Bay system. A part of the flood wave could also propagate into Johns Creek through St. Leonard Creek via backwater effects; because these two creeks are connected to the Patuxent River (see Figure 2.4.4-1 of this report).

The staff used a simple bounding calculation to estimate the incremental increase in water-surface elevation resulting from upstream dam failures using a method described by the Washington State Department of Ecology (WDOE 2007). The staff used the method described in the WDOE 2007 document to select a flow velocity equal to 0.4 m/s (1.4 ft/s), which is associated with rough, mildly sloped channels. The staff's selection of flow velocity is conservative, because slower velocities generate higher water-surface elevations.

The staff then examined the topography near the confluence of St. Leonard Creek and the Patuxent River using the USGS National Map Viewer (http://viewer.nationalmap.gov/viewer/) and estimated that the width of the Patuxent River at this location is 3.2 km (2 mi). The staff estimated the peak discharge from failures of the Rocky Gorge and Brighton dams using the Froehlich equation (Wahl 1998; Froehlich 1995). The staff used the maximum dam storage volumes and dam heights for the two dams as reported in the NID database in the Froehlich equation. The staff used this information to compute the incremental water-surface elevation rises of about 3.6 m (11.8 ft) and 6.9 m (22.5 ft) resulting from the postulated Brighton and Rocky Gorge dam failures, respectively. The staff then conservatively 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; both of these assumptions are conservative and would result in higher water-surface elevations in the Patuxent River. At the confluence of St. Leonard Creek and the Patuxent River, the staff-estimated total incremental water-surface elevation increase is 10.5 m (34.3 ft).

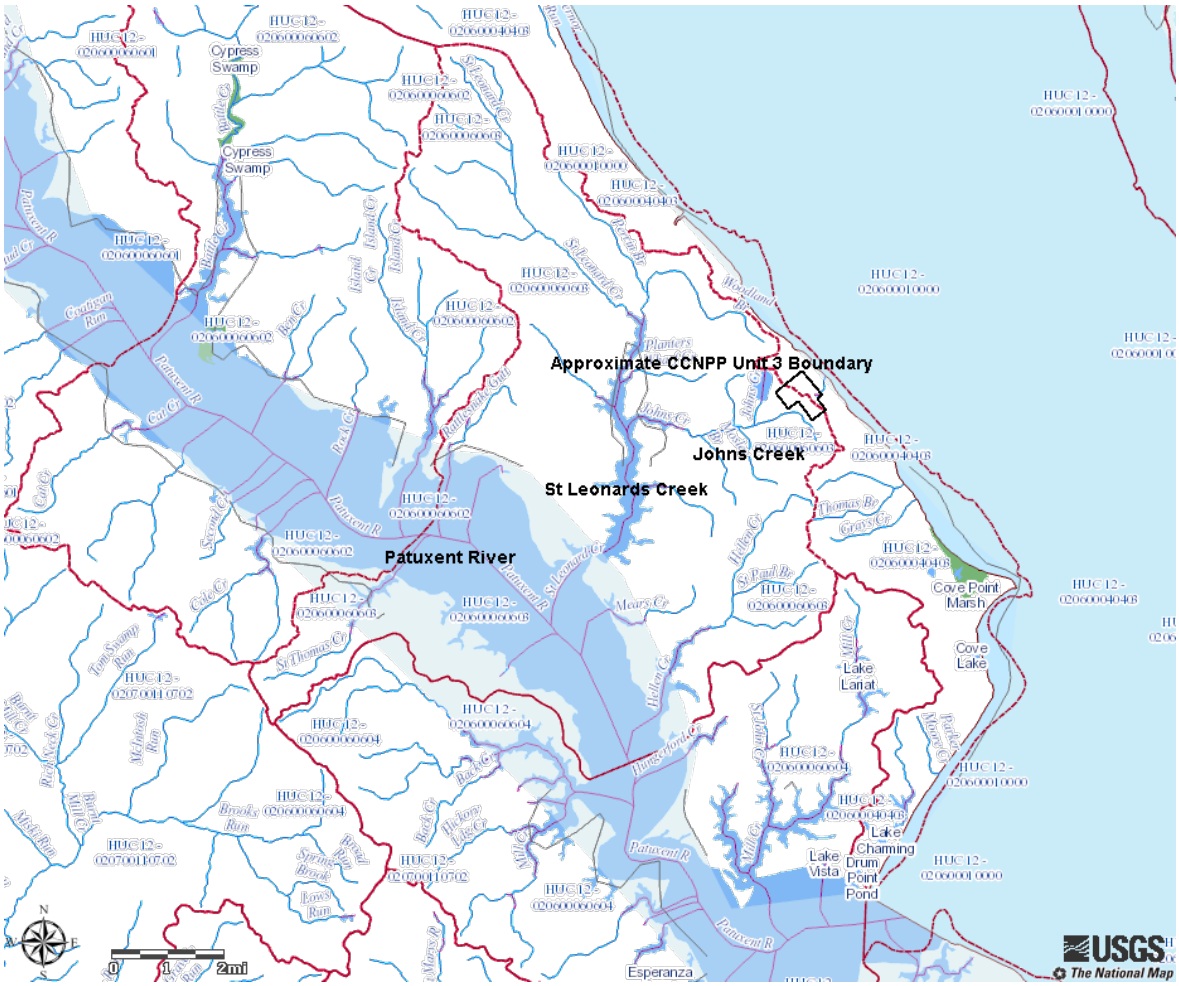

Figure 2.4.4-1. Surface-Water Bodies Considered for the Analysis of Flood Level Caused by the Combined Failures of Rocky Gorge and Brighton Dams

The water bodies include the Patuxent River, St. Leonard Creek, and Johns Creek; the approximate CCNPP Unit 3 site boundary is shown. (Base Source Map: USGS)

At the Solomons Island NOAA station (ID# 8577330), the highest observed tide water-surface elevation is about 1.34 m (4.4 ft) (NOAA 2013a). The staff conservatively added the incremental increase in the Patuxent River water-surface elevation caused by dam failures (20.5 m (34.3 ft)) and the highest historical tide water-surface elevation at Solomons Island (1.34 m (4.4 ft)) to conservatively estimate a dam failure peak water-surface elevation of 11.7 m (38.5 ft) at the confluence of St. Leonard Creek and the Patuxent River. The staff-estimated peak water-surface elevation, 11.7 m (38.5 ft), is substantially below the elevation of 13.9 m (45.5 ft) of MD 2–4, the downstream boundary condition of PMF analysis described in Section 2.4.3 of this report. The staff measured the Patuxent River width at this elevation and determined that it would widen to about 4.1 km (2.6 mi) at this elevation and, therefore, some additional spreading of the floodwater would occur and lead to an overall reduction in the combined peak water-surface elevation. Therefore, the staff determined that an upstream dam failure event would not cause a higher water-surface elevation at the downstream boundary of the PMF analysis described in Section 2.4.3 of this report. This staff determination

demonstrated that the PMF analysis described in Section 2.4.3 of this report would result in higher water-surface elevations near the CCNPP Unit 3 site than those expected during an upstream dam failure event. The conservatism used in the staff analysis exceeds that used by the COL applicant and does not alter the COL applicant's conclusion. The staff concluded that the dam failure would not contribute to backwater effects in Johns Creek that may adversely affect the CCNPP Unit 3 site.

2.4.4.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.4.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR Revision 4. On the basis of its review, the staff confirmed that the COL applicant addressed the required information related to potential dam failures. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to potential dam failure incorporated by reference in the COL FSAR will be documented in the staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will update Section 2.4.4 of this report to reflect the final disposition of the design certification application.

2.4.5 Probable Maximum Surge and Seiche Flooding

2.4.5.1 *Introduction*

COL FSAR Section 2.4.5, "Probable Maximum Surge and Seiche Flooding," addresses the probable maximum surge and seiche flooding to ensure that any potential hazard to the safety-related SSCs at the proposed site has been considered in compliance with NRC regulations.

This section presents the evaluation of the following topics based on data provided by the COL applicant in the COL FSAR and information available from other sources: (1) PMH that causes the probable maximum surge as it approaches the site along a critical path at an optimum rate of movement; (2) probable maximum wind storm (PMWS) from a hypothetical extratropical cyclone or a moving squall line that approaches the site along a critical path at an optimum rate of movement; (3) a seiche near the site and the potential for seiche wave oscillations at the natural periodicity of a water body that may affect the elevations of the floodwater surface near the site or cause a low water-surface elevation affecting safety-related water supplies; (4) wind-induced wave runup under PMH or PMWS winds; (5) effects of sediment erosion and deposition during a storm surge and seiche-induced waves that may result in blockage or loss of function of SSCs important to safety; (6) the potential effects of seismic and non-seismic information about the postulated design bases and how they relate to a surge and seiche in the vicinity of the site and the site region; and (7) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.5.2 *Summary of Application*

This section of the COL FSAR addresses the information related to probable maximum surge and seiche flooding in terms of effects on structures and water supply. COL FSAR

Section 2.4.5 incorporates by reference U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.5, "Probable Maximum Surge and Seiche Flooding." In this section, the COL applicant provided supplemental information to address COL-specific information identified in U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-5

> A COL applicant that references the U.S. EPR design certification will provide site-specific information on the probable maximum surge and seiche flooding and determine the extent to which safety-related plant systems require protection. The applicant will also verify that the site-specific characteristic envelope is within the design maximum flood level, including consideration of wind effects.

The COL applicant provided additional information in COL FSAR Section 2.4.5 to address U.S. EPR FSAR Tier 2, Revision 4, Table 1.8-2, "U.S. EPR Combined License Information Items," COL Information Item 2.4-5, which describes probable maximum surge and seiche flooding at the CCNPP Unit 3 site. The COL applicant stated that two categories of meteorological events can cause coastal flooding at the CCNPP Unit 3 site: Hurricanes and northeasters. Northeasters are relatively long-lived wind events that occur along the Atlantic coast; northeaster winds blow from the northeast toward the southwest with wind speeds that are lower than those that are typically associated with hurricanes.

Based on a review of the literature, the COL applicant reported in the COL FSAR that 281 hurricanes have hit the U.S. East Coast between 1851 and 2005, and 12 of Category I or greater on the Saffir-Simpson scale have passed through Maryland and Virginia. The COL applicant stated that hurricanes moving from the south to the north in the open ocean generally cause an increase in water-surface elevation at the mouth of the Chesapeake Bay, and, at the same time, the northerly winds of the hurricane cause a set down in the northern part of the Chesapeake Bay. The COL applicant described a second scenario in which hurricanes move from the southeast along the western edge of the Chesapeake Bay. The COL applicant stated that the highest sustained wind speed during PMH would come from the east or southeast when the eye of the hurricane would be located a distance equal to the radius to maximum winds southwest of the CCNPP site.

The COL applicant stated that no significant oscillations within the Chesapeake Bay are observed in the storm surge records and that the shoreline near the MWIS would be protected against the PMSS and coincident wind waves by structural measures, and water-proofing pump motors, and electrical equipment. The COL applicant also stated that flood-protection measures relevant to the CCNPP Unit 3 UHS MWIS are described in COL FSAR Section 3.8.

2.4.5.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the effects of PMSS, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.5.

The applicable regulatory requirements for the identifying of PMSS hazards, design considerations, and the associated acceptance criteria, are set forth in the following:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.
- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water-surface elevations at the site.

The staff also used appropriate sections of the following Regulatory Guides for the acceptance criteria identified in NUREG-0800, Section 2.4.5:

- RG 1.29, "Seismic Design Classification," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to those SSCs intended to protect against the effects of flooding or those associated with the MWIS
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site

The regulatory basis of the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.5.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.5. The staff confirmed that the information in the COL application addresses the PMSS flooding. The staff's technical review of this section includes an independent review of the COL applicant's information contained in the COL FSAR and in the COL applicant's responses to the staff RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented in COL FSAR Section 2.4.5.

2.4.5.4.1 Probable Maximum Winds and Associated Meteorological Parameters

Information Submitted by COL Applicant

The COL applicant stated that two categories of meteorological events can cause coastal flooding at the CCNPP Unit 3 site: Hurricanes and northeasters. Based on relatively long periods of water-surface-elevation observations by NOAA at Baltimore and Annapolis, MD, and Sewells Point, VA tide gauges, the COL applicant stated that although the third-highest reported water-surface elevation at the Sewells Point tide gauge occurred during a winter storm

(a northeaster) (see NOAA 2013b), generally, the highest water-surface elevations are expected during hurricanes.

Table 2.4.5-1. COL Applicant's Probable Maximum Hurricane Parameter Values

Parameter, units	Symbol	Range
Peripheral Pressure, cm (in.) of Hg	P_w	76.50 (30.12)
Central Pressure, cm (in.) of Hg	P_o	67.28 (26.49)
Radius of Maximum Winds, km (nautical mi)	R	18.5 to 48.2 (10 to 26)
Forward Speed, km/hr (knots/hr)	T	31.5 to 70.4 (17 to 38)

Hg = mercury; in. of Hg = one-thirtieth of atmospheric pressure (e.g., 0.49 psia).

The COL applicant estimated the PMH winds using guidance provided in NOAA National Weather Service (NWS) Report 23 (NOAA 1979). A summary of the COL applicant's PMH parameters is provided in Table 2.4.5-1 below. The COL applicant's updated values were updated in the COL FSAR, Revision 7 as described in the COL applicant's June 30, 2010, response to RAI 103, Question 02.04.05-2.

Using the above characterization of a PMH, the COL applicant estimated that the pressure difference between the hurricane peripheral and central pressures, ΔP, is 9.22 cm of Hg (3.63 in. of Hg).

The Staff's Technical Evaluation

In RAI 103, Question 02.04.05-1, the staff requested that the COL applicant explain why the storm parameters, obtained from the USACE (1986) reference and reported in the COL FSAR, are consistent with the PMH estimation procedure described by NOAA, or justify an alternative approach. In a May 20, 2009, response to RAI 103, Question 02.04.05-1, the COL applicant estimated the PMH winds using guidance provided by NOAA NWS (NOAA 1979), also referred to as NWS 23.

The staff used NWS 23 guidance to independently estimate the PMH parameters for the CCNPP Unit 3 site. The staff-estimated PMH parameters are given in Table 2.4.5-2 below.

Table 2.4.5-2. PMH Parameter Values Estimated by Staff

Parameter, units	Value/Range	NWS 23 (NOAA 1979) Source
Latitude, degrees north	36.6	
Coriolis Parameter f, (1/s)	8.7×10^{-5}	
Coastal Distance, km (nautical mi)	4,200 (2,268)	Figures 1.1 and 1.2
Peripheral Pressure Pw, cm (in.) of Hg	76.5 (30.12)	Section 2.2.2
ΔP, cm (in.) of Hg	9.17 (3.61)	Figure 2.3
Central Pressure P_o, cm (in) of Hg	67.33 (26.51)	$P_w - \Delta P$
Radius of Maximum Winds R, km (mi)	17.8-47.3 (11.1-29.6)	Figure 2.5
Forward Speed T, km/hr (knots/hr)	26.3-68.0 (14.2-36.7)	Figure 2.7
Direction, degrees clockwise from north	69-153	Figure 2.9

Coefficient K	78.8	Figure 2.11
Moving Hurricane Gradient Velocity, km/hr (mph)	239.5 (149.7)	Equation 2.2
Stationary Hurricane Gradient Velocity, m/s (mph)	227.5 (142.2)	Equation 2.4

Based on these parameter values, the staff independently estimated that the maximum wind speed for a moving and a stationary hurricane at the CCNPP Unit 3 site would be approximately 239.5 and 227.5 km/hr (149.7 and 142.2 mph), respectively. The COL applicant computed a PMH maximum 10-m, 10-min wind speed at landfall at the CCNPP Unit 3 site of 245.6 km/hr (152.6 mph) and 179.8 km/hr (111.7 mph), respectively. The staff's estimate of the peripheral pressure of PMH matches that estimated by the COL applicant. The staff's estimate of the difference between the hurricane peripheral and central pressures, ΔP, is smaller than the COL applicant's estimate (see Table 2.4.5-1 and Table 2.4.5-2 above). Since a larger ΔP results in a more intense hurricane, the staff concluded that the COL applicant's estimate of ΔP is conservative.

RAI 103, Question 02.04.05-2, requested that the COL applicant describe the conservatism associated with the empirical method used to estimate surge elevations relative to the RG 1.59, USACE Engineering Manual 1110-2-1110 (USACE 2006), and the NOAA NWS's Sea, Lake, and Overhead Surges from Hurricanes (SLOSH) model. The staff reviewed the COL applicant's June 30, 2010, response to RAI 103, Question 02.04.05-2. The COL applicant replaced the older empirical analysis with an analysis to estimate PMSS by the SLOSH model driven by the PMH. Based on the reasons given above, the staff finds that the COL applicant's RAI response adequately addressed RAI 103, Question 02.04.05-2 because the COL applicant used an updated method that is currently accepted in standard engineering practice. Accordingly, the staff considers RAI 103, Question 02.04.05-2 resolved. RAI 103, Question 02.04.05-4 and RAI 289, Question 02.04.05-7 are addressed in Section 2.4.5.4.2 of this report.

In RAI 103, Question 02.04.05-3, the staff requested that the COL applicant explain how the storm surge water surface elevation estimation procedure accounted for more recent hurricanes that have occurred in the last three decades since the publication of the Probable Maximum Hurricane estimation procedure (NOAA, 1979). The COL applicant stated that NWS 23 us based on an historical period including severe hurricanes and concluded that the PMH parameters are applicable. The COL applicant referred to the period of 1945 to 1970 as being as active in terms of hurricane severity as the more recent period. The staff agreed with the COL applicant that, because the NSW 23 incorporated a period of severe hurricane activity, the COL applicant's use of NWS 23 was adequate. Therefore, the staff considers RAI 103, Question 02.04.05-3 resolved.

2.4.5.4.2 Surge and Seiche Water-Surface Elevations

Information Submitted by COL Applicant

The COL applicant reported in the COL FSAR that 281 hurricanes have hit the East Coast of the United States between 1851 and 2005, and 12 hurricanes of Category I or greater on the Saffir-Simpson scale have passed through Maryland and Virginia. The COL applicant stated that two hurricane paths characterize the surge within the Chesapeake Bay. The COL applicant stated that hurricanes moving from the south to the north in the open ocean cause an increase in water-surface elevation at the mouth of the Chesapeake Bay, and, at the same time, the northerly winds of the hurricane cause a setdown in the northern part of the Chesapeake Bay. The COL applicant described a second scenario in which hurricanes move from the southeast

along the western edge of the Chesapeake Bay. The hurricane winds in the second scenario cause a setdown in the lower Chesapeake Bay and a setup in the upper Chesapeake Bay. The COL applicant stated that the setup in the upper Chesapeake Bay is a result of the combination of the primary storm surge propagating north in the Chesapeake Bay and a wind setup caused by the crosswinds in the hurricane.

The COL applicant reported the five highest water-surface elevations at the Baltimore and the Annapolis tide gauges and noted that, except for the August 1955 Hurricane Connie, the highest water-surface elevations at the two locations were generated by hurricanes passing by the west side of the Chesapeake Bay. The COL applicant reported the maximum water-surface elevation at Baltimore as 2.26 m (7.4 ft) and at Annapolis as 1.93 m (6.3 ft).

The COL applicant stated that RG 1.59 recommends using the 10 percent exceedance high spring tide including initial rise as the PMSS antecedent water-surface elevation. The COL applicant also stated that RG 1.59 indicates that a separate initial rise is not necessary for locations where the 10 percent exceedance high spring tide is estimated from tide gauge observations. The COL applicant reported the 10 percent exceedance high spring tide at the CCNPP Unit 3 site as 0.62 m (2.05 ft) mean low water or 0.47 m (1.53 ft) MSL estimated according to the procedure described in ANSI/ANS-2.8-1992 (ANS 1992). The COL applicant stated that the initial rise or sea-level anomaly is 0.34 m (1.1 ft) at Sewells Point (ANS 1992). The COL applicant adopted this value of initial rise for the CCNPP Unit 3 site. The COL applicant estimated a long-term sea-level rise of 3.26 mm/yr (1.07 ft per century) at the CCNPP Unit 3 site as the average of the long-term rates of sea-level rise at the Baltimore, MD, and the Solomons Island, MD, NOAA tide gauges. The MSL at the Cove Point, MD, NOAA tide gauge is 0.2 m (0.64 ft) above NGVD29. Therefore, the COL applicant estimated the antecedent water-surface elevation to be 1.34 m (4.4 ft) from the combination of 10 percent exceedance high spring tide, initial rise, and long-term sea-level rise.

The COL applicant estimated PMSS at the CCNPP Unit 3 site based on an empirical approach used by USACE (1959) for the August 1933 Chesapeake-Potomac hurricane. The COL applicant used RG 1.59 to estimate the PMSS just outside the entrance to the Chesapeake Bay due to a PMH. The COL applicant used the relationships in USACE (1959) to estimate the primary storm surge at the CCNPP site; wind setup, 10 percent high spring tide, initial rise, and long-term sea-level rise were also added to the storm surge estimate to obtain the PMSS. The COL applicant reported that the PMSS water-surface elevation would be 5.29 m (17.4 ft).

In RAI 103, Question 02.04.05-2, the staff requested that the COL applicant explain how the storm surge water-surface elevations obtained using RG 1.59 and adjusted for the CCNPP Unit 3 site using the model developed for the Chesapeake Bay (USACE 1959) are conservative with respect to current engineering practice described in USACE Engineering Manual 1110-2-1100 (USACE 2002) and those of the NOAA's NWS with regard to the SLOSH model (Jelesnianski et al. 1992), or justify an alternative approach. The staff also requested that the COL applicant provide additional justification for the method used to determine PMSS, demonstrating appropriate conservatism.

In RAI 249, Question 02.04.05-6, the staff requested that the COL applicant provide additional justification to the method used to determine PMSS demonstrating technical validity and appropriate conservatism or provide the following information: (1) An analysis of the PMSS event using a conservative approach such as those predicted by a storm surge model (e.g., SLOSH) with input from appropriate scenarios; (2) reasons why the use of historical

estimations of sea level rise is more conservative than current climatic predictions; and (3) if factored into the PMSS analysis, provide a detailed description of the process for determining uncertainty estimations.

In a June 30, 2010, response to RAI 103, Question 02.04.05-2, and a July 12, 2010, response to RAI 249, Question 02.04.05-6, the COL applicant used NOAA's SLOSH model Version 3.94 to directly simulate the PMSS within the Chesapeake Bay during a PMH event. The COL applicant stated that the CCNPP Unit 3 site is located within the SLOSH model grid at location (31, 59). The COL applicant stated that the SLOSH model simulations have been validated against observed hurricane surge elevations at several locations with a mean error of -0.09 m (-0.3 ft) and an error range of -2.16 m (-7.1 ft) to 2.68 m (8.8 ft). The COL applicant also reported that NOAA considers the SLOSH results to be within 20 percent of the actual surge for significant storms. The COL applicant stated, as an example, that if the SLOSH model predicts a peak storm surge of 3 m (10 ft), the actual observed peak could range from 2.4 to 3.7 m (8 to 12 ft).

The COL applicant used the results of SLOSH sensitivity runs were used to evaluate the effects of forward speed, size, direction, and track distances from the CCNPP Unit 3 site and the Chesapeake Bay entrance on the predicted storm surge at the site. The COL applicant used two forward speeds (the lower and the upper bound), three radii to maximum winds (the mean, lower bound, and upper bound), nine directions, and 13 track distances in the sensitivity runs. The COL applicant did not list the PMH parameters used in the individual sensitivity runs. Moreover, the COL applicant did not state how many sensitivity runs were performed. The COL applicant, based on the results of the sensitivity runs, stated that the PMSS at the CCNPP Unit 3 site would be generated by a PMH that would be characterized by (1) the lower bound of forward speed, 31.5 km/hr (19.5 mph or 17 knots); (2) the upper bound of size, radius of maximum winds of 48.2 km (29.9 mi or 26 nautical mi); (3) approaching the Atlantic Ocean shoreline of Virginia directly from the east (approach direction of 270° clockwise from the north[3]) with the eye located approximately 1.5 times the upper bound of radius to maximum winds south of the Chesapeake Bay entrance; and (4) passing the CCNPP Unit 3 site moving north on the west of the site approximately 0.25 times the upper bound of radius to maximum winds.

The COL applicant reported that the NWS 23 methodology (NOAA 1979) was used to estimate the wind field at the CCNPP Unit 3 site, which accounts for reduction in wind speeds after landfall. The COL applicant reported that the maximum 10-m, 10-min wind speed at the point of landfall would be 245.6 km/hr (152.6 mph). The COL applicant reported that the 10-min maximum wind speed at the site would be 179.8 km/hr (111.7 mph), 8 hours after landfall.

The COL applicant stated that the SLOSH model was initialized with the water-surface elevation at 0.0 m (0.0 ft) and the 10 percent exceedance high spring tide, initial rise, and long-term sea-level rise were added afterward to the SLOSH model predictions. The COL applicant reported that the SLOSH model predicted a maximum surge elevation of 3.35 m (11.0 ft) at the CCNPP Unit 3 site. The COL applicant added the 20 percent margin due to SLOSH model

[3] The COL applicant's convention of specifying PMH approach direction is based on a direction arrow pointing along the path in which the hurricane moves with the angle to the arrow's head measured clockwise from north. The NWS 23 (NOAA 1979) convention is to just specify from which direction the hurricane approaches. Therefore, the COL applicant's approach direction of 270° clockwise from north and the NWS 23 (NOAA 1979) direction of 90° clockwise from north both specify a hurricane moving from due east to due west.

prediction uncertainty, and the antecedent water-surface elevation of 1.34 m (4.4 ft) to arrive at the final PMSS elevation of 5.35 m (17.6 ft).

The Staff's Technical Evaluation

The staff reviewed the COL applicant'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 concluded that both hurricanes and northeasters could produce storm surges at the CCNPP Unit 3 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 concluded that consideration of PMH is the more conservative phenomenon to consider in a determination of the PMSS near the CCNPP Unit 3 site. Based on this review, the staff agreed with the COL applicant that both a setup and a setdown are possible at the CCNPP Unit 3 site, depending on the track direction and relative location of the hurricane. The staff also agreed that a hurricane moving north along the west edge of the Chesapeake Bay would produce the largest positive surge near the CCNPP Unit 3 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. Also, the staff determined that the CCNPP Unit 3 site could experience a water-level setdown if a hurricane were to pass moving north on the east side of the Chesapeake Bay. Section 2.4.11 of this report describes the staff's review of water-elevation setdown from PMH.

Although the COL applicant did not state the range of direction of approach of a PMH in the COL FSAR, the staff determined from NWS 23 (NOAA 1979) that the direction of approach for a PMH near the CCNPP Unit 3 site could range from 69° to 153° clockwise from north (see Table 2.4.5-2 above). The staff reviewed the COL applicant's chosen PMH track with respect to the range of direction of approach given in NWS 23 (NOAA 1979) and noted that the COL applicant's chosen PMH track conforms to existing guidance. The COL applicant also stated that the PMSS at the CCNPP Unit 3 site would be generated by a PMH that would be characterized by (1) a forward speed of 31.5 km/hr (19.5 mph or 17 knots), (2) a radius of maximum winds of 48.2 km (29.9 mi or 26 nautical mi), (3) a direction of approach from the east (90° clockwise from the north) with the eye located approximately 1.5 times the upper bound of radius to maximum winds south of the Chesapeake Bay entrance, and (4) its passage by CCNPP Unit 3 site moving north on the west of the site approximately 0.25 times the upper bound of radius to maximum winds. Based on the staff's independently estimated PMH characteristics given in Table 2.4.5-2 above, and based on the reasons given above, the staff finds that the COL applicant's selected PMH characteristics follow NWS 23 (NOAA 1979) guidance. The COL applicant provided sufficient details in the proposed COL FSAR text (UniStar June 30, 2010, letter) to show that the chosen PMH track and associated PMH parameters would produce the largest storm surge at the CCNPP Unit 3 site. Therefore, in RAI 103, Question 02.04.05-4, the staff requested that the COL applicant provide alternate locations of the PMH eye to demonstrate that the chosen location would maximize the overwater fetch and, therefore, result in the most severe plausible storm surge.

The staff reviewed the COL applicant's response to RAI 103, Question 02.04.05-4 and determined that more details regarding the various PMH tracks and their associated PMH parameters simulated by the applicants were needed. Therefore, in RAI 289,

Question 02.04.05-7, the staff requested that the COL applicant provide a table of PMH tracks and associated parameters that were used in the COL applicant's SLOSH simulations and that a description of the simulated PMSS characteristics and water surface elevations for these simulations near the PWIS. The staff further requested that the PMSS surface water elevation estimates directly account for the antecedent surface water elevation in the SLOSH simulation.

In RAI 289, Question 02.04.05-7, the staff requested that the COL applicant provide details regarding the various PMH tracks and their associated PMH parameters, elaborate on the simulated PMSS characteristics and water-surface elevations near CCNPP Unit 3 MWIS, describe how the antecedent water-surface elevations affect the SLOSH simulations, and provide an updated estimate of wind-wave effects. In an April 18, 2011, response to RAI 289, Question 02.04.05-7, the COL applicant described the use of NWS 23 to determine the PMH parameters. The COL applicant stated that the antecedent water-surface elevations used for the SLOSH simulations included 10 percent exceedance high tide, sea-level anomaly, and long-term sea-level rise, resulting in a total of 1.3 m (4.3 ft). The COL applicant described the SLOSH results for a case where the initial water-surface elevation was included as SLOSH input and for a case where the initial water-surface elevation was added to the SLOSH results afterward. The COL applicant demonstrated that the latter procedure resulted in a more conservative (i.e., higher) PMSS than in the former case. The COL applicant applied the methods described in the *Shore Protection Manual* (USACE 1984) to estimate for wind setup and wave runup. The COL applicant determined that the PMSSs, including wind-wave effects, were more conservatively estimated than the PMSSs derived from adding the antecedent water-surface elevation to the SLOSH results than derived from incorporating the antecedent water-surface elevations as a SLOSH input. The COL applicant concluded that a revision to the COL FSAR was not warranted based on the more conservative results presented in COL FSAR, Revision 7. The staff finds the COL applicant's April 18, 2011, response to RAI 289, Question 02.04.05-7, as it relates to PMH track selection, adequate based on the methods used to incorporate antecedent water-surface elevations to the determination of the PMSS. Accordingly, the staff considers RAI 289, Question 02.04.05-7 resolved.

In RAI 249, Question 02.04.05-6, the staff requested that the COL applicant provide an analysis of the PMSS event using a conservative approach such as that predicted by a storm surge model (e.g., SLOSH) with input from appropriate PMH scenarios. In a July 12, 2010, response to RAI 249, Question 02.04.05-6, the COL applicant discussed the good agreement between the COL applicant's SLOSH results and empirical data and referred to the COL FSAR updates described in the response to RAI 103, Question 02.04.05-2. The COL applicant concluded that a revision to the COL FSAR was not warranted. The staff reviewed the July 12, 2010, response to RA 249, Question 02.04.05-6 and finds the COL applicant's response acceptable. Accordingly, the staff considers RAI 249, Question 02.04.05-6 resolved.

2.4.5.4.3 Wave Action

Information Submitted by COL Applicant

The COL applicant stated that CCNPP Unit 3 is located at an elevation of 25.8 m (84.6 ft), approximately 305 m (1,000 ft) from the shoreline. The COL applicant stated that the only safety-related SSCs that would be affected by the PMSS including wind waves are the forebay, the UHS MWIS, and the UHS Electrical Building. The COL applicant noted that the hurricane wind direction would change from southwestward to southeastward as the PMH passes the site

to the west. The COL applicant reported that the maximum wind speed at the site during a PMH would be 179.8 km/hr (111.7 mph).

The COL applicant stated that the SLOSH simulations that produce the highest sustained wind speed during a PMH would come from the east or southeast when the eye of the hurricane is located a distance equal to the radius to maximum winds southwest of the CCNPP Unit 3 site. The COL applicant also stated that the growth of the wind-induced waves at the CCNPP Unit 3 site would be limited by the short duration of the time the peak winds from the PMH would be active at the site. The COL applicant estimated the characteristics of wind-induced waves at the CCNPP Unit 3 site using the procedures in the *Coastal Engineering Manual* (USACE 2008). The COL applicant reported a significant wave height of 3.31 m (10.8 ft) and a one percent wave height as 5.52 m (18.1 ft).

The COL applicant stated that the grade elevation around the UHS MWIS is 3.05 m (10 ft) and, therefore, a storm surge of 5.35 m (17.6 ft) would inundate the grade around the MWIS by 2.3 m (7.6 ft). The COL applicant estimated that the maximum sustainable unbroken wave height at the MWIS would be 4.76 m (15.6 ft). The COL applicant reported, therefore, that the PMSS runup would reach an elevation of 10.1 m (33.2 ft). The COL applicant reported that waves that travel past the MWIS would break on a 3H:1V slope farther inland. The COL applicant estimated the runup for waves traveling past the MWIS to be 4.96 m (16.3 ft) and, therefore, reported that the PMSS runup for these waves could reach elevation 10.3 m (33.9 ft).

The COL applicant stated that, because the grade elevation of CCNPP Unit 3 nuclear power block is at 26 m (85 ft) and the nuclear power block is located about 305 m (1,000 ft) from the shoreline, the nuclear power block would not be affected by the PMSS runup. The COL applicant stated that the safety-related UHS MWIS and Electrical Building would be flooded during the PMSS, and the UHS MWIS would be overtopped by the waves. The COL applicant stated that these structures are designed to conform to the guidance of RG 1.27. The COL applicant also stated that the accesses into the UHS MWIS and the UHS electrical building that would be below the maximum PMSS water-surface elevation would be designed to be watertight to prevent flooding of the facilities.

The Staff's Technical Evaluation

The staff agreed with the COL applicant's use of the *Coastal Engineering Manual* (USACE 2008) as the source of applicable methodology for the performing relevant calculations.

The staff finds the COL applicant's April 18, 2011, response to RAI 289, Question 02.04.05-7 adequate as it relates to wind-wave effects. The staff finds the COL applicant's determination that the design-basis flood elevation is related to the PMH, including wind-wave effects, adequate. The design-basis flood water-surface elevation is 10.11 m (33.2 ft) as stated in COL FSAR, Revision 9. Therefore, the staff closed RAI 289, Question 02.04.05-7 and RAI 103, Questions 02.04.05-1 through 02.04.05-04.

2.4.5.4.4 Resonance

Information Submitted by COL Applicant

The COL applicant stated that no significant oscillations within the Chesapeake Bay have been observed in the storm surge records. The COL applicant indicated that during the 2003 Hurricane Isabel, water-surface elevations at several locations gradually rose and fell without oscillations. The COL applicant stated that sustained wind speed along the north-south axis of the Chesapeake Bay may cause a seiche. The COL applicant stated that the period of these oscillations is reported to be 2 to 3 days. The COL applicant 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. The COL applicant concluded, therefore, that resonance (seiche oscillations) in conjunction with the PMSS would not occur.

The Staff's Technical Evaluation

In RAI 103, Question 02.04.05-5, the staff requested that the COL applicant provide a reference for, and a summary of, the method used to estimate the period of oscillation of wind-induced seiches in the Chesapeake Bay. In a May 20, 2009, response to RAI 103, Question 02.04.05-5, the COL applicant provided a reference and details regarding the methodology. The Chesapeake Bay is approximately 280 km (175 mi) long and seiche motion in the Chesapeake Bay is a common occurrence. The natural period of the Chesapeake Bay is about 2 days and the longitudinal (north-south) and the lateral (east-west) winds are capable of setting up seiches in the Chesapeake Bay. The natural period for transverse oscillations in the bay may be shorter, about 1.6 days, and one such observed event resulted in water-surface-elevation oscillations of about 0.5 m (1.6 ft).

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. However, the wind would have to match the natural period of the Chesapeake Bay, which is about 1.6 days, to result in resonant seiche oscillations. The COL applicant-proposed PMH track, with the staff-determined forward speed of 26.3 to 68 km/hr (16.4 to 42.5 mph) and radius to maximum winds of 17.8 to 47.3 km (11.1 to 29.6 mi) would result in a reversal in lateral wind direction in less than an hour. Therefore, the staff concluded that lateral winds reversing at the natural period of oscillation in the Chesapeake Bay would not occur during PMH conditions. Also, because the PMH eye is expected to skim along the west edge of the bay, a reversal of north-south wind in the bay would not occur. Due to a lack of resonant forcing (a wind reversal period that matches the natural period of oscillation in the bay), the staff concluded that a resonance during a PMH in the Chesapeake Bay would not occur. Based on the above discussion, the staff finds the COL applicant's May 20, 2009, response to RAI 103, Question 02.04.05-5 acceptable. Accordingly, the staff considers RAI 103, Question 02.04.05-5 resolved.

2.4.5.4.5 Protective Structures

Information Submitted by COL Applicant

The COL applicant stated that the shoreline near the UHS MWIS would be protected against the PMSS and coincident wind waves. The COL applicant stated that flood-protection measures are described in COL FSAR Section 2.4.10.

The Staff's Technical Evaluation

In RAI 328, Question 02.04.10-1, the staff requested more information about storm surge flood protection for the UHS intake, intake pumphouse, and associated pipes, which established a maximum permissible velocity necessary to ensure drainage channel stability. The COL applicant described the flooding of the UHS MWIS as a result of the PMSS. The COL applicant stated the flood protection measures at the UHS MWIS and its interface with the shore. The COL applicant stated that these protection measures afford full protection to the buried pipes associated with the UHS system. Therefore, the staff considers RAI 328, Question 02.04.10-1, resolved, because the COL applicant has provided sufficient information to specify site characteristics needed to ensure appropriate design inputs to the UHS MWIS and buried safety-related pipes.

The UHS MWIS is a Seismic Category I structure designed to be protected from air-borne missiles, including automobiles, generated by tornadoes and extreme winds. The associated tornado wind speeds (a maximum of 103 m/s (230 mph)) are used in the standard design of all Seismic Category I structures (COL FSAR Sections 3.5 and 3.8). Other air-borne missile sources (except aircraft) are discussed in COL FSAR Section 2.2. The basic conditions for a design-basis accident (DBA) involving hydrostatic, hydrodynamic, debris load, and water-borne missiles are associated with a PMSS possessing a PMH wind speed of 50 m/s (111.7 mph). The results of an independent staff evaluation indicate water velocities and wave speeds no greater than 15 m/s (49.2 mph) and 10 m/s (32.8 mph), respectively. Thus, protection from water-borne missiles is conservatively enveloped by the Seismic Category I design.

2.4.5.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.5.6 *Conclusions*

The staff reviewed the COL application and confirmed that the COL applicant has addressed the information demonstrating that the characteristics of the site fall within the site parameters specified in the U.S. EPR FSAR, and that no outstanding information is required to be addressed in the COL FSAR related to this section.

As discussed above, the COL applicant presented and substantiated information to establish the site description. The staff reviewed the COL applicant's information and, for the reasons stated above, concluded that, as documented in Section 2.4.5 of this report, the COL applicant has provided sufficient detail about the site description to allow the staff to evaluate whether the COL applicant has met the relevant requirements of 10 CFR 52.79(a)(1) and 10 CFR Part 100, with respect to determining the acceptability of the site.

2.4.6 Probable Maximum Tsunami Hazards

2.4.6.1 *Introduction*

This section of the COL FSAR addresses the hydrological design basis developed to ensure that any potential tsunami hazards to the SSCs important to safety are considered in plant design.

This section presents the staff's review of the flood levels caused by postulated tsunami wave-forming scenarios. The specific areas of the review include the description of the PMT, historical tsunami records, source generator characteristics, tsunami analyses, tsunami water levels, hydrograph and harbor or breakwater influences of a tsunami-like wave, and its effects on safety-related facilities.

2.4.6.2 *Summary of Application*

COL FSAR Section 2.4.6 incorporates by reference U.S. EPR FSAR Tier 2, Section 2.4.6, "Probable Maximum Tsunami Flooding." In COL FSAR Section 2.4.6, the COL applicant provided site-specific information about potential tsunami effects at the site.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-6

> A COL applicant that references the U.S. EPR design certification will provide site-specific information and determine the extent to which safety-related facilities require protection from tsunami effects, including Probable Maximum Tsunami Flooding.

The COL applicant provided additional information in COL FSAR Section 2.4.6 to address COL Information Item 2.4-6, "Probable Maximum Tsunami Hazards," from U.S. EPR FSAR Tier 2, Table 1.8-2. The COL applicant evaluated three different tsunami sources for the definition of a PMT and also reviewed three primary sources of information to establish the historical record of teletsunamis affecting the U.S. Atlantic coast. A teletsunami (also called an ocean-wide tsunami, distant tsunami, distant-source tsunami, far-field tsunami, or trans-ocean tsunami) is a tsunami that originates from a distant source, defined as more than 1,000 km (620 mi) away or three hours of travel time from the area of interest, sometimes travelling across an ocean (Intergovernmental Oceanographic Commission, 2013).

Additionally, the COL applicant examined published information to determine the source generator characteristics for three different teletsunami source scenarios.

Water levels at the CCNPP Unit 3 site attributed to the formation of a tsunami wave were determined by the COL applicant from tsunami propagation models of the Chesapeake Bay based on the shallow water wave equation as described in COL FSAR Section 2.4.6.4. The COL applicant used both the nonlinear propagation model with the bottom friction (NLSWE) and the linear tsunami propagation model without bottom friction (TSU).

2.4.6.3 Regulatory Basis

The relevant requirements of NRC regulations for the consideration of probable maximum tsunami hazards, design considerations, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.6.

The applicable regulatory requirements for identifying PMT hazards are as follows:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been

historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water levels at the site.

The related acceptance criteria are as follows:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding or those associated with the MWIS
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site

The regulatory basis of the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.6.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.6. The staff confirmed that the information in the COL application addresses the relevant information related to the PMT. The staff's technical review of this section includes an independent review of the COL applicant's information in the COL FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information in COL FSAR Section 2.4.6.

2.4.6.4.1 Probable Maximum Tsunami

Information Submitted by COL Applicant

The COL applicant evaluated three different teletsunami sources for the establishment of the PMT. The COL applicant first obtained estimates of maximum tsunami amplitudes and dominant periods offshore the entrance of Chesapeake Bay for each source and then performed hydrodynamic simulations within Chesapeake Bay itself to determine the tsunami water levels from these sources. For the Norfolk Canyon submarine landslide source, the COL applicant indicated that the maximum offshore amplitude is 2-4 m (7-13 ft). For the La Palma volcanic flank failure distal source, the COL applicant indicated that the maximum offshore

amplitude is less than 3 m (9.8 ft). For the distal Greater Antilles submarine earthquake source, the COL applicant indicated that the maximum offshore amplitude is about 1 m (3.3 ft). The COL applicant indicated that maximum computed amplitude and drawdown at the CCNPP Unit 3 site is 0.5 m (1.6 ft) above antecedent water level and 0.5 m (1.6 ft) below antecedent water level, respectively. These limiting values are from the Norfolk Canyon submarine landslide tsunami source using the TSU model.

The COL applicant did not compute tsunami water levels for earthquakes along the Grand Banks offshore region of Canada and along the South Sandwich subduction zone in the southern Atlantic Ocean because of their historically small intensities. The COL applicant also indicated that slope failures along the coastal cliffs in the Chesapeake Bay near the CCNPP Unit 3 site, have not resulted in tsunami-like waves; the eastern shore location immediately opposite the CCNPP Unit 3 site is not subject to slope failure. Therefore, the COL applicant concludes that local subaerial landslides will not trigger local tsunami-like waves.

The Staff's Technical Evaluation

The staff conducted an independent confirmatory analysis to determine the PMT at the CCNPP Unit 3 site that is described in detail in the sections that follow. The staff considers both far-field seismogenic (Puerto Rico subduction zone (see Figure 2.4.6-1 below)) and far-field (Canary Islands (see Figure 2.4.6-2 of this report)) and near-field (Currituck (see Figure 2.4.6-3 of this report)) landslide sources as potential generators for the PMT. Initial analysis indicates 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.

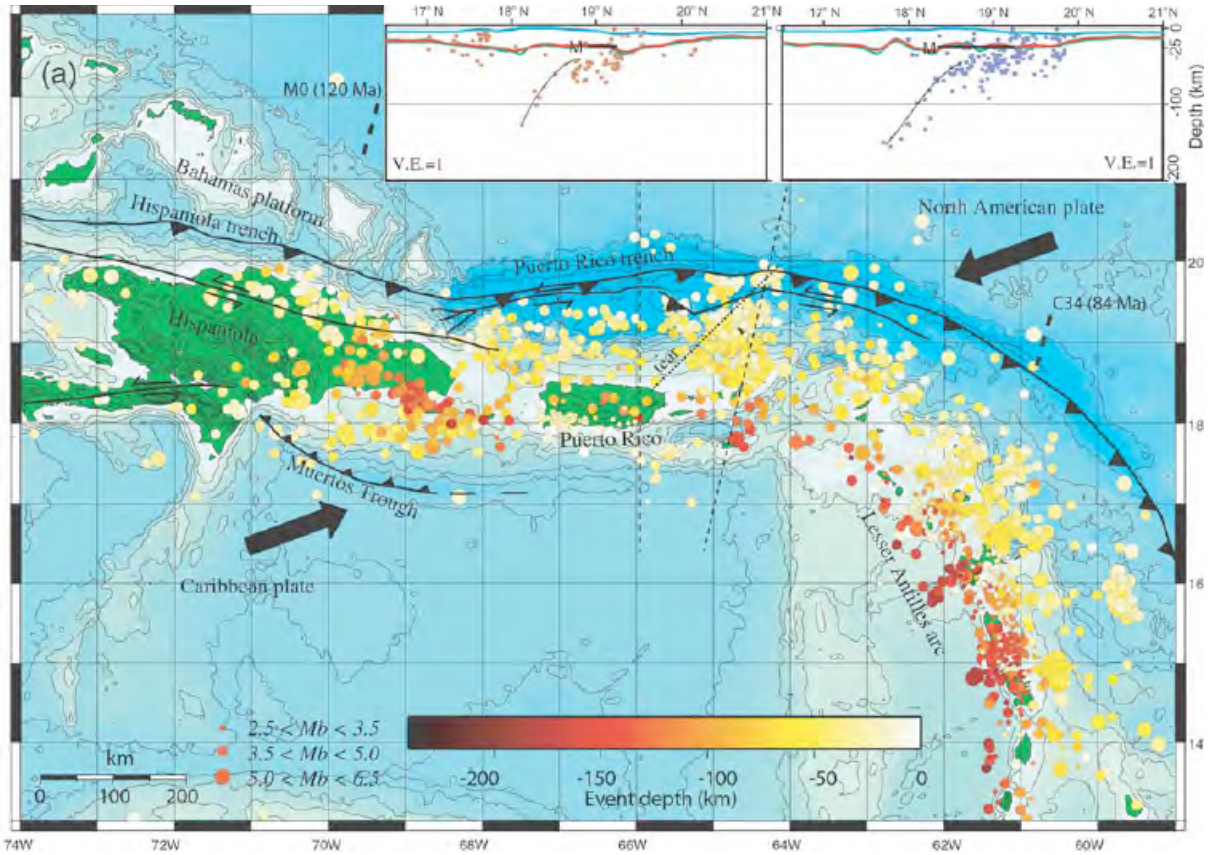

Figure 2.4.6-1. Major faults in the Greater Antilles region.

Subduction zone fault is represented by line with barbed pattern. Insets show the subduction of the North American plate beneath the Caribbean plate along two different transects. Large arrows show the direction of relative convergence between the two plates. North latitudes are shown. Figure from ten Brink (2005).

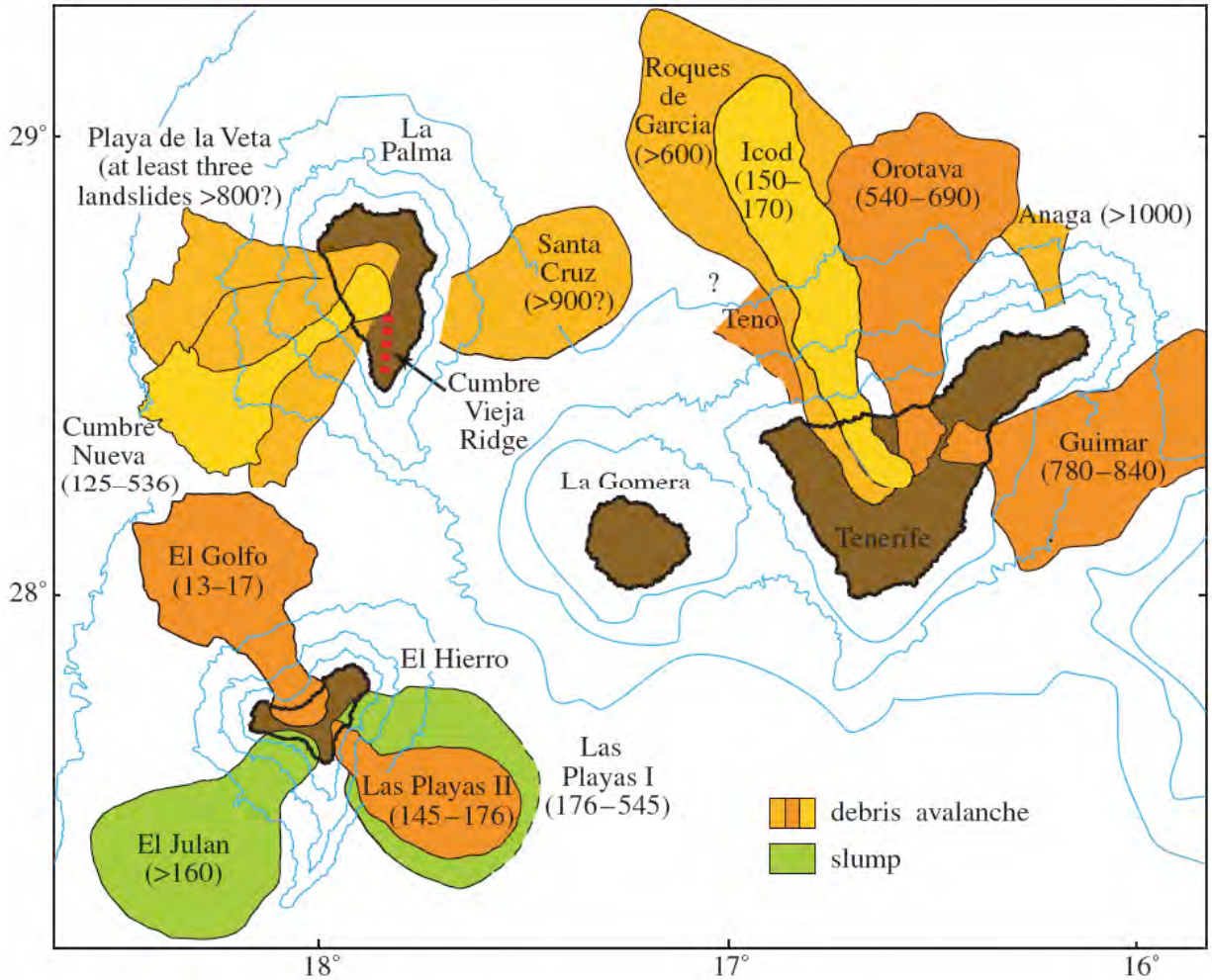

Figure 2.4.6-2. Location and ages (in thousands of ybp) of landslides in the Canary Islands. Figure from Masson et al. (2006). North latitudes and west longitudes are shown. Bathymetric contour interval is 1 km.

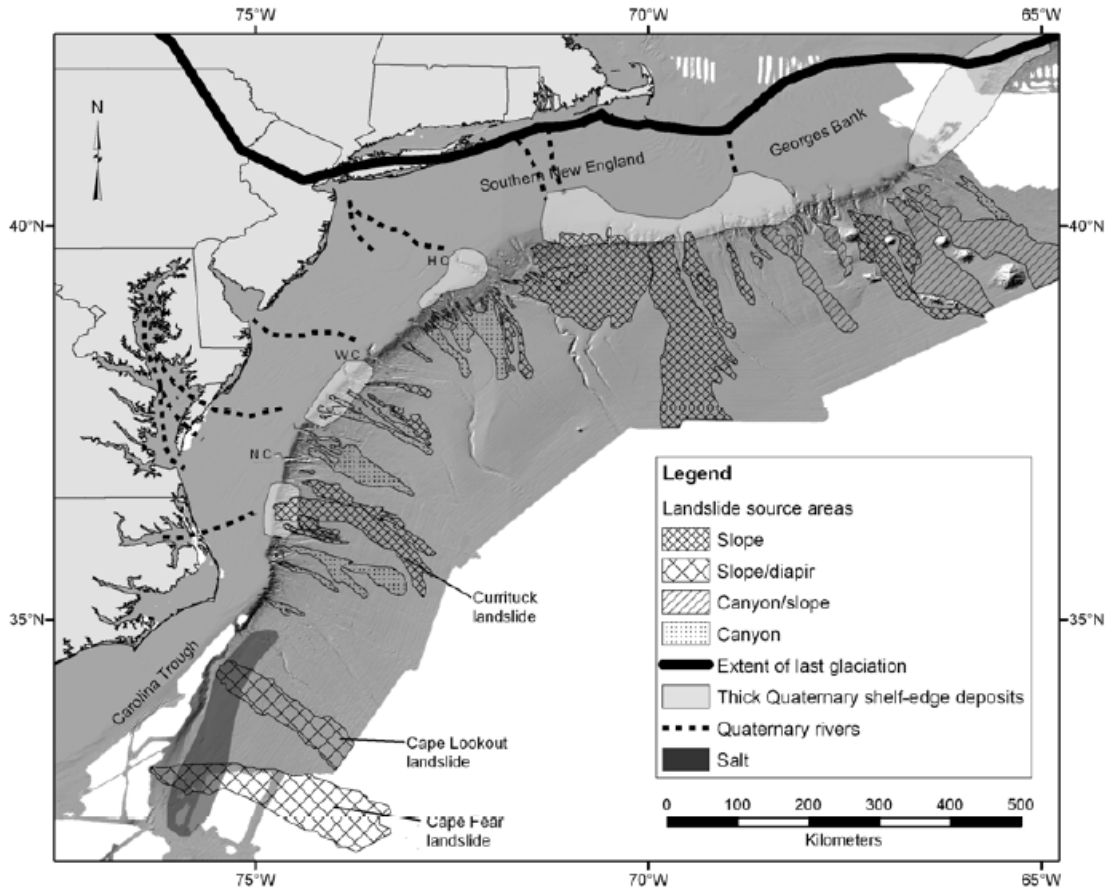

Figure 2.4.6-3: Observed landslides offshore NE Atlantic coast (Twichell and others, 2009)

In RAI 99, Questions 02.04.06-1, 02.04.06-2, 02.04.06-3, and 02.04.06-4, the staff requested that the COL applicant provide a discussion of the generation of tsunami-like waves from hill-slope failures and earthquake-induced sources near the CCNPP Unit 3 site. In May 18, 2009, and June 3, 2009, responses to RAI 99, Questions 02.04.06-1, 02.04.06-2, 02.04.06-3, and 02.04.06-4, the COL applicant provided an explanation of the tsunami hazard from potential hill-slope failures with references to geotechnical analysis presented in COL FSAR Sections 2.5.1, "Basic Geologic and Seismic Information," and 2.5.5.2, "Geologic and Tectonic Characteristics of Site and Region." The COL applicant also provided a clear statement that no evidence of seismic seiches was found. The staff notes that the COL FSAR was updated accordingly. Accordingly, the staff considers RAI 99, Questions 02.04.06-1 through 02.04.06-4 resolved.

There are significant differences in how the PMT is determined as described in the COL applicant's August 24, 2009, and December 7, 2009, responses to RAI 99, Questions 02.04.06-10, 02.04.06-12 through 02.04.06-17 and the staff's confirmatory analysis. The differences are primarily related to conservatism applied in the tsunami analysis. The COL applicant applied a high level of conservatism for tsunami propagation in the Chesapeake Bay by using a linearized version of the shallow water equations without bottom friction and by including a runup factor in addition to the maximum tsunami amplitude (and antecedent water-level conditions such as sea-level rise). However, the hydrodynamics physically governing the

propagation and runup of tsunami waves have less attendant uncertainty than the characteristics of tsunami sources that govern maximum initial displacement of the water surface. In both cases, the source for the PMT maximum water levels is from a near-field landslide. In the independent confirmatory analysis, a conservative landslide source was used, producing maximum tsunami amplitude at the entrance of the Chesapeake Bay much greater than that used by the COL applicant from previously published analyses. However, the difference between the COL applicant's and staff's PMT water levels as the tsunami propagates through Chesapeake Bay to the site are minor, even though no bottom friction was considered for submerged regions in either analysis (see Section 2.4.6.4.5 of this report). The COL applicant's PMT maximum water-level estimate at the site of 3.5 m (11.5 ft) is slightly greater than the staff's PMT water-level estimate of 2.8 m (9.2 ft), taking into account antecedent water levels. Moreover, the PMT water-level estimates by both the COL applicant and the staff are below the design-basis flood elevation of 6.6 m (21.7 ft) the COL applicant determined from PMH storm surge. Accordingly, the staff considers RAI 99, Questions 02.04.06-10; and 02.04.06-12 through 02.04.06-17 resolved.

2.4.6.4.2 Historical Tsunami Record

Information Submitted by COL Applicant

The COL applicant reviewed three primary sources of information to establish the historical record of tsunamis affecting the U.S. Atlantic coast. They included: (1) The NOAA/National Geophysical Data Center (NGDC), "Historical Tsunami Database" (on the internet[4]); (2) the Maine Geological Survey's "Tsunamis in the Atlantic Ocean" internet website[5]; and (3) various technical articles published in referred journals.

Five potential teletsunami source regions (scenarios) are identified by the COL applicant from this information: (1) Submarine landslide along the U.S. Atlantic continental slope; (2) tsunamigenic sources in the eastern Atlantic Ocean, including submarine earthquakes near Portugal and volcanogenic sources in the Canary Islands; (3) plate-boundary earthquakes in the Caribbean; (4) submarine earthquakes in the northern Atlantic Ocean offshore Newfoundland, Canada; and (5) subduction zone earthquakes along the South Sandwich Islands in the southern Atlantic.

The COL applicant then describes three historic tsunamis from three source regions − eastern Atlantic Ocean, Caribbean plate-boundary earthquakes, and northern Atlantic Ocean submarine earthquakes − for which there are measured or computed tsunami amplitudes along the U.S. East Coast.

The COL applicant described three historical tsunamis with measured or computed tsunami amplitudes and runups. Based on published numerical computations (Mader 2001b), the 1755 Lisbon seismogenic tsunami associated with the eastern Atlantic source region is estimated to have had 3 m (9.9 ft) maximum amplitude along the U.S. East Coast. The 1918 Puerto Rico earthquake in the Caribbean source region resulted in a 0.06 m (0.2 ft) tsunami amplitude measurement at Atlantic City, NJ. The 1929 Grand Banks earthquake and associated landslide

[4] http://www.ngdc.noaa.gov/hazard/tsu_db.shtml.
[5] http://maine.gov/doc/nrimc/mgs/explore/hazards/tsunami/jan05.htm.

from the northern Atlantic source region resulted in a 0.7 m (2.3 ft) tsunami amplitude measurement at Atlantic City, NJ.

The COL applicant notes that although there is geologic evidence of a bolide impact in the Chesapeake Bay and possible tsunami (Poag and others, 1992), no paleo-tsunami stratigraphic evidence of a tsunami caused by that event has been identified in the CCNPP Unit 3 site region.

The Staff's Technical Evaluation

The COL applicant summarized the essential historical record of tsunamis in the region. In RAI 99, Question 02.04.06-5, the staff requested that the COL applicant provide specific guidance with respect to the historical tsunami record, including paleo-tsunami evidence. The staff also requested that the COL applicant provide a discussion in the updated COL FSAR of the literature search conducted that was used to conclude the non-existence of tsunami deposits preserved in the vicinity of the CCNPP Unit 3 site. In a June 3, 2009, response to RAI 99, Question 02.04.06-5, the COL applicant stated that it found no evidence of a paleo-tsunami in the region.

With regard to the1755 Lisbon tsunami, the staff noted that the 3 m (9.9 ft) tsunami amplitude estimated at the U.S. East Coast was derived from a numerical model. The source parameters for the earthquake that generated this tsunami are highly uncertain and geologically complex (e.g., possibly compound earthquake rupture). Barkan and others (2009) indicate that tsunami amplitudes from this earthquake are likely to be less than 1 m (3 ft) and demonstrate the effect that source parameter uncertainty has on computed tsunami amplitudes at the U.S. East Coast.

With regard to the 1918 Puerto Rico tsunami, the source is most likely a landslide triggered by an intra-arc earthquake (López and others, 2007). It is not necessarily representative of a Greater Antilles subduction zone earthquake.

A more thorough record of tsunamis and tsunami-like waves along the Atlantic seaboard of the U.S. than described in the COL FSAR is given by Lockridge and others (2002). The most relevant tsunami amplitude measurement near the Chesapeake Bay entrance appears to be the 0.68 m (2.2 ft) reading at Atlantic City, NJ, that was attributed to the 1929 Grand Banks tsunami (as mentioned by the COL applicant in the COL FSAR). The source of the April 19, 1964, waves recorded from New Jersey to Rhode Island is unknown; Lockridge and others (2002) have suggested that it was possibly from a submarine landslide. The December 26, 2004, readings are from the Indian Ocean tsunami that propagated into the Atlantic Ocean (Titov and others, 2005; Rabinovich and others, 2006). The October 28, 2008, records are likely from large, meteorologically-induced waves rather than from some tsunami (cf., Monserrat and others, 2006). Based on the reasons given above, the staff finds that the COL applicant has indicated the most relevant historical tsunamis in the COL FSAR. Accordingly, the staff considers RAI 99, Questions 02.04.06-5 and 02.04.06-6 resolved.

2.4.6.4.3 Source Generator Characteristics

Information Submitted by COL Applicant

The COL applicant examined published information to determine the source generator characteristics for three different teletsunami source scenarios: Norfolk Canyon submarine landslide; Canary Islands volcanic flank failure; and the Greater Antilles plate-boundary

earthquake. For each source, the COL applicant described the types of tsunami-generating mechanisms and relevant tsunami generation parameters. Then, the COL applicant estimated the maximum tsunami amplitude and dominant period offshore the Chesapeake Bay entrance for each source.

The majority of the COL applicant's source material was obtained from published journal literature (Norfolk Canyon and Canary Islands sources) and Brandsma, Divoky, and Hwang (1979), "Tsunami Atlas for the Coasts of the United States," (the Greater Antilles source). The COL applicant provided some discussion of the justification for the source scenarios and parameters used in the analysis is provided for the Norfolk Canyon landslide and Canary Islands flank failure scenarios.

The COL applicant determined or interpreted estimates of maximum tsunami amplitude and dominant period offshore of the Chesapeake Bay entrance from published journal articles (Norfolk Canyon and Canary Islands sources) and NRC Report NUREG/CR-1106 (Greater Antilles source). These values are used by the COL applicant as a boundary forcing function for the Chesapeake Bay propagation model described in COL FSAR Section 2.4.6.4 and are as follows:

- For the Norfolk Canyon landslide scenario, the maximum amplitude is 4 m (13.1 ft), and the dominant period is 60 min.
- For the La Palma volcanic flank failure, the maximum amplitude is 3 m (9.8 ft), and the dominant period is 60 min.
- For the Greater Antilles earthquake, the maximum amplitude is 0.9 m (3 ft), and the dominant period is 86.7 min.

The Staff's Technical Evaluation

In RAI 99, Question 02.04.06-6, the staff requested that the COL applicant provide a discussion of the literature search that was used to determine the tsunami source parameters for the Norfolk Canyon landslide scenario. In a June 15, 2009, response to RAI 99, Question 02.04.06-6, the COL applicant provided further explanation of parameters used for the Norfolk Canyon landslide scenario. The source parameters used by the COL applicant are taken from an analysis by Ward (2001). A more recent analysis (Locat and others, 2009) can be used for comparison. The total volume used by the COL applicant (i.e., Ward, 2001) is 150 km^3 (36 mi^3), compared to 165 km^3 (39.6 mi^3) by Locat and others (2009). The width used by the COL applicant is 30 km (18.6 mi) (20 km (12.4 mi) in Locat and others); downslope length is 40 km (24 9 mi) (30 km (18.6 mi) in Locat and others); initial thickness is 136 m (446 ft) (275 m (902 ft) in Locat and others). The COL applicant (based on a study by Ward, 2001) chose a constant downslope speed for the failed mass of 35 m/s (115 ft/s) for the slide front and 15 m/s (49 ft/s) for the back of the slide, with a total duration of 55 min. In comparison, Locat and others (2009) used a dynamic debris flow model to compute the landslide speeds: 32-43 m/s (105-141 ft/s) for the frontal part, and approximately 8 m/s (26.2 ft/s) for the back part. The main acceleration phases of slide deformation occur within the first 20 min of slide failure. The staff finds the COL applicant's response acceptable and, therefore, considers RAI 99, Question 02.04.06-6 resolved.

For the predicted tsunami amplitude, the COL applicant used a maximum of 4 m (13.1 ft) at the Chesapeake Bay entrance. This compares to maximum runup of 6.1-8.8 m (20-20.6 ft) at the

entrance of the Chesapeake Bay for a short duration landslide (7.2-20 min, respectively) as modeled by Geist and others (2009). The independent confirmatory analysis described in Section 2.4.6.4.5 of this report uses more conservative estimates of tsunami source parameters that are based on assuming instantaneous initial tsunami-forming conditions in the simulation, resulting in approximately 30 m (98.4 ft) wave amplitude near the Chesapeake Bay entrance.

The tsunami source parameters selected by the COL applicant for this scenario were less conservative than those used by the staff. However, differences in tsunami analysis techniques used within the Chesapeake Bay may partially offset this underestimation of initial tsunami wave heights, as described in Section 2.4.6.4.5, "Tsunami Water Levels," of this report. The staff finds the COL applicant's response acceptable, and, therefore, considers RAI 99, Question 02.04.06-6 resolved since the COL applicant has provided the necessary information regarding the Norfolk Canyon landslide scenario.

In RAI 99, Question 02.04.06-7, the staff requested that the COL applicant provide details of the literature search that was used to determine the tsunami source parameters for the La Palma landslide scenario. In a June 15, 2009, response to RAI 99, Question 02.04.06-7, the COL applicant provided further description of the source for the parameters used for the Cumbre Vieja volcano flank failure scenario. The staff finds the COL applicant's response acceptable and, therefore, considers RAI 99, Question 02.04.06-7 resolved.

In RAI 99, Question 02.04.06-8, the staff requested that the COL applicant provide a discussion of the numerical model used to determine the 0.9 m (2.95 ft) maximum amplitude at the Chesapeake Bay entrance for the Haiti (Greater Antilles) earthquake scenario. In a June 15, 2009, response to RAI 99, Question 02.04.06-8, the COL applicant provided a description of the hydrodynamic model used to determine a 0.9 m (2.95 ft) maximum amplitude at the Chesapeake Bay entrance for the Haiti earthquake scenario. The staff finds the COL applicant's response acceptable and, therefore, considers RAI 99, Question 02.04.06-8 resolved.

The staff identified four primary tsunami source regions for determining the PMT at the CCNPP site: (1) The Currituck/Norfolk region (landslide source); (2) the Canary Islands region (landslide source); (3) the Greater Antilles subduction zone (earthquake source); and (4) the Azores-Gibraltar oceanic convergence boundary (earthquake source). A brief description of the geologic/tectonic setting and tsunami source parameters are given below for each region.

Currituck/Norfolk region: The estimated maximum credible landslide event in this region is based on the past occurrence of the Currituck landslide (approximately 60 km (37.3 mi) south of Norfolk Canyon). The Currituck landslide, located southeast of the Chesapeake Bay has one of the four largest source areas for the entire margin.

The Currituck landslide is considered to have been two sub-events that occurred contemporaneously (Locat and others 2009). The total volume of the landslide is estimated to be between 128 km^3 (30.7 mi^3) (Prior and others, 1986) and 165 km^3 (39.6 mi^3) (Locat and others, 2009). The latter estimate is used as the maximum credible volume for the purposes of the tsunami simulation.

In terms of the geologic setting for the Currituck landslide, Quaternary-age delta deposits make up the bulk of the material that failed along the continental slope, but some Pliocene-age strata

may have been removed as well (Bunn and McGregor, 1980; Prior and others, 1986). There is approximately 4-9 m (13.1-29.5 ft) of sediment that has accumulated since the Currituck slope failure (Prior and others, 1986). The age of the failure has been estimated to be approximately 25,000-50,000 years before present (ybp), based on average sedimentation rates of 5 cm/year for sediment burying the scar and deposits (Prior and others, 1986; Lee, 2009).

The staff notes that the COL applicant drew on available literature for the maximum tsunami amplitude for the three source areas. Information on the Norfolk Canyon landslide was drawn from Driscoll and others (2000). A more recent publication by Hill and others (2004) suggests that the series of elongate depressions near the shelf edge may not be incipient fault scarps as originally suggested by Driscoll and others (2000), but instead may be the result of gas discharge from the seafloor.

Research conducted by the USGS (Geist and others, 2009) suggests the maximum amplitude of the tsunami generated by the Currituck landslide would have been approximately 25 m (82 ft) at the source area, but waves would have decreased in height as they traveled shoreward to 4-6 m (13.1-19.7 ft) in deep water associated with the continental shelf (22 m (72 ft)), and would have decreased to around 3 m (9.8 ft) at the shoreline.

Canary Islands Region: The maximum credible landslide event producing a tsunami is postulated to be a catastrophic flank failure of a volcano along the SW portion of La Palma Island.

The maximum estimated landslide volume is 500 km^3 (120 mi^3) (Ward and Day, 2001), though Masson and others (2006) note that this volume is 2-3 times bigger than a typical Canary Island landslide and that such landslides often fail as separate (in terms of tsunami generation) sub-events. The geologic age of these landslides ranges from 13,000-17,000 ybp for the El Golfo landslide on El Hierro Island to more than 1 million ybp. From these studies, the age of the Cumbre Nueva landslide, for which the maximum credible landslide event is based, is 125,000-536,000 years before present (ybp).

The initial research on the La Palma flank failure (Ward and Day, 2001) predicts wave heights of 10-25 m (33-82 ft) on the eastern shore of North America from a landslide whose magnitude is on the order of 500 km^3. The hydrodynamic model was used by Ward and Day (2001); however, it does not include the effects of nonlinear advection or wave breaking. More recent research that incorporates these effects suggests that wave heights along the eastern seaboard that might be attributed to this failure scenario would be less than 3 m (9.8 ft) (Mader, 2001) or less than 1 m (3.3 ft) (Gisler and others, 2006) at the entrance of Chesapeake Bay. Results of the staff analysis of this event are presented below in Section 2.4.6.4.4, “Tsunami Analysis,” of this report.

The Greater Antilles Subduction Zone: North of the Greater Antilles Islands is a major fault along which great earthquakes (>M8) can be generated. This fault represents the boundary between the North American and Caribbean tectonic plates, in which the North American plate is being subducted (pulled beneath) the Caribbean plate. The types of earthquakes that are generated along subduction zones involve thrust motion with large amounts of vertical seafloor displacement and are relatively efficient at generating tsunamis. In comparison, transform plate boundaries involve strike-slip motion and are much less efficient at generating tsunamis. Since the relative convergence direction between the two plates at the Greater Antilles subduction

zone is highly oblique to the orientation of the fault, it is possible that there may be a mixed mode of thrust and strike-slip motion for earthquakes at this subduction zone.

Due to the large surface area of these faults, the world's largest earthquakes occur on subduction zone thrusts. As explained by Geist and Parsons (2009), there are several methods to determine the maximum magnitude earthquake that can occur on subduction zones. The most conservative method is a statistical fit to the frequency-magnitude distribution of earthquakes (known as the Gutenberg-Richter distribution) that occur on all of the world's subduction zones (Bird and Kagan, 2004). Since the length of the Greater Anitlles subduction zone may limit the maximum earthquake magnitude possible, parametric and empirical methods are also considered**.**

The maximum tsunami amplitude offshore of the Chesapeake Bay entrance from a M9.1 Greater Antilles subduction earthquake is approximately 1-3 m (3.3-9.85 ft) (ten Brink, and others, 2008). The annual probability for a M9.1 earthquake is approximately 1.0-1.5 x 10^{-4} (mean return time in the range of 6,700 to 10,000 years).

The Azores-Gibraltar Oceanic Convergence Boundary: The offshore boundary between the African and Eurasian tectonic plates is classified as an oceanic convergence boundary (Bird, 2003). An M8.4 to 8.7 earthquake along this plate boundary offshore of Lisbon in 1755 generated a transoceanic tsunami that was observed in the Caribbean and Canada. The specific faults that make up this plate boundary in the Azores Gibraltar region are highly complex.

Using the statistical analysis prepared by Bird and Kagan (2004), the staff estimated the magnitude distribution of earthquakes along the world's oceanic convergence zones. Due to a much smaller sample size in comparison to subduction zones, however, there is much greater uncertainty in the distribution curves for the earthquakes. (See the 95 percent confidence interval curves (thin lines) in Geist and Parsons, 2009.) Size distributions were computed using tectonic moment rates and distribution shape parameters from Bird and Myesrn (2004). In McBird and Kagan (2004), the heavy line represents tapered Gutenberg-Richter distribution showing a likely maximum magnitude earthquake obtained from the global earthquake catalog; thin lines represent distributions for 95 percent confidence interval.

The maximum tsunami amplitude offshore of the Chesapeake Bay entrance from an M=8.4-8.7 Azores-Gibraltar earthquake is approximately 1 m (3.3 ft) (Barkan and others, 2009). The annual probability for this size earthquake is in the range of 1.0 x 10^{-6} to 2.5 x 10^{-4} events/year (assuming a 4,000 to 1,000,000 year mean return time), representing a high degree of uncertainty.

2.4.6.4.4 Tsunami Analysis

Information Submitted by COL Applicant

Tsunami water levels at the CCNPP site were determined by the COL applicant from tsunami propagation models of the Chesapeake Bay based on the shallow-water wave equation as described in COL FSAR Section 2.4.6.4. The COL applicant used both the NLSWE and the TSU model. The COL applicant used a sinusoid forcing function at the Chesapeake Bay entrance to emulate the tsunami wave history incident from the three regional and distant sources considered in COL FSAR Section 2.4.6. The COL applicant equated the amplitude of

the sinusoid to the maximum offshore tsunami amplitude and equated the period of the sinusoid to the dominant period. Both the maximum offshore tsunami amplitudes and dominant periods for the three sources were determined or interpreted by the COL applicant from previous reports in peer-reviewed journal publications.

The COL applicant derived the bathymetric grid, which is important for establishing the tsunami propagation characteristics inside Chesapeake Bay, from the NOAA digital elevation model. That model is derived from a variety of data sources − primarily depth soundings between the years 1859 and 1993. The COL applicant reference water level used for the simulations is an adjusted MSL at the Chesapeake Bay Bridge Tunnel, but is not based on NGVD29 as explained by the COL applicant. The optimal grid size for the bathymetry and model calculations, in terms of model accuracy and computational time requirements, was determined by the COL applicant to be 360 m (1181 ft) by 360 m (1181 ft) based on sensitivity analysis.

Using the offshore maximum tsunami amplitude and dominant period estimates described in COL FSAR Section 2.4.6.3, the COL applicant computed tsunami propagation within the Chesapeake Bay and estimated tsunami water levels at the CCNPP site for each of the three tsunami source scenarios. The COL applicant used two hydrodynamic models based on the shallow-water wave equations for the tsunami analyses: The NLSWE model that includes nonlinear terms and the effects of bottom friction in the momentum equations; and the TSU model which neglects these terms. The COL applicant implemented the equations numerically using a leap-frog finite-difference algorithm. For the NLSWE model, the COL applicant implemented the nonlinear terms using an upwind differencing scheme, and the bottom friction terms using an implicit corrector method. The COL applicant used a constant Manning's roughness coefficient of 0.025 for the bottom friction term. The COL applicant approximated frequency dispersion by using the numerical dispersion available from finite-differencing, and using a "hidden grid" technique to emulate physical dispersion. The spatial grid size used by the COL applicant for the model grid and bathymetry is 360 m (1181 ft) by 360 m (1181 ft). The COL applicant used a time step of 5 seconds over a total elapse time of 10 hours. The COL applicant verified the numerical models against analytical solutions from a Gaussian hump.

For the model boundary conditions, the COL applicant imposed a zero-flux or reflection boundary condition was imposed along the shoreline. Therefore, the COL applicant did not explicitly compute overland flow and runup. For the open-water boundaries, the COL applicant used a sponge-layer for the boundaries inside the Chesapeake Bay (i.e., Rappahannock River, Potomac River, and Pocomoke Sound). For the open-water boundary at the entrance of the Chesapeake Bay (between Plume Tree Point and Cape Charles) where the COL applicant applied the boundary forcing function. The COL applicant applied a radiation boundary condition.

The Staff's Technical Evaluation

Background: Numerical simulations of tsunami propagation have improved in the last 30 years. Several tsunami computational models are currently used in the National Tsunami Hazard Mitigation Program, sponsored by NOAA, to produce tsunami inundation and evacuation maps for the states of Alaska, California, Hawaii, Oregon, and Washington. The computational models include *Method Of Splitting Tsunami* (MOST), developed originally by researchers at the University of Southern California (Titov and Synolakis, 1998); Cornell Multi-grid Coupled Tsunami Model (COMCOT), developed at Cornell University (Liu and others, 1995); and TSUNAMI2, developed at Tohoko University in Japan (Imamura, 1996). All three models solve

the same depth-integrated and 2-dimensional (2D) horizontal (2DH) nonlinear shallow-water (NSW) equations with different finite-difference algorithms. The COL applicant's NLSWE model is based on these same equations.

For a given source region condition, existing models can simulate propagation of a tsunami over a long distance with sufficient accuracy for estimating tsunami water levels at the site, provided that accurate bathymetry data exist. However, the shallow-water equation models commonly lack the capability of simulating dispersive waves, which could be the dominating features in landslide-generated tsunamis and for tsunamis traveling a long distance. Several high-order, depth-integrated wave hydrodynamic models (e.g., Boussinesq-based models) are now available for simulating nonlinear and weakly dispersive waves, such as the Cornell University Long and Intermediate Wave Modeling Package (COULWAVE) (Lynett and Liu, 2002) and FUNWAVE (Kennedy and others, 2000). The major difference between the two is their treatment of moving shoreline boundaries. Lynett and others (2003) applied COULWAVE to the 1998 Papua New Guinea tsunami with the landslide source; the results agreed with field survey data. Recently, several finite element models have also been developed based on Boussinesq-type equations (Woo and Liu, 2004). Boussinesq-based models require higher spatial and temporal resolutions and, therefore, are more computationally intensive.

Technical Approach: The length scale of a submarine failure tends to be much less than that of an earthquake, and thus the wavelength of the created tsunami is shorter. To correctly simulate the shorter wave phenomenon, equations are needed with excellent shallow to intermediate water properties, such as the Boussinesq equations. While the Boussinesq model also has accuracy limitations on how deep (or short) the landslide can be (Lynett and Liu, 2002), the model is able to simulate the majority of tsunami generating landslides. For this report, the Boussinesq-based numerical model COULWAVE (Lynett and Liu, 2002) will be used. (See Appendix C of Lynett and Liu, 2002) for reprints of peer-reviewed papers that form the foundation of COULWAVE.) This model solves the fully nonlinear extended Boussinesq equations on a Cartesian grid. COULWAVE has the capability of accurately modeling the wind waves with both nonlinear and dispersive properties. A particular advantage of the model is the use of fully nonlinear equations for both deep and shallow water. This avoids the common problem of "splitting" the analysis when the wave reaches shallow water.

In RAI 99, Question 02.04.06-13, the staff requested that the COL applicant provide a complete description of the analysis procedure used to calculate tsunami wave height and period at the site. The staff also requested that the COL applicant indicate how tsunami runup on land was estimated from near-shore tsunami amplitude. In an August 29, 2009, response to RAI 99, Question 02.04.06-13, the COL applicant clarified that tsunami runup is not computed explicitly by either model (TSU or NLSWE). The COL applicant also used a runup amplification factor of three applied to the maximum tsunami amplitude in Chesapeake near the site. This factor is derived from analytic arguments described in the COL applicant's August 29, 2009, response to RAI 99, Question 02.04.06-13, which the staff finds acceptable. Accordingly, the staff considers RAI 99, Question 02.04.06-13 resolved.

As indicated in the COL FSAR, the COL applicant performed modeling of tsunami wave height and periods at the site, using nonlinear and linear versions of the shallow-water equations (NLSWE and TSU models, respectively). These models do not include the effects of dispersion and turbulence. Water-level analysis conducted under the staff's confirmatory analysis uses COULWAVE that accounts for these effects.

In RAI 99, Questions 02.04.06-9 and 02.04.06-10, the staff requested that the COL applicant provide input files, hydrodynamic model codes (NLSWE and TSU), and runup validation and/or field comparisons of the hydrodynamic model codes (NLSWE and TSU) used in the model simulations, in addition to the Gaussian hump comparison with analytic solutions. In July 30, 2009, and December 7, 2009, responses to RAI 99, Questions 02.04.06-9 and 02.04.06-10, the COL applicant submitted the input files and model codes for staff confirmatory analysis. The COL applicant also indicated that the linear modeling (TSU) results are bounding in terms of maximum and minimum water levels at the site. The model has been verified against the Gaussian source analytic solution and field observations of the 1983 Nihonkai-Chubu (Sea of Japan) tsunami. The COL applicant also provided additional verification of the nonlinear model with bottom friction (NLSWE). The staff finds the COL applicant's response acceptable and, therefore, considers RAI 99, Questions 02.04.06-9 and 02.04.06-10 resolved.

In RAI 99, Question 02.04.06-14, the staff requested that the COL applicant describe the data source and method used to develop bathymetric grid for the tsunami model including a description of the grid-size sensitivity test. In an August 29, 2009, response to RAI 99, Question 02.04.06-14, the COL applicant provided additional details of the bathymetric data and the development of the digital elevation model used for the tsunami model. The COL applicant also provided the results of the sensitivity test regarding grid size. The COL applicant indicated that these descriptions and model results will be included in combined license application (COLA) changes as part of the response to RAI 99, Question 02.04.06-17. The staff finds the COL applicant's August 29, 2009, response to RAI 99, Question 02.04.06-14, acceptable and, therefore considers RAI 99, Question 02.04.06-14 resolved.

In summary, although not using state-of-the-art modeling, the COL applicant's tsunami analysis is acceptable and conservative.

2.4.6.4.5 Tsunami Water Levels

Information Submitted by COL Applicant

The COL applicant performed four propagation simulations to establish the PMT water levels relative to MSL at the CCNPP site:

- Norfolk Canyon landslide source scenario using the NLSWE model
- Canary Islands volcanic flank failure source scenario using the NLSWE model
- Greater Antilles earthquake source scenario using the NLSWE model
- Norfolk Canyon landslide source scenario using the TSU model

Using the NLSWE and TSU tsunami propagation models inside the Chesapeake Bay, the COL applicant established the PMT water levels (maximum amplitude and drawdown) relative to MSL from the three regional and distant tsunami sources described in COL FSAR Section 2.4.6.3. The COL applicant adjusted the water levels from the simulations by including a 0.1 m (0.33 ft) margin of error (approximately 20 percent of the simulated values) and including the 10 percent exceedance tides at the CCNPP site; the staff considers this increase negligible. For the latter adjustment, the maximum amplitude includes the 10 percent exceedance high tide (0.66 m (2.2 ft)) and the maximum drawdown includes 10 percent exceedance mean lower low water (MLLW) level (0.003 m (0.0098 ft)).

From the simulations, different tsunami sources are associated with the maximum tsunami amplitude and drawdown water levels. The COL applicant results from the NLSWE model indicate that the maximum amplitude water level is associated with the Norfolk Canyon landslide (Case 1); whereas, the maximum drawdown is associated with the Greater Antilles earthquake (Case 3). Since the TSU model does not have energy dissipation from bottom friction and nonlinear terms, the COL applicant used the water levels from this model with the Norfolk Canyon source (Case 4) to establish the PMT water levels at the CCNPP site. With an added margin of uncertainty of 0.1 m (0.33 ft), the COL applicant's water levels are 0.5 m (1.64 ft) in amplitude and -0.5 m (-1.64 ft) in drawdown. Taking into account for the 10 percent exceedance high tide and MLLW levels, the COL applicant's PMT high and low-water levels are 1.16 m (3.8 ft) and -0.5 m (-1.64 ft), respectively.

The COL applicant indicated that the PMT high and low-water levels are less than the design-basis high and low-water levels of 5.82 m (19.1 ft) and -1.83 m (-6 ft), respectively. The design-basis high-water level is from a probable maximum storm surge event; whereas, the design-basis low-water level is from the passage of the probable maximum hurricane along the East Coast.

The Staff's Technical Evaluation

The staff performed numerical modeling simulations of three different tsunami sources to determine their impact on the CCNPP 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 CCNPP site.

In RAI 99, Question 02.04.06-15, the staff requested that the COL applicant provide an estimate of maximum (high water) and minimum (low water) tsunami wave heights from both distant and local generators. The staff also requested that the COL applicant provide a discussion in the updated COL FSAR of the water levels for all simulations (NLSWE and TSU models), so that the limiting water levels can be confirmed. In an August 24, 2009, response to RAI 99, Question 02.04.06-15, the COL applicant provided all tsunami water-level results from the different tsunami source scenario/hydrodynamic model combinations. The COL applicant also indicated that these results will be included in COLA changes as part of the response to RAI 99, Question 02.04.06-17. The staff confirmed that these results are included in COL FSAR, Revision 8 (March 2012), Section 2.4.6.5 (Tsunami Water Levels). Accordingly, the staff considers RAI 99, Question 02.04.06-15 resolved.

In RAI 99, Question 02.04.06-16, the staff requested that the COL applicant provide a discussion of how uncertainty in simulated tsunami water levels was determined. In an August 24, 2009, response to RAI 99, Question 02.04.06-16, the COL applicant indicated that in the initial COL FSAR, an uncertainty in water levels of 20 percent was based on engineering judgment. The COL applicant provided a more thorough uncertainty analysis, resulting in a factor of 60 percent added to the modeled tsunami water levels predicted by the linear model (TSU). These results will be included in COLA changes as part of the response to RAI 99, Question 02.04.06-17. The staff confirmed that these results are included in COL FSAR, Revision 8 (March 2012), Section 2.4.6.5 (Tsunami Water Levels). Accordingly, the staff considers RAI 99, Question 02.04.06-16 resolved.

In RAI 99, Question 02.04.06-17, the staff requested that the COL applicant provide a discussion of long-term sea-level rise that may be coincident with tsunami water levels. In an August 24, 2009, response to RAI 99, Question 02.04.06-17, the COL applicant indicated that the nominal increase in sea-level rise at the site is 0.3 m (1 ft) for the design period of the plant. The staff finds the COL applicant's response acceptable and, therefore, considers RAI 99, Question 02.04.06-17 resolved.

Numerical Grid Development: The bathymetry/topography grid required by the hydrodynamic model (COULWAVE) is created via two main sources: (1) The general bathymetric chart of the oceans 1-minute global elevation database; and (2) 100-m (320-ft) resolution elevation taken from the USGS database for Chesapeake Bay. For the latter, the bathymetric grid for Chesapeake Bay was created using hydrographic soundings data extracted from the NOAA National Ocean Service database (http://www.ngdc.noaa.gov/mgg/bathymetry/hydro.html). Elevation data for land areas within the grid were extracted from the USGS National Elevation Dataset (http://ned.usgs.gov/). These data were gridded using a Gaussian-weighted average scheme with interpolation of empty cells via a 2D thin-plate spline algorithm. Data density over much of the Chesapeake Bay was high enough to support a pixel resolution for the computational grid of 100 m (328 ft). Questions concerning PMT effects on coastal waterways, Chesapeake Bay channels, and associated rivers are all resolved.

Numerical Simulations – Physical Limits: The purpose of these simulations is to provide an absolute upper limit on the tsunami wave height that could be generated by the three potential sources. Note that these limiting simulations use physical assumptions that are arguably unreasonable for landslide sources; the results of these simulations will be used to filter out tsunami sources that are incapable of adversely impacting the CCNPP site under even the most conservative assumptions. Specifically, these assumptions are as follows:

1. Time scale of the seafloor motion is very small compared to the period of the generated water wave (tsunami).

2. Bottom roughness and the associated energy dissipation are negligible in locations that are initially wet (i.e., locations with negative bottom elevation, offshore).

Assumption (1) simplifies the numerical analysis considerably. With this assumption, the sea surface response matches the change in the seafloor profile exactly. This type of approximation is used commonly for subduction-earthquake-generated tsunamis, but is known to be very conservative for landslide tsunamis (Lynett and Liu, 2002). The modeling simplification arises, because the time during which the submarine landslide evolves is not considered in the simulation. The initial pre-landslide bathymetry profile, as estimated by examination of neighboring depth contours, is subtracted by the post (existing) landslide bathymetry profile. This "difference surface" is smoothed and then used directly as an instantaneous initial free surface condition in the hydrodynamic model.

Assumption (2) does not simplify the analysis significantly; however, it does prevent the use of an overly high bottom roughness coefficient, which could artificially reduce the tsunami energy reaching the shoreline owing to friction. The staff notes that while the offshore regions are assumed to be without bottom friction, such an assumption is too physically unrealistic to accept for the inland regions where the roughness height may be the same order as the flow depth. For tsunami inundation, particularly for regions such as this project location where the run-up

wave might inundate long reaches of densely vegetated land, inclusion of some measure of bottom roughness in the analysis is necessary.

Currituck Landslide Source: As described previously, the excavation depth associated with this submarine landslide event is approximately 300 m (984 ft). This length provides the trough elevation (i.e., -300 m (-984 ft)) initial instantaneous water-surface condition. The horizontal dimensions of the slide source region are ~20 km (12 mi) in width and 50 km (31 mi) in length. With this information, and knowledge of characteristic slide-generated waves taken from the literature (Lynett and Liu, 2002; Lynett and Liu, 2005), the instantaneous initial condition is assumed for the purposes of tsunami formation.

For this tsunami hazard investigation, the simulation domain will be divided into two separate, but coupled, components; an offshore domain and a nearshore domain. First, a simulation is performed to examine waves near the offshore generation source and their subsequent evolution in shallow water approaching the Chesapeake Bay. These simulations provide a time series of water-surface elevation and fluid velocity near the Chesapeake Bay entrance. These time series are used to force the nearshore domain, which encompasses the entire Chesapeake Bay. The two domains, offshore and nearshore were both too large in memory and computational requirements to be run simultaneously.

For the Currituck landslide, only a one-horizontal-dimension (1HD) simulation was performed to examine the offshore source; as will be shown in this section, the tsunami height at the CCNPP site resulting from the 1HD offshore source does not justify the further investigation into the less conservative 2HD simulation. The 1HD simulations require a small fraction of the CPU time of the 2HD runs but do not include the radial spreading and refraction effects. Physically, a 1HD simulation is approximating a simultaneous slope failure of the entire continental shelf along the eastern seaboard.

First, results from the 1HD offshore domain are discussed. The depth transect is taken from the source location directly to the Bay entrance. A constant spatial grid size of 25 m (82 ft) is used across the transect for the 1HD cases. The simulation is based on the fully nonlinear Boussinesq equations, with wave breaking included. The staff notes that the entire bottom profile is submerged and, thus, there is no bottom friction dissipation in any form in this simulation.

Although the generated wave is initially characterized as a leading depression wave, this depression is quickly overrun by the following and faster-moving positive elevation wave. The wide shallow shelf leads to a depth-limiting effect on the wave height. This height decreases from ~200 m (656 ft) at the shelf break to ~30 m (99.4 ft) near the Chesapeake Bay entrance. By this time, the incident wave has transformed into a long period pulse of positive elevation energy.

While there is little in the literature to evaluate these results in any context, they can be compared with the numerical simulations presented in Geist and others (2009). In that paper, attempts were made to simulate the waves directly from an assumed landslide motion. That is to generate the waves physically from the bottom boundary condition rather than use an initial hot start condition. Also in this paper, the wave on the shelf was simulated in 1HD, similar to the USGS study prepared for the NRC. (See Atlantic and Gulf of Mexico Tsunami Hazard Assessment Group (2007)). Geist and others (2009) determined the tsunami elevation near the shoreline to be ~ 6 m (19.7 ft), while at the shelf break it was ~15 m (49.2 ft). The difference in

reduction factors, 6/15 equal to 0.4 from the Geist and others (2009) paper and 30/200 equal to 0.15 from this NRC study, is attributed to the depth-limiting effect. With long lengths of shallow depth propagation, large amplitude waves will be dissipated – here meaning reduced in amplitude -- much faster than relatively smaller waves.

Next, with a time series from the 1HD offshore simulation taken near the Chesapeake Bay entrance, the nearshore domain simulation can proceed. The nearshore domain uses a constant spatial grid size of 100 m (328 ft). The simulation is based on the fully nonlinear Boussinesq equations, with wave breaking included. On initially dry land, bottom friction due to small/moderate roughness characteristic for grass/turf (equal to 0.01) is employed; elsewhere, again there is no friction.

Of immediate note is the rapid attenuation of tsunami wave height through the initial bend of the Chesapeake Bay. As the Chesapeake Bay entrance and the main channel are not in-line, there is a significant amount of directional scattering just inside the Chesapeake Bay; the tsunami elevation immediately inside the Chesapeake Bay is greater than 20 m (65.6 ft), yet 50 km (164.1 ft) up channel, the maximum elevation is close to 5 m (16.4 ft). The wave height continues to diminish as the wave propagates further up channel due to directional interference. Near the CCNPP site, the maximum water-surface elevation is less than 1.5 m (4.92 ft). Additionally, the steep wave front has all but disappeared near the CCNPP site, and the wave rises gradually over a time period of approximately 20 min.

The CCNPP site is at a local sea surface elevation minimum, with a maximum elevation of 1.4 m (4.6 ft). This local minimum is due to the constriction of the channel. This constriction should lead to relatively lower water-surface elevations but high-water velocities. Again, the largest values are isolated to locations near the entrance, and quickly diminish inside the Chesapeake Bay. However, water velocities near the entrance are extreme, with a large area experiencing speeds greater than 10 m/s (32.8 ft/s). As expected, the channel just offshore of the CCNPP site shows a local maximum in water velocities. Here, the water velocity reaches 2.5 m/s (8.2 ft/s). This large velocity is largely isolated to the Chesapeake Bay channel, and maximum speeds within a kilometer of the shoreline near the CCNPP site do not exceed 1.3 m/s (4.3 ft/s).

Again, the 1HD offshore simulation is used to drive the approaching tsunami. The staff deemed it unnecessary to repeat this analysis with a 2HD offshore simulation, which would yield a considerably smaller wave height near the Chesapeake Bay entrance. The reason for this was that the 1HD-source waves do not produce hydrodynamic effects of concern at the site; a 2HD-source would create an even smaller effect. It is also noted that, by using the 1HD simulation and assuming an instantaneous landslide, the staff effectively estimated an absolute upper limit on the wave impact from any landslide source on the eastern U.S. seaboard.

Canary Islands Source: The initial tsunami assessment by Ward and Day (2001), due to a coherent failure of an entire island into the ocean, leads to runup predictions of 10-25 m (32.8-82 ft) along nearly the entire U.S. East Coast. Subsequent studies (e.g., Mader, 2001) have attempted to downplay the hazard, with reductions in runup by a factor of 10 for the most extreme case. In this study, the most conservative published source values will be employed.

The approach for the Canary Island simulation utilized three different computational domains. The first is the Atlantic Ocean domain (ocean domain), which was used to model the tsunami from its source to the continental shelf of the eastern U.S. The output from the ocean domain

was used to force domain-focused effects of the continental shelf break and the shallow shelf waters (shelf domain). The reason for this separation of offshore domains is due to the fact that important physical spatial scales in the open ocean are on the order of magnitude of 1-10 km (0.62-6.2 mi), while on the shelf, where front steeping and breaking play a role, the relevant length scales are on the order of 10-100 m (32.8-328 ft). To accommodate this variability across several orders of magnitude, it is computationally acceptable to tackle the problem with separated domains, executed independently. The third domain used for this modeling scenario was the same nearshore domain as used with the Currituck scenario, which is forced with output from the shelf domain.

Following Ward and Day (2001), a coherent La Palma (one of the volcanic features in the Canary Islands) collapse scenario would generate an initial wave with amplitude approaching 1,000 m (3280 ft). For the simulations here, an instantaneous wave formation condition is assumed just offshore of La Palma, with a crest elevation of +1,000 m (3,280 ft) and a trough elevation of -1,000 m (-3,280 ft). The wave front disturbance has a length of 50 km (31.1 mi) and a width of 25 km (15.05 mi), again taken approximately from the information in Ward and Day (2001). The wave propagation is modeled in the entire northern Atlantic Ocean in the ocean domain, using a grid length of 2 km (1.24 mi). The numerical simulation is based on the fully nonlinear Boussinesq equations, with wave breaking occurring 30 minutes after generation. The wave field spreads radially and in frequency, almost as a point source. In time, the tsunami has transformed into a long train with the longest frequencies at the lead; note that the largest crest does not occur with the first wave. When reaching the continental shelf break along the eastern U.S., the maximum tsunami crest elevation is less than 10 m (32.8 ft); in a depth of 2,000 m (6,562 ft). The leading tsunami wave has a period of ~750 seconds and decreases to ~350 seconds near the back end of the train. The largest wave heights are located within this period range.

The 2 km (1.24 mi) grid used in the Atlantic Ocean simulation described above does not have sufficient resolution to evaluate tsunami shoaling and dissipation processes associated with the shallow continental shelf. Thus, to estimate the wave height at the entrance of the Chesapeake Bay from the Canary Islands teletsunami, a second offshore simulation must be run, described above as the shelf domain. The wave disturbance as it approaches the shelf has little along-coast variability, and it is deemed that a 1HD, cross-sectional simulation will very reasonably capture the transformation of this wave train over the shelf break and across the shallow shelf.

At the continental shelf break, the largest of the waves shoal to a great height, with crest elevations close to 40 m (131 ft), and break immediately thereafter. These breaking waves then form individual bore fronts which quickly travel across the shallow water shelf, decreasing in crest elevation as they approach the shoreline. The resulting disturbance consists of a large number of ~5 m (16.4 ft) high bore fronts, one after the next, spaced 2-8 min apart. These bore fronts can become stacked on top of one another. This superposition process, driven by amplitude dispersion, can lead to an amplified bore front if a trailing large bore overtakes and combines with a leading smaller and slower traveling bore.

The numerical setup of this domain component is identical to that described in the earlier Currituck scenario section. Due to the relatively short period of the individual pulses compared to the Currituck wave, as well as the smaller incident crest elevations, less wave energy is able to travel farther into the Chesapeake Bay. Similar to Currituck, the directional scattering of the wave at the entrance is the primary wave height reducer. The maximum simulated

water-surface elevation and fluid speed at the CCNPP site for the Canary Islands tsunami is also smaller than that due to the Currituck event. For the Canary Islands tsunami, the maximum sea surface elevation is 1.25 m (4.10 ft), and the maximum wave speed is 1.1 m/s (3.6 ft/s) at the CCNPP site. Thus, despite the relatively large wave heights at the source region, by the time the wave train has spread radially in the Atlantic, spread energy through frequency dispersion, and dissipated due to breaking along the continental shelf having traversed the geometrically irregular Bay, the tsunami elevation height is reduced by a factor of ~800.

Puerto Rico Subduction Zone Source: The last teletsunami source to be investigated for the CCNPP site is the subduction zone that borders much of the northeastern and eastern extent of the Caribbean Islands. Here, the staff assumed that the entire fault zone, composed of five different fault segments, ruptured during a single earthquake event (see Chapter 8 in ten Brink and others, 2008). Using a stochastic slip model (Geist, 2002), fault slip was varied keeping the overall seismic moment constant (~M 9.1). Tsunami amplitudes at the Chesapeake Bay entrance were calculated using the linearized shallow-water equations for 100-slip realizations. The slip producing the maximum tsunami amplitude at the Chesapeake Bay entrance was then used for the more accurate COMCOT calculations.

The initial sea surface condition, which is a direct mapping of the vertical seafloor displacement to the ocean surface, considers five fault segments. The largest tsunami waves will be directed toward the northeast Atlantic basin. Fault slip along the Lesser Antilles segment likely has little influence on Chesapeake Bay tsunami amplitudes. With a subduction zone earthquake, the generated waves are long in wavelength. This implies that the physics of the waves are simpler, relative to the dispersive waves created by the two landslide sources examined previously. To numerically model this source, the open-source tsunami model COMCOT is used. A grid covering the entire western Atlantic is generated with a spatial grid size of 1 min (1/60 of a degree latitude or longitude or approximately 1.8 km (1.1 mi)). A single grid layer is used; there is no nesting of domains for refinement. The time step used in the model is 1 second. The linear version of the model is used, and there is no bottom friction applied anywhere in the domain. The linear version of the model is deemed acceptable because, as will be shown, the wave height to water depth ratio is less than 0.1 at all areas of interest, and usually no greater than 0.01.

Once the tsunami wave exits the source generation area, the crest elevation of the main wave is about 2 m (6.06 ft) in the open ocean. Thus, the U.S. East Coast, while certainty feeling effects from this source, would see relatively minor wave impact. By the time the wave has reached the continental shelf offshore of the Chesapeake Bay, the maximum crest elevation of the wave is approximately 1 m (3.28 ft). When the wave hits the shallow shelf, the wavelength shortens quickly, and the wave height increases.

Due to the small offshore height of the wave compared to the two previously examined sources, it is not expected that this wave would break and steepen into bore fronts near the shelf break. In this location, at the shelf break, the water depth is roughly 50 m (164 ft), while the wave height is approximately 3 m (9.8 ft), such that the transformation processes will still be largely governed by linear shallow water physics. As the wave approaches the Chesapeake Bay entrance, shoaling amplification and refractive spreading approximately cancel each other, and the resulting wave crest elevation entering the Chesapeake Bay is 1.5 m (4.92 ft). Compared to the near-Chesapeake Bay maximum crest elevation of 20 m (65.6 ft) for the Currituck source and 10 m (32.8 ft) for the Canary Islands source, the Puerto Rico subduction zone source is not likely to produce larger impacts at the CCNPP site. The water level from the tsunami generated

by an earthquake along the Puerto Rico subduction zone at the CCNPP site is well below 0.25 m (0.82 ft).

Summary: The local submarine landslide source (Currituck landslide) proved to have the largest impact at the CCNPP site, with the tsunami generating maximum water-surface elevations of 1.4 m (4.59 ft) and maximum fluid speeds of 1.3 m/s (4.27 ft/s). The Canary Islands source, despite generating a sea surface elevation of 1 km (0.6 mi) at the source, leads to a tsunami crest elevation of 1.25 m (4.10 ft) near the CCNPP site. The earthquake source (Puerto Rico subduction zone) has by far the smallest effect on the site, with a maximum water-surface elevation of less than 0.25 m (0.82 ft). Thus, the local Currituck-like landslide source is the PMT in terms of the maximum water-level elevation (1.4 m (4.59 ft)).

Independent analysis of the 10 percent exceedance high tide was conducted for 13 years of NOAA NOS-CO-OPS data at the Solomons Island, MD tide gauge station (years 1996-2009). The 10 percent exceedance high tide was determined to be 0.68 m (2.23 ft) (NAVD88) for these years, compared to 0.66 m (2.17 ft) indicated by the COL applicant's analysis. 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 0.68 m (2.2 ft) (high tide) plus 0.34 m (±1. ft) (sea-level anomaly) plus 0.37 m (± 1 ft) (100-year sea-level rise plus 1 standard deviation) which equals 1.39 m (4.52 ft). The PMT high-water level is, therefore, 1.39 m (4.52 ft) plus 1.4 m (4.6 ft), which equals 2.79 m (9.15 ft). This is less than the PMT high-water level of 3.5 m (11.5 ft) estimated by the COL applicant. The staff notes that the COL applicant included maximum tsunami amplitude in the antecedent water level. The COL applicant added this water level to the estimated runup (their offshore amplitude proxy) to obtain the PMT high-water level.

The minimum water-level drawdown is from the Puerto Rico subduction zone earthquake and is no less than -0.25 m (-0.82 ft). The COL applicant's estimate of the minimum water-level drawdown is –0.38 m (-1.25 ft). The COL applicant used MLLW as the antecedent water level to establish PMT low water level described in Section 2.4.11, "Low Water Considerations," of this report. The staff has not reviewed the appropriateness of this antecedent water level.

2.4.6.4.6 Hydrography and Harbor or Breakwater Influences on Tsunami

Information Submitted by COL Applicant

The COL applicant indicated that the bathymetry of the Chesapeake Bay is explicitly accounted for in the tsunami propagation computations. The COL applicant identified a platform associated with the Dominion Cove Point liquefied natural gas (LNG) facility and indicated that it would not likely obstruct tsunami propagation.

The Staff's Technical Evaluation

Based on the staff evaluation of the COL applicant's numerical simulations provided in COL FSAR Section 2.4.6.4.5, the staff concurs that the bathymetry of the Chesapeake Bay is adequately included in the tsunami propagation computations. The Dominion Cove Point LNG facility likely will not obstruct tsunami propagation.

2.4.6.4.7 Effects on Safety-Related Facilities

Information Submitted by COL Applicant

The COL applicant's analysis indicated that the maximum water level from tsunamis is less than the grade elevation for the plant. Therefore, the COL applicant concluded that there will be no tsunami waves affecting safety-related facilities on the nuclear power block. For the safety-related UHS MWIS, the COL applicant indicated that the limiting design bases for high- and low-water levels are controlled by the PMSS and not by the PMT.

The Staff's Technical Evaluation

The staff concurs with the COL applicant that because the maximum tsunami water level associated with the PMT is below grade elevations at the site, there will be no onsite tsunami waves affecting safety-related facilities. Minimum low water levels associated with the PMT do not define the design basis for the safety-related UHS MWIS.

2.4.6.4.8 Hydrostatic and Hydrodynamic Forces

Information Submitted by COL Applicant

For the safety-related UHS MWIS, the COL applicant indicated that the hydrostatic and hydrodynamic design bases are controlled by the PMSS and not by the PMT.

The Staff's Technical Evaluation

The staff concurs that the PMT does not define the hydrostatic and hydrodynamic design basis.

2.4.6.4.9 Debris and Water-Borne Projectiles

Information Submitted by COL Applicant

For the safety-related UHS MWIS, the COL applicant indicated that it is unlikely the baffle channel in front of the intake channel would be affected by tsunami-related debris and projectiles. The COL applicant also indicated that the elevation of the UHS intake operating deck and pump room floors where safety-related equipment is located will be at 3.51 m (11.69 ft) and, thus, higher than the high-water level associated with the PMT.

The Staff's Technical Evaluation

The staff concurs that the debris and water-borne projectiles associated with the PMT will likely not affect the UHS water intake baffle channel. Debris impact forces vary with the flow velocity either linearly (Federal Emergency Management Administration 2011) or to a power of 1.2 (Matsutomi, 1999). Maximum tsunami flow velocity at the site is estimated to be 1.3 m/s (4.27 ft/s). By comparison, the COL applicant indicated in COL FSAR Section 2.4.1.2.1.3, "The Chesapeake Bay Estuary," that the maximum tidal currents reported by NOAA in the Chesapeake Bay range from 0.56 m/s (1.8 ft/s) (Baltimore Harbor entrance) to 1.3 m/s (4.3 ft/s) (Chesapeake City). Therefore, the staff finds there no significant hazard from debris and water-borne projectiles entrained in a tsunami compared to debris and water-borne projectiles entrained by tidal currents. A summary of impact forces, as well as other tsunami-induced forces on structures is given by Nistor and others (2010).

2.4.6.4.10 Effect of Sediment Erosion and Deposition

Information Submitted by COL Applicant

For the safety-related UHS MWIS, the COL applicant indicated that the currents in the forebay intake channel would be too small to cause erosive effects. The COL applicant also indicated debris transported by the tsunami would be screened at the baffle wall that is of a height greater than the range of high- and low-water levels associated with the PMT.

The Staff's Technical Evaluation

The staff concurs that the effects related to sediment erosion and deposition in the forebay intake channel are likely to be minimal. Maximum currents at the site are estimated to be 1.3 m/s (4.3 ft/s).

2.4.6.4.11 Consideration of Other Site-Related Evaluation Criteria

Information Submitted by COL Applicant

The COL applicant indicated that the tsunami-generating sources considered in establishing the PMT result in either no ground motions (in the case of landslides) or smaller ground motions (in the case of far-field earthquakes) than the design-basis earthquake at the CCNPP site. Therefore, the PMT sources will not be combined with the design-basis earthquake when evaluating the design of safety-related systems, structures, and components.

The Staff's Technical Evaluation

The staff concurs with the COL applicant's determination that the PMT sources will not be combined with the design-basis earthquake when evaluating the design of safety-related systems, structures, and components.

2.4.6.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.6.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR. On the basis of its review and the staff's independent calculations, the staff confirmed that the COL applicant addressed the required information relating to probable maximum tsunami hazards. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to probable maximum tsunami hazards incorporated by reference in the COL FSAR will be documented in the staff safety evaluation report on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will update Section 2.4.6 of this report to reflect the final disposition of the design certification application.

2.4.7 Ice Effects

2.4.7.1 *Introduction*

COL FSAR Section 2.4.7 addresses ice effects to ensure that safety-related facilities and water supply are not affected by ice-induced hazards.

This section presents an evaluation of the following topics based on data provided by the COL applicant in the COL FSAR and information available from other sources: Ice conditions and historical ice formation; ice jam events; the effect of ice on the cooling-water system; and any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.7.2 *Summary of Application*

This section of the COL FSAR addresses the information related to ice effects at the proposed CCNPP Unit 3 site. COL FSAR Section 2.4.7 incorporates by reference U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.7, "Ice Effects." In this section, the COL applicant provided site-specific supplemental information to address COL-specific information identified in U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-7

> A COL applicant that references the U.S. EPR design certification will provide site-specific information regarding ice effects and design criteria for protecting safety-related facilities from ice-produced effects and forces with respect to adjacent water bodies.

COL Information Item 2.4-8

> A COL applicant that references the U.S. EPR design certification will evaluate the potential for freezing temperatures that may affect the performance of the ultimate heat sink makeup, including the potential for frazil and anchor ice, maximum ice thickness, and maximum cumulative degree-days below freezing.

The COL applicant stated that the UHS and mechanical draft cooling towers are described in COL FSAR Section 9.2.5. The water temperature in each of the four UHS cooling-tower basins is monitored. If basin water temperature drops to 4.4°C (40°F), an alarm would alert the operator to place the associated train in operation to prevent the formation of ice in the basin. Under extended low-load or low ambient temperature conditions, it may be necessary to have all four essential service water (ESW) trains operating. Chemicals may also be added to the ESW system to lower the point at which the cooling water freezes. The UHS cooling-tower fans would be capable of operation in reverse direction for short periods to minimize ice buildup at the air inlets. As described in the U.S. EPR FSAR, makeup water supply to the UHS cooling-tower basin would be site-specific.

2.4.7.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the identification and evaluation of ice effects, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.7.

The applicable regulatory requirements for identifying ice effects are set forth in

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.
- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d) sets forth the criteria to determine the siting factors for plant design bases with respect to water-surface elevations at the site.

The staff also used the appropriate sections of the following RGs for the acceptance criteria identified in NUREG-0800, Section 2.4.7:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding or those associated with the MWIS
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site

The regulatory basis of the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.7.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.7. The staff confirmed that the information in the COL application addresses the relevant information related to ice effects at the CCNPP Unit 3 site. The staff's technical review of this section included an independent review of the COL applicant's information in the COL FSAR. The staff supplemented this information with other publicly available sources of data. The staff independently assessed the potential for formation of ice at the CCNPP Unit 3 site using available data. The staff's evaluation is described below.

Information Submitted by COL Applicant

The COL applicant evaluated the historical air temperatures at the CCNPP Unit 3 site using data from the nearby Patuxent River Naval Air Station meteorological tower for the period from 1945 through 2006. The COL applicant estimated the maximum Accumulated Freezing Degree Days (AFDD) to be 147.4 °C-days (265.3 °F-days) occurring on February 9, 1977, and the corresponding ice thickness to be approximately 0.33 m (13 in.). In the COL FSAR, the COL applicant stated that if ice forms at maximum thickness on the surface of the Chesapeake Bay, it would not affect the water supply at the UHS MWIS channel. The water supply would remain unaffected, because the bottom of the intake channel would be 10.4 m (34 ft) to 13.7 m (45 ft) below the minimum operating water elevation of -1.8 m (-6 ft) at the Chesapeake Bay. The COL applicant suggested that formation of frazil or anchor ice is considered highly unlikely based on the historical climate records. Furthermore, the COL applicant 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 CCNPP Units 1 and 2 back to the MWIS forebay.

The Staff's Technical Evaluation

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 CCNPP Unit 3 site. The staff estimated the maximum AFDD to be 136.7 °C-days (246 °F-days) compared to the 147.4 °C-days (265.3 °F-days) reported in the COL FSAR. Ice thickness, t, in inches is then estimated using the modified Stefan equation (USACE 2002):

$$t = C\sqrt{AFDD}$$

where C is a dimensionless coefficient, usually ranging between 0.2 and 0.8, and AFDD is in °F-days. Using the equation above with a conservatively large coefficient of 0.8 (for windy lake with no snow; USACE 2002), the staff estimated the maximum possible thickness of an ice sheet in the Chesapeake Bay near the CCNPP Unit 3 site to be approximately 0.32 m (12.5 in.). The staff's estimate of ice thickness is less than the COL applicant's estimate reported in the COL FSAR. An ice sheet thickness of 0.05 to 0.2 m (2 to 8 in.) was observed south of the Chesapeake Bay Bridge in early February 1977 (Foster 1982). Since brackish water has a lower freezing point than freshwater, a larger fraction of AFDD would be needed to initiate freezing and, therefore, the thickness of the ice sheet would be less compared to that in freshwater. Therefore, the staff concluded that its estimate of ice sheet thickness is conservative because (1) the largest recommended value for the coefficient C was used in the equation above, and (2) a correction for later initiation of freezing in the brackish waters of the Chesapeake Bay was not made.

Since the existing MWIS channel is 10.4 m (34 ft) to 13.7 m (45 ft) below the minimum operating elevation of -1.8 m (-6 ft) in the Chesapeake Bay, the staff determined that in the worst icing event, formation of an ice sheet of 0.33 m (13 in.) thick on the surface of the Chesapeake Bay, the water supply at the UHS MWIS would not be affected.

According to NOAA's Chesapeake Bay water temperature map, 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. As reported in the COL FSAR, no frazil ice or anchor ice has been observed in the MWIS since the start of operation of the existing CCNPP Units 1 and 2. Based on the historical climate records, the staff determined that frazil

ice or anchor ice is unlikely to occur and that even if it did occur it would not affect the function of the MWIS intakes.

In RAI 328, Question 02.04.10-1, the staff requested that the COL applicant provide quantified information in the COL FSAR demonstrating that the UHS system and associated safety-related pipe system would be protected from adverse effects of natural phenomena, including flooding from local intense precipitation, storm surge, tsunami, ice formation, and groundwater. The staff reviewed the COL applicant's December 20, 2012, response to RAI 328, Question 02.04.10-1, in which the COL applicant stated that the safety-related pipes would be buried at a depth greater than 2.2 m (6.75 ft), which is well below the frost-line depth. The staff reviewed the Calvert County, MD, building construction regulations related to frost damage protection, which specifies a 0.6-m (2-ft) frost-line depth design criterion (Calvert County 2009). The staff compared this criterion to the burial depth of the pipes stated in the COL applicant's response and agrees with the COL applicant's conclusions. The margin between the Calvert County's frost-line depth design criterion and the safety-related pipes' burial depth is more than twice the design depth, 1.4 m (4.75 ft), which provides additional assurance that ice and frost events more severe than those considered in Calvert County's regulations would not adversely affect the pipes' safety. Based on the discussion above, the staff finds the COL applicant's December 20, 2012, response to RAI 328, Question 02.04.10-1 acceptable. Accordingly, the staff considers RAI 328, Question 02.04.10-1 resolved.

2.4.7.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.7.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR Revision 4. On the basis of its review, the staff confirmed that the COL applicant addressed the required information related to ice and frazil formation. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to ice and frazil formation incorporated by reference in the COL FSAR will be documented in the staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will update Section 2.4.7 of this report to reflect the final disposition of the design certification application.

2.4.8 Cooling-Water Canals and Reservoirs

2.4.8.1 *Introduction*

COL FSAR Section 2.4.8, "Cooling Water Canals and Reservoirs," addresses the cooling-water canals and reservoirs used to transport and impound water supplied to the safety-related SSCs. This section presents an evaluation of the design basis for the capacity and operating plan for safety-related cooling-water canals and reservoirs, and any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.8.2 *Summary of Application*

This section of the COL FSAR describes site cooling-water canals and reservoirs and the flooding and low-water risk that they pose to the proposed unit. COL FSAR Section 2.4.8 incorporates by reference U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.8, "Cooling Water Canals and Reservoirs."

The COL applicant addressed the issues as follows:

COL Information Item 2.4-9

> A COL applicant that references the U.S. EPR design certification will provide site-specific information and describe the design basis for cooling water canals and reservoirs used for makeup to the UHS cooling-tower basins.

The COL applicant provided additional information in COL FSAR Section 2.4.8 to address U.S. EPR FSAR Tier 2, Revision 4, Table 1.8-2, "U.S. EPR Combined License Information Items," COL Information Item 2.4-9, which describes the site water canals and reservoirs. For the U.S. EPR, the UHS would be provided by mechanical draft cooling towers, as described in COL FSAR Section 9.2.5, "Ultimate Heat Sink." As described in the U.S. EPR FSAR, makeup water supply to the UHS cooling-tower basin would be site-specific.

The COL applicant reported that the design of CCNPP Unit 3 does not contain any cooling-water canals or reservoirs.

2.4.8.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the cooling-water canals and reservoirs, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.8.

The applicable regulatory requirements for cooling-water canals and reservoirs are set forth in

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.
- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water-surface elevations at the site.

The staff also used the appropriate sections of the following RGs for the acceptance criteria identified in NUREG-0800, Section 2.4.8:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding or those associated with the MWIS

- RG 1.59, “Design Basis Floods for Nuclear Power Plants,” as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterize
- RG 1.102, “Flood Protection for Nuclear Power Plants,” as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site

The regulatory basis of the information incorporated by reference will be addressed in the staff’s FSER related to the U.S. EPR design certification application.

2.4.8.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.8. The staff confirmed that the information in the COL application addresses the relevant information related to the site cooling-water canals and reservoirs. The staff’s technical review of this section included an independent review of the COL applicant’s information in the COL FSAR. The staff supplemented this information with other publicly available sources of data. The staff reviewed the COL applicant’s information in COL FSAR Section 2.4.8. The staff’s evaluation is described below.

Information Provided by COL Applicant

In COL FSAR Section 2.4.8, the COL applicant stated that CCNPP Unit 3 would not include any canals or reservoirs used to transport or impound plant safety-related water for heat dissipation.

The Staff’s Technical Evaluation

The staff independently determined that there would be no plausible scenario, involving either cooling canals or cooling reservoirs that poses either a flooding risk or a low-water risk for CCNPP Unit 3. The staff determination is based on the CCNPP Unit 3 design, which would not include canals or reservoirs, but rather storage tanks that would be designed according to specifications described in the U.S. EPR FSAR Revision 4, and the UHS MWIS, which would withdraw makeup water from the Chesapeake Bay.

Based on a review of the COL applicant’s information in the COL FSAR, the staff determined that the COL applicant has appropriately considered flood-causing and low-water phenomena and their combinations that are relevant to cooling-water canals and reservoirs. Based on the reasons given above, the staff finds that the requirements of 10 CFR Part 100.20(c), as they relate to identifying and evaluating hydrological features of the site, are met. The staff agreed that the combinations of flood-causing and low-water phenomena considered by the COL applicant are appropriate for the CCNPP Unit 3 site. Therefore, based on the reasons given above, the staff finds that the requirements of 10 CFR 52.79(a)(1)(iii) are met, as they relate to the determination of hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2.4.8.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.8.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR Revision 4. On the basis of its review, the staff confirmed that the COL applicant addressed the required information related to cooling-water canals and reservoirs used to transport and impound water supplied to the SSCs important to safety. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to cooling-water canals and reservoirs used to transport and impound water supplied to the SSCs important to safety incorporated by reference in the COL FSAR will be documented in the staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will update Section 2.4.8 of this report to reflect the final disposition of the design certification application.

2.4.9 Channel Diversions

2.4.9.1 *Introduction*

COL FSAR Section 2.4.9, "Channel Diversions," addresses channel diversions. COL FSAR Section 2.4.9 also evaluates plant and essential water supplies used to transport and impound water supplies to ensure that they will not be adversely affected by stream or channel diversions. The evaluation includes stream channel diversions away from the site (which may lead to a loss of safety-related water) and stream channel diversions toward the site (which may lead to flooding). In addition, during such an event, it must be ensured that alternative water supplies are available to safety-related equipment.

This section presents an evaluation of the following specific areas: (1) Historical channel migration phenomena including cutoffs, subsidence, and uplift; (2) regional topographic evidence that suggests whether a future channel diversion may or may not occur (used in conjunction with evidence of historical diversions); (3) thermal causes of channel diversion, such as ice jams, which may result from downstream ice blockages that may lead to flooding from backwater or upstream ice blockages that can divert the flow of water away from the MWIS intake; (4) potential for forces on safety-related facilities or the blockage of water supplies resulting from channel migration-induced flooding (flooding not addressed by hydrometeorologically induced flooding scenarios in other sections); (5) potential of channel diversion from human-induced causes (i.e., land-use changes, diking, channelization, armoring, or failure of structures); (6) alternative water sources and operating procedures; (7) potential effects of seismic and non-seismic information about the postulated worst-case channel diversion scenario for the proposed plant site; and (8) any additional information requirement prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.9.2 *Summary of Application*

This section of the COL FSAR describes the channel diversions. COL FSAR Section 2.4.9 incorporates by reference U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.9. In this section, the COL applicant provides site-specific supplemental information to address the COL-specific information identified in U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-10

> A COL applicant that references the U.S. EPR design certification will provide site-specific information and demonstrate that in the event of diversion or rerouting of the source of cooling water, alternate water supplies will be available to safety-related equipment.

The COL applicant provided additional information in COL FSAR Section 2.4.9 to address COL Information Item 2.4-10, "Channel Diversions," from U.S. EPR FSAR Tier 2, Revision 4, Table 1.8-2, "U.S. EPR Combined License Information Items," which describes channel diversions. The COL applicant provided information about the following: Historical channel diversions, regional topographic evidence, ice causes, site flooding due to channel diversions, human-induced channel flooding, alternate water sources, and other site-related criteria.

2.4.9.3 *Regulatory Basis*

The relevant requirements of NRC regulations for channel diversions, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.9, "Channel Diversions."

The applicable regulatory requirements for identifying and evaluating channel diversions are set forth in the following:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.
- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water-surface elevations at the site.

The staff also used the appropriate sections of the following RGs for the related acceptance criteria identified in NUREG-0800, Section 2.4.9.

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.29, "Seismic Design Classification," as it relates to those SSCs intended to protect against the effects of flooding or those associated with the MWIS
- RG 1.59, "Design Basis Floods for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site

The regulatory basis of the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.9.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.9. The staff confirmed that the information in the COL application addresses information relevant to the channel diversions. The staff's technical review of this section included an independent review of the COL applicant's information in the COL FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data. This section describes the staff's evaluation of the technical information presented by the COL applicant in COL FSAR Section 2.4.9. The staff evaluation is described below.

Information Provided by COL Applicant

In COL FSAR Section 2.4.9, the COL applicant provided information about the following: Historical channel diversions; regional topographic evidence; ice causes; site flooding due to channel diversions; human-induced channel flooding; alternate water sources; and other site-related evaluation criteria. The COL applicant described the geological processes that formed the Chesapeake Bay, as well as the geomorphology of the Chesapeake Bay. The geomorphology over the past few centuries is primarily limited to a gradual erosion of the shoreline. Erosion and deposition in the vicinity of the MWIS are limited due to the existing CCNPP Units 1 and 2 MWIS and the barge jetty. The hill behind the location of the proposed CCNPP Unit 3 UHS MWIS is more recessed from the shoreline and possesses a more gradual slope than the cliffs above and below the CCNPP Unit 3 site.

The Staff's Technical Evaluation

The focus of the staff's review was on channel diversion events that might either (1) increase the flooding hazard at the upland nuclear power block area, (2) cause a blockage to the shoreline UHS MWIS, or (3) cause sufficient erosion around the UHS MWIS to risk failure of the MWIS intake. During the June 24–26, 2008, site audit, the staff observed the cliffs to the southeast of the barge jetty and the existing terrain in the vicinity of the proposed nuclear power block and in the vicinity of the proposed UHS MWIS intake. Obvious signs of undercutting the cliffs and bank failure were present. The steep faces of the cliffs south of the barge offloading structure showed a tendency of the cliffs to fail along near vertical failure planes.

The location of the proposed nuclear power block on a watershed divide would limit the effects of channel diversions to those from backwater caused by downstream blockages due to large-scale hill-slope failure events. Through the course of its review, the staff determined that flooding at the nuclear power block site, due to channel diversions, is implausible. This determination is based on (1) a review of the topography in the Johns Creek watershed, (2) the absence of evidence of large-scale hill-slope failures within the Johns Creek watershed, and (3) the absence of features capable of blocking Johns Creek.

To block the UHS MWIS intake, significant sediment deposition in the vicinity of that structure would need to occur. Based on the existing shoreline to the northwest of the CCNPP Units 1 and 2 intake, the existing shoreline at the location of the UHS MWIS intake between the CCNPP Units 1 and 2 intake and the barge jetty, and the existing shoreline to the southeast of the barge jetty, the staff determined that the cliffs northwest of the CCNPP Units 1 and 2 intake and to the

southeast of the barge slips would be plausible sources of large amounts of sediment. These cliffs could fail during extreme storm events. However, the staff determined that any sediment migration from either source would be significantly attenuated by the dredged area and structures associated with CCNPP Units 1 and 2 intakes, the barge jetty, and the sheet pile wall at the location of the UHS MWIS. Therefore, the staff found that blockage of the UHS MWIS intake is not a plausible concern associated with channel diversions.

The staff also noted that significant erosion in the vicinity of the UHS MWIS intake or the UHS pumping facility is not a plausible channel diversion concern. The same structures that would attenuate the deposition – the CCNPP Units 1 and 2 intake structure, barge jetty, and sheet pile wall adjacent to the intake, and the shoreline armoring – would also prevent significant shoreline erosion.

Based on a review of the COL applicant's information in the COL FSAR, and based on the reasons given above, the staff finds that the COL applicant appropriately considered channel-diverting phenomena and their combinations that are relevant for the CCNPP Unit 3 site. Therefore, based on the reasons given above, the staff finds that the requirements of 10 CFR 52.79(a)(1)(iii), 10 CFR Part 100, and 10 CFR 100.23(d), as they relate to identifying and evaluating hydrological features of the site, are met.

2.4.9.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.9.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR Revision 4. On the basis of its review, the staff confirmed that the COL applicant addressed the required information related to channel diversions. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to channel diversions incorporated by reference in the COL FSAR will be documented in the staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will update Section 2.4.9 of this report to reflect the final disposition of the design certification application.

2.4.10 Flooding Protection Requirements

2.4.10.1 *Introduction*

COL FSAR Section 2.4.10 addresses the design bases required to ensure that safety-related facilities will be capable of surviving all design flood conditions, and those of structures and components required for protection of safety-related facilities. This section also describes various types of flood protection used and the emergency procedures to be implemented where applicable.

This section provides an evaluation of the following specific areas: (1) Safety-related facilities exposed to flooding; (2) type of flood protection (e.g., "hardened facilities," flood doors, bulkheads, etc.) provided for the SSCs exposed to flooding; (3) emergency procedures needed to implement flood-protection activities and warning times available for their implementation reviewed by the organization responsible for reviewing issues related to plant emergency

procedures; (4) potential effects of seismic and non-seismic information about the postulated flood-protection for the proposed plant site; and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.10.2 *Summary of Application*

This section of the COL FSAR addresses the needs for site-specific information about flood-protection requirements. COL FSAR Section 2.4.10 incorporates by reference U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.10, "Flooding Protection Requirements." In this section, the COL applicant provided site-specific supplemental information to address COL-specific information identified in U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-11

> A COL applicant that references the U.S. EPR design certification will use site-specific information to compare the location and elevations of safety-related facilities, and of structures and components required for protection of safety-related facilities, with the estimated static and dynamic effects of the design-basis flood conditions.

The COL applicant provided additional information in COL FSAR Section 2.4.10 to address U.S. EPR FSAR Tier 2, Revision 4, Table 1.8-2, "U.S. EPR Combined License Information Items," COL Information Item 2.4.11, which describes flood-protection requirements. The COL applicant stated that flood protection is not required for the nuclear power block area. However, the COL applicant did acknowledge that the UHS MWIS would be subject to flooding and, therefore, would require flood-protection measures.

Additional flood-protection measures are described in COL FSAR Section 3.4, "Water Level (Flood) Design."

2.4.10.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the flood-protection requirements, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.10.

The applicable regulatory requirements for identifying and evaluating flood-protection requirements are as follows:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated
- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to water-surface elevations at the site.

The staff also used appropriate sections of the following RGs for the acceptance criteria identified in NUREG-0800, Section 2.4.10:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.59, "Design Basis Flood for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized
- RG 1.102, "Flood Protection for Nuclear Power Plants," as it relates to providing assurance that SSCs important to safety have been designed to withstand the effects of natural flooding phenomena likely to occur at the site
- RG 1.206, "Combined License Applications for nuclear power plants (LWR Edition)"

The regulatory basis of the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.10.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.10. The staff confirmed that the information in the COL application addresses the relevant information related to the flood-protection requirements. The staff's technical review of this section includes an independent review of the COL applicant's information in the COL FSAR. This section describes the staff's evaluation of the technical information presented by the COL applicant in COL FSAR Section 2.4.10. The staff evaluation is described below.

Information Submitted by COL Applicant

Based on the information provided in COL FSAR Sections 2.4.2 through 2.4.9, the COL applicant stated that the nuclear power block area grade is in excess of DBA flooding. Therefore, the COL applicant asserts that flood protection is not required for the power block area. However, the COL applicant does acknowledge that the UHS MWIS is subject to flooding and is subject to flood protection.

Based on the COL applicant's analysis in COL FSAR Section 2.4.5, the basis conditions for the DBA hydrostatic, hydrodynamic, and debris load are associated with a PMSS. The COL applicant provided a detailed description of the engineering features of the UHS MWIS in COL FSAR Section 3.8, "Design of Category I Structures."

The Staff's Technical Evaluation

In RAI 328, Question 02.04.10-1, the staff requested that the COL applicant provide quantified information in the COL FSAR demonstrating that the UHS system and associated safety-related pipes would be protected from adverse effects of natural phenomena, including flooding from local intense precipitation, storm surge, tsunami, ice formation, and groundwater. In a December 20, 2012, response to RAI 328, Question 02.04.10-1, the COL applicant referenced Maryland State guidance of 0.6-m (2-ft) frost line depth near the site and stated that the burial

depths of safety-related pipes would be 1.4 m (4.75 ft). The staff documented its evaluation of the adverse effects of the natural phenomena in the preceding sections of this report and finds the COL applicant's response adequate. As a result of its conclusion, the staff considers RAI 328, Question 02.04.10-1, resolved. In addition, the staff documented that the Seismic Category I design requirements (COL FSAR Sections 3.5 and 3.8) provide protection to the UHS MWIS from water-borne missiles.

2.4.10.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.10.6 *Conclusions*

The staff reviewed the COL application and confirmed that the COL applicant has addressed the information demonstrating that the characteristics of the site fall within the site parameters specified in the U.S. EPR FSAR, and no outstanding information is required to be addressed in the COL FSAR related to this section.

Based on the reasons given above, the staff finds that the COL applicant has presented and substantiated information relative to the flood protection measures important to the design and siting of this plant. Based on the reasons given above, the staff also finds that the COL applicant has considered the appropriate site phenomena to establish the flood protection measures for SSCs. The staff reviewed the COL applicant's information and, for the reasons stated above, concluded that the COL applicant, as documented in Section 2.4.10 of this report, has provided sufficient details about the site description to allow the staff to evaluate whether the COL applicant has met the relevant requirements of 10 CFR 52.79(a)(1) and 10 CFR Part 100, with respect to determining the acceptability of the site.

2.4.11 Low Water Considerations

2.4.11.1 *Introduction*

COL FSAR Section 2.4.11, "Low Water Considerations," addresses natural events that may reduce or limit the available safety-related cooling-water supply.

This section provides an evaluation of the following specific areas: (1) Low-water conditions due to the worst drought considered reasonably possible in the region; (2) effects of low water-surface elevations caused by various hydrometeorological events and a potential blockage of MWIS intakes by sediment, debris, littoral drift, and ice because they can affect the safety-related water supply; (3) effects of low water on the MWIS and pump design bases in relation to the events described in Sections 2.4.5, 2.4.6, 2.4.7, 2.4.8, 2.4.9, and 2.4.11 of this report, which consider the range of water-supply required by the plant (including minimum operating and shutdown flows during anticipated operational occurrences and emergency conditions) compared with availability (considering the capability of the UHS to provide adequate cooling water under conditions requiring safety-related cooling); (4) use limitations imposed or under discussion by Federal, State, or local agencies authorizing the use of the water; (5) potential effects of seismic and non-seismic information about the postulated worst-case low-water scenario for the proposed plant site; and (6) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.11.2 *Summary of Application*

This section of the COL FSAR addresses the effects of low water on safety-related water supply. COL FSAR Section 2.4.11 incorporates by reference U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.11. In this section, the COL applicant provided site-specific supplemental information to address the COL-specific information identified in U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-12

> A COL applicant that references the U.S. EPR design certification will identify natural events that may reduce or limit the available cooling-water supply, and will verify that an adequate water supply exists for operation or shutdown of the plant in normal operation, anticipated operational occurrences, and in low-water conditions.

The COL applicant provided additional information in COL FSAR Section 2.4.11 to address U.S. EPR FSAR Tier 2, Revision 4, Table 1.8-2, "U.S. EPR Combined License Information Items," COL Information Item 2.4-12, which describes the effects of low water. The COL applicant stated that CCNPP Unit 3 would rely on the broader Chesapeake Bay water body for its safety-related water supply and, therefore, low-flow conditions in rivers and streams are not relevant. The COL applicant also stated that the effect of seiches on the CCNPP Unit 3 site is negligible and low-water conditions resulting from seiches would have no effect on the plant. The COL applicant stated that the effect of a PMH would produce a negative surge, rundown, and setdown associated with the PMH that would be higher than the lower design water-surface elevation in the forebay and for the UHS makeup water pumps. The COL applicant estimated that the extreme low-water-surface elevation associated with PMSS would be below the minimum design elevation at the MWIS channel for about 24 hours or less and, therefore, concluded that an alternative emergency UHS makeup source under PMH low-water conditions would not be necessary.

The COL applicant reported that, because there are no recorded tidal data at the site, data from two nearby tide stations were used to determine the low-water-surface elevation. In addition, the COL applicant stated that there are no future controls for the Chesapeake Bay that could affect water availability from the Chesapeake Bay and water-surface elevations there.

2.4.11.3 *Regulatory Basis*

The relevant requirements of NRC regulations for low-water considerations, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.11.

The applicable regulatory requirements for identifying the effects of low water are set forth in the following:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).
- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used the appropriate sections of the following RGs for the acceptance criteria identified in NUREG-0800, Section 2.4.11:

- RG 1.27, "Ultimate Heat Sink for Nuclear Power Plants," as it relates to providing high assurance that the water sources relied on for the sink will be available where needed
- RG 1.59, "Design Basis Flood for Nuclear Power Plants," as supplemented by best current practices, as it relates to providing assurance that natural flooding phenomena that could potentially affect the site have been appropriately identified and characterized.

The regulatory basis for the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.11.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.11. The staff confirmed that the information in the COL application addresses the relevant information related to the low-water considerations. The staff's technical review of this section includes an independent review of the COL applicant's information in the COL FSAR and in the responses to the RAIs. The staff supplemented this information with other publicly available sources of data.

This section describes the staff's evaluation of the technical information presented in COL FSAR Section 2.4.11.

2.4.11.4.1 Low Flow in Rivers and Streams

Information Submitted by COL Applicant

The COL applicant stated that CCNPP Unit 3 would rely on the Chesapeake Bay for safety-related water supply and, therefore, low-flow conditions in rivers and streams are not relevant. The COL applicant stated that, because the Chesapeake Bay is physically connected to the Atlantic Ocean, drought conditions in the drainage basins of rivers and streams flowing into it would not affect the bay's water-surface elevation significantly. The COL applicant stated that the low-water-surface elevations in the Chesapeake Bay are related to tides, storm surges, and tsunami events.

The Staff's Technical Evaluation

The staff reviewed the COL applicant's information in the COL FSAR. Based on the hydrology of the Chesapeake Bay, and based on the reasons given above, the staff finds that the low-water conditions in the bay would only be affected by tides, storm surges, and tsunamis. Based on the review of seiches in the Chesapeake Bay (see Section 2.4.5 of this report), the staff also concluded that seiches in the Chesapeake Bay can result in low-water-surface elevations.

2.4.11.4.2 Low Water Resulting from Surges, Tides, and Tsunamis

Information Submitted by COL Applicant

The COL applicant stated that the effect of seiches on the CCNPP Unit 3 site is negligible and that resulting low-water conditions would have no effect on the plant. The COL applicant stated that extreme negative surges would occur at the CCNPP Unit 3 site when a hurricane travels close to the Atlantic coastline. The COL applicant stated that such a hurricane path would be the most critical, because the strength of the hurricane would not likely be diminished by its passage over land. The COL applicant reported historical negative surge data for several hurricanes based on a compilation of hurricanes through 1960.

The COL applicant reviewed Pore (1960) and reported that the lowest negative surges at Baltimore, Annapolis, and Solomons Island, MD over the period 1929–1958 were -0.98 m (-3.2 ft), -0.73 m (-2.4 ft), and -0.43 m (-1.4 ft), respectively. The COL applicant stated that since 1960, two hurricanes passed by the CCNPP Unit 3 site − Hurricane Gloria in September 1985 and Hurricane Emily in August 1993. The COL applicant stated that the annual minimum water-surface elevations recorded at Annapolis and Solomons Island NOAA stations do not correspond to these two hurricanes. Consequently, the COL applicant selected Hurricane Donna, which occurred in September 1960, as the typical hurricane to estimate negative surge in the Chesapeake Bay. The COL applicant stated that the maximum sustained wind speed during Hurricane Donna at Cove Point and Lookout Point, which are 9.7 and 43.5 km (6 and 27 mi) south of the CCNPP Unit 3 site, respectively, was 88.5 km/hr (57 mph or 50 knots (kn)). The COL applicant stated that, because the winds rotate in a counterclockwise direction in a hurricane, the wind direction would change from northeast to north as the hurricane travels past the Chesapeake Bay. The COL applicant stated that the northerly winds would drive the water toward the south in the Chesapeake Bay creating low-water conditions near the CCNPP Unit 3 site. The COL applicant estimated the negative surge at the site from Hurricane Donna as -0.37 m (-1.2 ft) by interpolation between Annapolis and Solomons Island stations.

The COL applicant stated that the lowest water-surface elevation due to wind effects would occur during a PMH that travels in a path similar to the one described above. The COL applicant used the procedure described in USACE Engineering Manual 1110-2-1412 (USACE 1986) to estimate the PMH parameters. The COL applicant estimated that the location of the hurricane would be latitude 37 degrees north, a corresponding K factor for wind speed in kilometers per hour would be 68.8 (km/hr)(kPa$^{-0.5}$) (78.7 mph (in. of Hg$^{-0.5}$)), and the Coriolis parameter f would be 0.315 per hour. The COL applicant estimated the radius of maximum winds of the PMH to vary from 18.53 km (11.51 mi or 10 nautical mi) to 48.52 km (30.15 mi or 26.2 nautical mi). The COL applicant estimated the central and peripheral pressures of the hurricane as 67.46 and 76.5 cm of Hg (26.56 and 30.12 in. of Hg), respectively. The COL applicant estimated the maximum sustained PMH wind speed at the CCNPP Unit 3 site as 165.6 km/hr (102.9 mph). The COL applicant used an empirical equation based on the assumption that storm surges are generally proportional to the square of wind speeds. The equation used by the COL applicant was

$$\frac{W_{Donna}^2}{NS_{CCNPP,Donna}} = \frac{W_{PMH}^2}{NS_{CCNPP,PMH}}$$

where W^2_{Donna} and W^2_{PMH} denote the winds speeds for Hurricane Donna and the PMH, respectively, and $NS_{CCNPP, Donna}$ and $NS_{CCNPP, PMH}$ denote the negative storm surges for Hurricane Donna and the PMH, respectively. The COL applicant estimated that the negative surge at the CCNPP Unit 3 site from the PMH would be −1.2 m (−3.9 ft). The COL applicant stated that an additional westerly wind-induced setdown would also occur coincidentally. The COL applicant assumed that the setdown would be equal to the setup from PMH winds estimated previously in COL FSAR Section 2.4.5. Therefore, the COL applicant reported a total PMH setdown of −1.53 m (−5.03 ft), which includes the PMH negative surge of -1.2 m (−3.9 ft) and the PMH setdown of −0.34 m (−1.13 ft).

The COL applicant stated that the minimum water-surface elevation at the CCNPP Unit 3 site from a tsunami-caused drawdown is estimated to be -0.5 m (-1.64 ft). The COL applicant stated that tsunami drawdown effects would be less than those from a PMH drawdown. The COL applicant estimated the combined low-water-surface elevation as the sum of effects of the MLLW and the negative surge. The COL applicant estimated the datum at Cove Point to be 0.3 cm (0.1 in.) below MLLW. Therefore, the COL applicant estimated the lowest water-surface elevation at the CCNPP Unit 3 site as the sum of MLLW and total PMH setdown, which is -1.53 m (-5.02 ft).

The Staff's Technical Evaluation

The staff reviewed the COL applicant's descriptions of the historical negative surges and negative surge caused by a PMH. The staff noted that a PMH passing by the Chesapeake Bay, close to the Atlantic shoreline, would create northerly winds that can result in significant setdown near the CCNPP Unit 3 site. The staff also reviewed the PMH parameters estimated by the COL applicant for a PMH near the Chesapeake Bay. The staff concluded that the COL applicant's PMH parameters are reasonable.

However, the staff did not agree with the COL applicant's use of the empirical equation to predict the negative storm surge during a PMH. The staff determined that the COL applicant needed to simulate the storm surge from a PMH passing by the Chesapeake Bay in the Atlantic Ocean using a dynamic storm surge simulation model to ensure that the low-water-surface elevation had been conservatively estimated. The staff also determined that the COL applicant needed to account for conservative antecedent water-surface elevations in the Chesapeake Bay that may occur in combination with the PMH event to ensure that conservative coincident events are accounted for. Therefore, in RAI 288, Question 02.04.11-1, the staff requested that the COL applicant revise the COL FSAR accordingly and provide an updated estimate of low-water conditions during a PMSS event that (1) accounts for antecedent water levels appropriate for low-water conditions, (2) uses a conservative approach such as a storm surge model (e.g., SLOSH) with input from appropriate PMH scenarios, and (3) accounts for any concurrent wind-wave activity. In a May 4, 2011, response to RAI 288, Question 02.04.11-1, the COL applicant stated that a 10 percent exceedance low tide was estimated to be −0.8 m (−2.6 ft). The COL applicant used this antecedent water-surface elevation in the SLOSH computer code, and determined a maximum negative surge of −2.0 m (−6.5 ft). The COL applicant applied 20 percent model prediction uncertainty and concluded that the total PMSS would be −2.2 to −1.7 m (−7.3 to −5.7 ft). In the RAI response, the COL applicant described modifications to be made in COL FSAR Section 2.4.11.5. The staff finds the COL applicant's response acceptable and verified that the modifications related to RAI 288, Question 02.04.11-1, were implemented in COL FSAR Revision 9. Accordingly, the staff considers RAI 288, Question 02.04.11-1 resolved.

The staff's review of tsunami effects is described in Section 2.4.6 of this report where a maximum PMT-induced drawdown elevation estimate of −0.3 m (−0.8 ft) is described. The staff determined that tsunami-induced drawdown at the CCNPP Unit 3 site is smaller (less severe) than that induced by a PMH (−2.0 m (−6.5 ft)). The COL applicant stated that the minimum design operating elevation of the two new intake pipes would be set at -1.8 m (-6 ft) for the safety-related UHS MWIS. Since the design operating elevation of the MWIS intake pipes would be higher than the PMSS drawdown elevation, the staff concluded that it is plausible that during the PMSS event, the MWIS intakes could be inoperable during the period of time the water-surface elevation near the MWIS intake is lower than the design operating elevation. However, the staff determined that a conservative estimate of the duration that the MWIS intakes might be inoperable would be provided by the time it would take for the PMH to pass over the CCNPP site, at an orientation that causes a drawdown in the Chesapeake Bay near the CCNPP UHS MWIS. The staff estimated this time to be twice the PMH radius to maximum winds divided by the PMH forward speed. The staff estimated that the duration of drawdown exceeding the MWIS intake pipes' minimum design operating elevation would be 3.6 hours. The staff determined that this conservatively estimated time would not affect the safety of the plant, because the plant would have a 72-hour emergency water supply and the water-surface elevations in the Chesapeake Bay near the UHS MWIS would be restored to normal elevations in less than 4 hours.

2.4.11.4.3 Historical Low Water

Information Submitted by COL Applicant

The COL applicant stated that because there are no tidal data at the site, data from two nearby tide stations were used to determine the low-water-surface elevation. The COL applicant used tide stations at Annapolis, and Solomons Island, MD, which are 59.5 km (37 mi) to the north and 12.9 km (8 mi) south of the CCNPP Unit 3 site, respectively. The COL applicant used eight different probability density functions to analyze the tidal data and determined that none of the distributions were an accurate fit for return periods exceeding 10 years. Therefore, the COL applicant estimated the 100-year low-water-surface elevation by visual inspection of plotted data. The COL applicant reported that the 100-year low-water-surface elevation is 0.2 m (0.5 ft) above station datum at Annapolis and 0.1 m (0.4 ft) above station datum at Solomons Island.

The COL applicant stated that it conservatively selected the 100-year low-water-surface elevation based on the observations recorded at the Annapolis station, which is lower than those recorded at the Solomons Island station. The COL applicant reported that the 100-year low-water elevation is -1.2 m (-3.9 ft) and that the minimum operating elevation of the non-safety-related circulating water system MWIS intake is set to be -1.2 m (-4 ft).

The Staff's Technical Evaluation

The staff reviewed the COL applicant's description of historical low water near the CCNPP Unit 3 site and confirmed that the COL applicant addressed the required information related to RAI 288, Question 02.04.11-1. As stated above in Section 2.4.11.4.2, while the most severe and longest-lasting PMSS drawdown could render the UHS MWIS intake inoperative for a duration of less than 4 hours, there would be no adverse effect to the safety of CCNPP Unit 3.

2.4.11.4.4 Future Controls

Information Submitted by COL Applicant

The COL applicant stated that there are no planned future controls for the Chesapeake Bay that could affect water availability from the Chesapeake Bay and Chesapeake Bay water-surface elevations.

The Staff's Technical Evaluation

The Chesapeake Bay is approximately 280 km (175 mi) long. Since the Chesapeake Bay is physically connected to the Atlantic Ocean, the staff concluded that the water-surface elevation in the bay would not be affected by future man-made controls on the bay, but it would be affected by tides, storm surges, seiches, and tsunamis. As stated above in Section 2.4.11.4.2, while the most severe and longest-lasting PMSS drawdown could render the UHS MWIS intake inoperable for a duration of less than 4 hours, there would be no adverse effect to the safety of CCNPP Unit 3.

2.4.11.4.5 Plant Requirements

Information Submitted by COL Applicant

The COL applicant stated that, 72 hours after a DBA, emergency makeup water to the cooling-tower basin is provided by the safety-related UHS emergency makeup water pumps located in the UHS MWIS. The COL applicant also stated that under DBA conditions, makeup water would be required at a maximum rate of 3,566 lpm (942 gallons per minute (gpm)). The COL applicant stated that the amount of water withdrawn from the Chesapeake Bay would be permitted by the State of Maryland.

The COL applicant stated that the minimum design operating elevation of the two new intake pipes is set at -1.8 m (-6 ft) for the safety-related UHS MWIS intake.

The Staff's Technical Evaluation

The staff reviewed the information provided by the COL applicant. Since the Chesapeake Bay is connected to the Atlantic Ocean, which is virtually an unlimited source of water, the staff concluded that the water supply from the Chesapeake Bay would be sufficient for plant requirements.

As stated above in Section 2.4.11.4.2, while the most severe and longest-lasting PMSS drawdown could render the UHS MWIS intake inoperable for a duration of less than 4 hours, there would be no adverse effect to the safety of CCNPP Unit 3.

2.4.11.4.6 Heat Sink Dependability Requirements

Information Submitted by COL Applicant

The COL applicant stated that, 72 hours after a DBA, emergency makeup water is provided to the cooling-tower basin by the safety-related UHS emergency makeup water pumps located in the UHS MWIS. The COL applicant also stated that the invert elevation of the UHS makeup

pump sump is set at an elevation to provide sufficient depth below the water surface to prevent vortex formation while maintaining sufficient net positive suction head.

The COL applicant stated that the CCNPP Unit 3 site would be out of the severe-influence area of a PMH in about 24 hours, because the PMH would be moving at a forward speed of 32.7 km/hr (20.3 mph). The COL applicant also stated that even though the CCNPP Unit 3 site may not experience the PMH conditions for an extended duration, the minimum design elevation of the pipes would be set at -1.83 m (-6 ft) based on the PMH-caused low-water condition.

The Staff's Technical Evaluation

The staff reviewed the information provided by the COL applicant. CCNPP Unit 3 would employ four mini UHS mechanical cooling towers that would contain 72 hours' worth of UHS cooling water. The reservoirs for the UHS towers will draw water from the Chesapeake Bay for water makeup due to evaporative losses. Since the Chesapeake Bay is physically connected to the Atlantic Ocean, which is virtually an unlimited source of water, the staff concluded that the water supply from the Chesapeake Bay would be sufficient for plant requirements. As stated above in Section 2.4.11.4.2, while the most severe and longest-lasting PMSS drawdown could render the UHS MWIS intake inoperable for a duration of less than 4 hours, there would be no adverse effect on the safety of CCNPP Unit 3.

2.4.11.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.11.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR Revision 4. On the basis of its review, the staff confirmed that the COL applicant addressed the required information related to potential for low-water conditions. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to potential for low-water conditions incorporated by reference in the COL FSAR will be documented in the staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will update Section 2.4.11 of this report to reflect the final disposition of the design certification application.

2.4.12 Groundwater

2.4.12.1 *Introduction*

This section describes the hydrogeological characteristics of the site. One of the key objectives of groundwater investigations and monitoring at this site is to evaluate the maximum groundwater-surface elevation at the site, which is used in Section 2.5 of this report to determine the effects of groundwater on the stability of the plant foundations and slopes. The evaluation is performed to ensure that the maximum groundwater-surface elevation remains less than the COL FSAR site parameter value of 1 m (3.3 ft) below plant grade. Other significant objectives are to examine whether groundwater provides any safety-related water supply, to determine whether dewatering systems are required to maintain groundwater-surface

elevations below the required elevation, and to describe subsurface pathways for potential groundwater contaminants.

The specific areas of review are as follows: (1) Identification of the aquifers, types of onsite groundwater use, sources of recharge, present withdrawals and known and likely future withdrawals, flow rates, travel time, gradients, and other properties that affect movement of accidental contaminants in groundwater, groundwater-surface elevations beneath the site, seasonal and climatic fluctuations, monitoring and protection requirements, and man-made changes that have the potential to cause long-term changes in local groundwater regime; (2) effects of groundwater-surface elevations and other hydrodynamic effects of groundwater on design bases of plant foundations and those of other SSCs important to safety; (3) reliability of groundwater resources and related systems used to supply safety-related water to the plant; (4) reliability of dewatering systems to maintain groundwater conditions within the plant's design bases; (5) potential effects of seismic and non-seismic information about the postulated worst-case groundwater conditions for the proposed plant site; and (6) any additional information requirements prescribed within the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.12.2 *Summary of Application*

COL FSAR Section 2.4.12, "Groundwater," incorporates by reference U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.12, "Groundwater." This section of the COL FSAR addresses the groundwater in terms of effects on structures and water supply.

In addition, in COL FSAR Section 2.4.12, the COL applicant addressed COL Information Item 2.4-13, "Groundwater," identified in U.S. EPR FSAR Tier 2, Revision 4, Table 1.8-2 and U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.12.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-13

> A COL applicant that references the U.S. EPR design certification will provide site-specific information to identify local and regional groundwater reservoirs, subsurface pathways, onsite use, monitoring or safeguard measures, and to establish the effects of groundwater on plant structures.

The COL applicant addressed this COL information item by including in the COL FSAR the following information as a supplement to U.S. EPR FSAR Revision 4:

- The COL applicant described the regional and local groundwater aquifers, formations, sources, and sinks. The COL applicant stated that the CCNPP site is underlain by approximately 762 m (2,500 ft) of sedimentary deposits of primarily silt, clay, sand, and gravel. The principal hydrogeologic units at the site were identified by the COL applicant (from shallow to deep) as the surficial aquifer, the Chesapeake confining unit, the Piney Point-Nanjemoy aquifer, the Aquia aquifer, the Magothy aquifer, and the Upper and Lower Patapsco aquifers. Two thin, semi-continuous, water-bearing sand units were identified by the COL applicant in the upper part of the Chesapeake confining unit. The COL applicant referred to these units as the Upper Chesapeake and Lower Chesapeake units in COL FSAR Section 2.4.12. The resulting three lower-permeability portions of

the Chesapeake confining unit were referred to in COL FSAR Section 2.4.12 as the Upper, Middle, and Lower Chesapeake aquitards.

- The COL applicant described future groundwater use at the plant. The COL applicant stated that new groundwater wells in the Aquia, Upper Patapsco, or Lower Patapsco aquifers would supply water for plant construction and for plant operation when the desalination plant is out of service. Yearly groundwater appropriation currently cannot exceed 379 m^3/d (100,000 gpd) on average. The maximum monthly use cannot exceed a daily average of 681 m^3/d (180,000 gpd).
- The COL applicant described the present and projected future regional water use and characterized the groundwater flow field. The COL applicant stated that no sole source aquifers lie within the regional hydrogeologic system containing the CCNPP site. The primary aquifers used for water supply in Calvert and St. Mary's counties are the Piney Point-Nanjemoy and the Aquia aquifers.
- The COL applicant provided an analysis of groundwater pathways in the event of a liquid effluent release at the site. Soil physical and hydraulic properties of the hydrogeologic units of interest were reported based on the results of laboratory and field testing conducted as part of the CCNPP Unit 3 field investigation. Minimum, maximum, and geometric mean values for saturated hydraulic conductivity were reported in COL FSAR Table 2.4-41, "CCNPP Unit 3 Observation Wells – Hydraulic Conductivities from Slug Tests."
- The COL applicant described and discussed monitoring programs to be used to protect present and potential future groundwater users.
- The COL applicant described the site characteristics, including the maximum operational groundwater-surface elevation, for groundwater-induced hydrostatic loadings on subsurface portions of safety-related SSCs. The COL applicant developed and applied a series of groundwater flow models of the CCNPP Unit 3 site to evaluate post-construction groundwater-surface elevations. The COL applicant provided a detailed description of a model of the surficial aquifer, the Upper and Lower Chesapeake units, and the intervening low-permeability layers of the Chesapeake confining unit in Bechtel Calculation No. 25237-000-30R-GEK-00002, Revision 000, "Groundwater Model for the Calvert Cliffs Nuclear Power Plant Unit 3 Site," (ML101300094 and ML101300095). These model results supported the evaluation of site characteristics for subsurface hydrostatic loading and dewatering provided by the COL applicant in COL FSAR Section 2.4.12.5.

2.4.12.3 *Regulatory Basis*

The relevant requirements of the Commission regulations for the groundwater, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.12.

The applicable regulatory requirements for groundwater are set forth in the following:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

The staff also used the acceptance criteria identified in NUREG-0800, Section 2.4.12:

- Local and Regional Groundwater Characteristics and Use: The applicant should supply a complete description of regional and local groundwater characteristics and groundwater use, groundwater monitoring and protection requirements, and any man-made changes with a potential to affect regional groundwater characteristics over a long period of time.

- Effects on Plant Foundations and other Safety-Related Structures, Systems, and Components: The applicant should supply a complete description of the effects of groundwater-surface elevations and other hydrodynamic effects on the design bases of plant foundations and other SSCs important to safety.

- Reliability of Groundwater Resources and Systems Used for Safety-Related Purposes: The applicant should supply a complete description of all SSCs important to safety that depends on groundwater, as well as data and analysis regarding the reliability of the groundwater source.

- Reliability of Dewatering Systems: The applicant should supply a complete description of the site dewatering system, including its reliability to maintain the groundwater conditions within the groundwater design bases of SSCs important to safety.

- Consideration of Other Site-Related Evaluation Criteria: The applicant should supply an assessment of the potential effects of seismic and non-seismic information about the postulated worst-case scenario related to groundwater effects for the proposed plant site.

The regulatory basis of the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.12.4 *Technical Evaluation*

The staff reviewed COL FSAR, Revision 9, Sections 2.4.12 and 2.5.4 ("Stability of Subsurface Materials and Foundations"), the Geotechnical Subsurface Investigation Data Report (Schnabel Engineering 2007), and other documents and calculations provided by the COL applicant. The staff also reviewed other relevant documents as referenced below. The staff confirmed that the information contained in the COL application and incorporated by reference addresses the relevant information related to this section.

To improve readability, the staff's discussion of groundwater characteristics is organized by the technical areas below.

2.4.12.4.1 Regional and Local Hydrogeological Description

Information Submitted by COL Applicant

The COL applicant described the regional and local hydrogeology in COL FSAR Section 2.4.12.1, "Description and Use." The description of the regional hydrogeology provided by the COL applicant was largely summarized from USGS *Groundwater Atlas of the United States* (Trapp and Horn 1997). The site is located in the Coastal Plain Physiographic Province. The sediments in this physiographic province range in thickness from 0 m (0 ft) at the Fall Line to 8,000 m (5 mi) at the Maryland Atlantic coast, generally thickening to the southeast. The principal aquifers of the North Atlantic Coastal Plain aquifer system are (from shallow to deep) the surficial, Chesapeake, Castle Hayne-Aquia, Severn Magothy, and Potomac aquifers, dipping east to southeast. The COL applicant stated that, in southern Maryland, the surficial and Chesapeake aquifers receive localized recharge, while the deeper aquifer units are recharged in outcrop areas to the west and northwest well beyond the CCNPP site.

The COL applicant stated that the CCNPP Unit 3 site is underlain by approximately 762 m (2,500 ft) of sedimentary deposits, consisting primarily of silt, clay, sand, and gravel. The COL applicant's description of the aquifer system beneath the site differs from the regional system described by Trapp and Horn (1997) in that the Chesapeake aquifer is treated primarily as a confining unit at the CCNPP Unit 3 site (referred to as the Chesapeake confining unit in the COL FSAR) and the Castle Hayne-Aquia aquifer is subdivided into the Piney Point-Nanjemoy aquifer and the underlying Aquia aquifer, which are separated by sandy clay and clay formations referred to collectively as the Nanjemoy confining unit. At the CCNPP Unit 3 site, two thin, semi-continuous, water-bearing sand units were identified by the COL applicant in the upper part of the Chesapeake confining unit. The COL applicant referred to these units as the Upper Chesapeake and Lower Chesapeake units in COL FSAR Section 2.4.12. The resulting three lower-permeability portions of the Chesapeake confining unit were referred to in COL FSAR Section 2.4.12 as the Upper, Middle, and Lower Chesapeake aquitards.

A geotechnical and hydrogeological investigation of the CCNPP Unit 3 site was described in COL FSAR Section 2.5.4. The COL applicant conducted this investigation in two phases: The first between April and August 2006, and the second between May and December 2008. The Phase I investigation included drilling 145 boreholes with standard penetration test sampling and collecting over 200 soil samples to a maximum depth of 123 m (403 ft), installing 40 groundwater monitoring wells to a maximum depth of 37.2 m (122 ft), and completing a slug test in each well. The COL applicant completed laboratory testing of the geotechnical properties of soil samples and the chemical characteristics of well-water samples. The COL applicant measured water-surface elevations in the wells monthly from July 2006 through June 2007. Borings and sample collection extended through the surficial aquifer and into the Chesapeake confining unit. Groundwater observation wells were completed by the COL applicant in the surficial aquifer and the Upper and Lower Chesapeake units.

In RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11, staff requested that the COL applicant provide additional discussion of the groundwater modeling conducted to evaluate the maximum post-construction groundwater-surface elevation. The staff was interested in the degree of conservatism in the model analysis and a description of additional modeling that the COL applicant had referenced in the modeling report and at the site audit. In Enclosure 1 of a September 10, 2010, response to RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11, the COL applicant provided information regarding Phase II of the COL site

investigation and stated that seven new wells were installed and monitored on a monthly basis from September 2008 through October 2009, and quarterly monitoring was conducted from that point forward. The COL applicant also stated that the 40 groundwater wells completed in 2006 were monitored quarterly from September 2007 through October 2009. Well construction data and groundwater-surface-elevation monitoring observations for the period July 2006 to October 2009 for all 47 wells were included in Enclosure 1 of the RAI response and subsequently incorporated in COL FSAR, Revision 7.

The surficial aquifer was described by the COL applicant as consisting primarily of fine- to medium-grained sands and silty or clayey sands. The COL applicant stated that the surficial aquifer is present at the CCNPP site above an elevation of 9.8 to 25.0 m (32 to 82 ft) with a thickness ranging from 0.3 to 20.7 m (1 to 68 ft) and an average thickness of 86.4 m (21 ft) at the CCNPP Unit 3 nuclear power block area and 8.5 m (28 ft) across the entire site as shown in COL FSAR Table 2.5-28, "Summary of Field Tests." In RAI 101, Question 02.04.12-2, the staff requested that the COL applicant provide resolution of discrepancies between COL FSAR Sections 2.4.12 and 2.5.4 in the description of hydrogeologic units. In a July 2, 2009, response to RAI 101, Question 02.04.12-2, the COL applicant indicated that the average thickness of the surficial aquifer is about 8.8 m (29 ft) as shown in Table 2.4.12-1 below. The surficial aquifer was referred to in COL FSAR Section 2.5.4 as Stratum I – Terrace Sand. The COL applicant noted there, that a sandy soil suspected to be fill, was observed in 25 borings, mainly in areas that had been previously developed.

In COL FSAR Section 2.4.12, the COL applicant described the Chesapeake confining unit as consisting of primarily silty clays, silt, and fine-grained sands. The base of the Chesapeake confining unit was observed by the COL applicant in two boreholes at an elevation of -62.5 and -65.5 m (-205 and -215 ft). The COL applicant stated that the thickness of this unit was 85.3 and 84.4 m (280 and 277 ft) in these boreholes and that the thickness of the Chesapeake confining unit was less where the surficial aquifer sediments were absent.

In COL FSAR Section 2.5.4, the COL applicant referred to the Chesapeake soils as the Stratum IIa − Chesapeake Clay/Silt, Stratum IIb – Chesapeake Cemented Sand, and Stratum IIc − Chesapeake Clay/Silt. The COL applicant described Strata IIa and IIc as consisting of primarily clay and silt, with the lower stratum (IIc) containing glauconitic sediments and interbedded layers of sandy silt, silty sand, and cemented sands. The COL applicant described Stratum IIb as predominantly sandy soil with interbedded layers of silty/clayey sands, sandy silts, and clays, with varying amounts of shell fragments and degree of cementation. Stratum IIb was subdivided by the COL applicant into three layers; the upper and lower layers generally had higher densities as determined by standard penetration test blow counts.

In the July 2, 2009, response to RAI 101, Question 02.04.12-2, the COL applicant clarified the relationship between the stratigraphic units discussed in COL FSAR Sections 2.4.12 and 2.5.4 and also resolved discrepancies between the reported thicknesses and elevations of the units. The COL applicant stated that the remaining differences in bottom elevations and unit thicknesses were due to differences in interpretations by the COL applicant's geotechnical and hydrogeologic staff.

In the July 2, 2009, response to RAI 101, Question 02.04.12-2, and a June 12, 2009, response to RAI 101, Question 02.04.12-5, the COL applicant revised COL FSAR Figures 2.4-74, "'Cross-Section A-A' Through Proposed Unit 3 Power Block Area," and 2.4-75, "Cross-Section B-B' Through Proposed Unit 3 Power Block Area," so that the geologic cross

sections were consistent with the stratigraphic column descriptions, thicknesses, and bottom elevations discussed in the COL FSAR text.

The Staff's Technical Evaluation

Based on a review of Trapp and Horn (1997) and Maryland Geological Survey (Drummond 2007), the staff concluded that the COL applicant's description of the regional hydrogeology was accurate. In evaluating local site hydrogeology, the staff reviewed the data provided in COL FSAR Sections 2.4.12 and 2.5.4 and in the Geotechnical Subsurface Investigation Data Report (Schnabel Engineering 2007). The staff identified a number of inconsistencies between the discussions in COL FSAR Sections 2.4.12 and 2.5.4, which were resolved by responding to RAI 101, Questions 02.04.12-2 and 02.04.12-5, as described above. A table of stratigraphic column descriptions, bottom elevations, and average unit thicknesses was provided by the COL applicant in response to RAI 101, Question 02.04.12-2. An adaptation of that table, which compares the hydrogeologic description of stratigraphic units from COL FSAR Section 2.4.12 and the geotechnical description from COL FSAR Section 2.5.4 Table 2.5-27, "Summary Thickness and Termination Elevation," is presented as Table 2.4.12-1 below. After verifying that the groundwater transport analysis in COL FSAR Section 2.4.13 was updated to reflect the revised hydrogeological description of stratigraphic units, the staff concluded that the local and regional hydrogeological description was complete. Accordingly, the staff considers RAI 101, Question 02.04.12-2 resolved.

Table 2.4.12-1. Stratigraphic Column Correlating Hydrogeologic and Geotechnical Units from COL FSAR 2.4.12 and 2.5.4*

Average Thickness [m] (ft)	Average Bottom Elevation [m] (ft)	Hydrogeologic Unit	Geotechnical Unit	Average Bottom Elevation [m] (ft)	Average Thickness [m] (ft)
8.8 (28.8)	18.6 (61)	Surficial aquifer	Terrace Sand (Stratum I)	18.6 (61)	8.5 (28)
6.1 (20)	12.5 (41)	Upper Chesapeake aquitard	Chesapeake Clay/Silt (Stratum IIa)	13.1 (43)	5.8 (19)
13.7 (45)	-1.5 (-5)	Upper Chesapeake unit	Chesapeake Cemented Sand (Stratum IIb)	-6.7 (-18)	19.2 (63)
3.4 (11.2)	-4.9 (-16)	Middle Chesapeake aquitard			
10.7 (35)	-15.5 (-51)	Lower Chesapeake unit	Chesapeake Clay/Silt (Stratum IIc)	-64.3 (-211)	58.8 (193)
51.8 (170)	-64 (-210)	Lower Chesapeake aquitard			
>33 (>10.1)	< -97 (-60)	Piney Point-Nanjemoy aquifer	Nanjemoy Sand (Stratum III)	**	>33 (>108)

Notes:
* Values are approximate
** Boring did not penetrate into confining unit below

2.4.12.4.2 Onsite Groundwater Use

Information Provided by COL Applicant

In RAI 101, Question 02.04.12-3, the staff requested that the COL applicant clarify the CCNPP Unit 3 groundwater use projections. In a June 12, 2009, response to RAI 101, Question 02.04.12-3, and in subsequent revisions of COL FSAR Section 2.4.12.1.4, "CCNPP Unit 3 Ground Water Use Projections," the COL applicant stated that freshwater for operation of CCNPP Unit 3 would come solely from a desalination plant using Chesapeake Bay water. The COL applicant stated that water needed for pre-construction and construction purposes would be obtained from new wells drilled into the Aquia, Upper Patapsco, or Lower Patapsco aquifers. Yearly groundwater appropriation currently cannot exceed 379 m^3/d (100,000 gpd) on average. The maximum monthly use cannot exceed a daily average of 681 m^3/d (180,000 gpd). The COL applicant stated that additional water sources would be required to meet the construction requirement of a daily average of 1,089.0 m^3 (287,333 gal) for the month of maximum use. The COL applicant stated that its preferred source for the additional water is effluent from dewatering the surficial aquifer but that this would require a revision to the State withdrawal permit.

The COL applicant also stated that approximately 3.41 m^3/min (900 gpm or 1.3 million gallons per day (Mgd)) of groundwater would be needed during plant operation when the desalination plant is out of service for repair and maintenance (a period of time estimated to last no more than 10 weeks). The source of this water was not specified in the COL FSAR. In RAI 400, Question 02.04-2, the staff requested that the COL applicant clarify the source of this water and identify any subsidence issues that could be caused by the associated groundwater withdrawal to support this need. **RAI 400, Question 02.04-2 is being tracked as an open item.**

The Staff's Technical Evaluation

The staff reviewed the information provided in the COL FSAR about current and projected onsite groundwater use. The staff determined that the groundwater to be used during periods when the desalination plant is out of service is not safety-related. The water source for the UHS is the Chesapeake Bay.

2.4.12.4.3 Regional Groundwater Use and Flow-Field Characterization

Information Submitted by COL Applicant

The COL applicant summarized the regional and local groundwater use in COL FSAR Section 2.4.12.2, "Sources." The COL applicant stated that no sole source aquifers lie within the regional hydrogeologic system containing the CCNPP site. The primary aquifers used for water supply in Calvert and St. Mary's counties are the Piney Point-Nanjemoy and the Aquia aquifers. The COL applicant stated that in Calvert County, the Aquia aquifer is the primary source for large water users (mostly public and private community water suppliers); whereas, the Piney Point-Nanjemoy aquifer is being increasingly reserved for domestic use. A relatively small proportion of groundwater in Calvert County is withdrawn from the Magothy and Patapsco aquifers. The COL applicant stated that groundwater withdrawals permitted in Calvert County totaled 20 ML/d (5.3 Mgd) in 2006. Since the COL applicant was unable to accurately determine the location of known groundwater users, the COL applicant assumed that the nearest permitted groundwater supply well would be adjacent to the southeastern boundary of

the CCNPP site and that the nearest community water well system would be located at the closest boundary of the nearest community (Lusby). The COL applicant determined these locations based on the direction of groundwater flow in the Aquia aquifer.

Of the 47 groundwater observation wells installed by the COL applicant as part of the CCNPP Unit 3 field investigation and the Supplemental COL Investigation, 19 wells were completed in the surficial aquifer, 23 wells were completed in the Upper Chesapeake unit, and 5 wells were completed in the Lower Chesapeake unit. The COL applicant installed eight two-well clusters to evaluate vertical hydraulic gradients: Four in the surficial aquifer and the Upper Chesapeake unit, and four in the Upper and Lower Chesapeake units.

The COL applicant stated that during the period from July 2006 to October 2009, groundwater-surface elevations in the surficial aquifer wells were observed to fluctuate an average of 1.4 m (4.7 ft), with a maximum fluctuation of 3.0 m (9.8 ft). The COL applicant described a northwest trending groundwater divide in the surficial aquifer extending through the southwest corner of the proposed nuclear power block area (e.g., COL FSAR Figure 2.4-80, "Water Table Elevation Map and Groundwater Flow Direction for the Surficial Aquifer, December 2006"). The COL applicant stated that groundwater in the surficial aquifer likely discharges via small (artesian) seeps and springs either to the northeast or the southwest of the groundwater divide. The COL applicant stated that horizontal hydraulic gradients varied across the site with a range given of 0.0086 to 0.015 and that groundwater heads in the well clusters were 10.0 to 13.1 m (33 to 43 ft) higher in the surficial aquifer than in the Upper Chesapeake unit, which indicated downward flow between these units.

The COL applicant stated that during the period of observations from July 2006 to October 2009, groundwater-surface elevations in the Upper Chesapeake unit wells fluctuated an average of 1.7 m (5.4 ft), with a maximum fluctuation of 3.9 m (12.8 ft). The COL applicant stated that the groundwater flow direction was generally to the north and east, with groundwater discharging to the lower reaches of the Branch 1 and Branch 2 streams, to seeps and springs where the Upper Chesapeake unit outcrops (including along the face of Calvert Cliffs), and possibly directly to Chesapeake Bay. The COL applicant described a possible groundwater divide along the southwestern boundary of the nuclear power block area that may result in groundwater flow toward Johns Creek and Branch 3. See Figure 2.4.1-2 of this report for further description of the stream branches. The COL applicant stated that horizontal hydraulic gradients varied across the CCNPP site with a given range of near zero in the southwest corner of the site to between 0.0091 and 0.017 around the nuclear power block area. The COL applicant stated that groundwater heads in the well clusters were 0.2 m (0.6 ft) to 1.7 m (5.4 ft) higher in the Upper Chesapeake unit than in the Lower Chesapeake unit and that this indicated downward flow between these units.

The COL applicant stated that groundwater-surface elevations fluctuated an average of 1.1 m (3.6 ft) in the Lower Chesapeake unit monitoring wells during the period from July 2006 to October 2009; the maximum fluctuation was 2.1 m (7.0 ft). Although only three to five wells were available to determine flow direction and gradient, the COL applicant suggested that groundwater flow was generally to the north-northeast, and groundwater likely discharged directly to the Chesapeake Bay. The COL applicant stated that the average horizontal hydraulic gradient in the Lower Chesapeake unit was 0.014.

The Staff's Technical Evaluation

The staff evaluated the recent and projected regional groundwater use and the regional groundwater flow-field by reviewing Drummond (2007), Wolman (2008), and groundwater-surface elevation data in the *USGS National Water Information System* (http://waterdata.usgs.gov/nwis) from wells located in Calvert County. The staff confirmed that the Piney Point and Aquia aquifers are the primary sources of groundwater supply in the region around the CCNPP site and that groundwater-surface elevations in the regional aquifers are generally decreasing as a result of groundwater pumping.

In RAI 101, Question 02.04.12-3, the staff requested that the COL applicant provide details regarding the potential effect of future onsite and offsite groundwater use on plant safety (e.g., subsidence). In the June 12, 2009, response to RAI 101, Question 02.04.12-3, the COL applicant described results from Drummond (2007) of the simulated drawdown in the Piney Point-Nanjemoy and Aquia aquifers under a variety of projected water-use scenarios for the period 2002 to 2030. Under the highest use scenario (Scenario 2b, which assumed a 20 percent increase over 2002 levels for all groundwater pumping in Calvert, Charles, and St. Mary's counties), the projected additional drawdown was less than 6 m (20 ft) in the Piney Point-Nanjemoy aquifer and approximately 15 m (50 ft) in the Aquia aquifer. The staff noted that the simulated drawdown exceeded the estimated preconsolidation stress of -20 m (-65.6 ft) only in the Aquia aquifer. The preconsolidation stress is the threshold groundwater head for the occurrence of significant subsidence due to groundwater withdrawal. The COL applicant also provided results from a simulation model of the Aquia aquifer within 8.0 km (5 mi) of the CCNPP Unit 3 site. The COL applicant used this model to estimate the drawdown resulting from an increase in pumping associated with construction of CCNPP Unit 3. According to the COL applicant's model, 6 years of withdrawing an additional 15.1 L/s (346,000 gpd) produced an additional drawdown of 15.8 m (52 ft) in the groundwater-surface elevation at the CCNPP Unit 3 site. The COL applicant stated that the model showed that groundwater-surface elevations recovered within approximately 3 years after the additional pumping ceased. The COL applicant provided a bounding estimate of 3.7 mm of subsidence per meter of drawdown (0.044 in. of subsidence per foot of drawdown) based on an estimate of maximum subsidence near Lexington Park (Achmad and Hansen 1997). This is a location south of the CCNPP site and across the Patuxent River where a groundwater cone of depression exists in the Castle Hayne-Aquia aquifer (Drummond 2007). Given the bounding estimate of subsidence, the COL applicant concluded that 15.8 m (51.8 ft) of drawdown would result in a maximum of 58 mm (2.3 in.) of subsidence at the CCNPP Unit 3 site.

Although Drummond (2007) states that subsidence due to groundwater withdrawal has not been documented in Maryland, Davis (1987) concluded that subsidence resulting from declining heads in confined aquifers is widespread throughout the Atlantic Coastal Plain. Davis (1987) documents subsidence at six sites in Georgia, Virginia, Delaware, and New Jersey with subsidence ranging from 1.9 to 3.7 mm/m (0.023 to 0.044 in./ft) of groundwater head decline. Based on this evidence, the staff concluded that the COL applicant's bounding estimate of subsidence at the CCNPP site (3.7 mm of subsidence per meter of head decline) is consistent with observed subsidence resulting from long-term declines in groundwater heads and that a subsidence of 58 mm (2.3 in.) is bounding. On this basis, the staff finds the COL applicant's June 12, 2009, response to RAI 101, Question 02.04.12-3, acceptable. Accordingly, the staff considers RAI 101, Question 02.04.12-3 resolved.

In RAI 101, Question 02.04.12-4, the staff requested that the COL applicant provide additional data referred to in the COL FSAR for groundwater head observations made between July 2006 and June 2007, although observations presented in COL FSAR Table 2.4-35, "CCNPP Unit 3 Observation Wells Water Level Elevations," only extended through March 2007. In a May 20, 2009, response to RAI 101, Question 02.04.12-4, the COL applicant provided additional measurements of groundwater heads during the months of April, May, and June 2007. The staff finds the COL applicant's June 12, 2009, response to RAI 101, Question 02.04.12-4, acceptable. Accordingly, the staff considers RAI 101, Question 02.04.12-4 resolved. As described above in Section 2.4.12.4.1, the COL applicant provided additional groundwater head measurements for the period from July 2007 to October 2009 in Enclosure 1 of the September 10, 2010, response to RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11. These measurements were reviewed and used by the staff in the evaluation of site characteristics and groundwater pathways.

In RAI 101, Question 02.04.12-6, the staff requested that the COL applicant provide a discussion of the potential effect of temporal variability in head on the estimated groundwater velocities and travel times. In a June 12, 2009, response to RAI 101, Question 02.04.12-6, the COL applicant provided estimates of the variability in horizontal hydraulic gradient over the period of head measurements by examining the gradients in the surficial aquifer and the Upper and Lower Chesapeake units during a month of high head and a month of low head in each unit. The COL applicant reported that the hydraulic gradient varied from 0.01 to 0.0104 in the surficial aquifer, from 0.0144 to 0.0174 in the Upper Chesapeake unit, and from 0.0115 to 0.0152 in the Lower Chesapeake unit. The staff evaluated the temporal variability of the horizontal groundwater gradient in the surficial aquifer for the period of observations from July 2006 to October 2009, and noted a somewhat greater variability in gradients than reported by the COL applicant. The gradient calculated by the staff in the surficial aquifer using observed heads at wells OW-423 and OW-752A varied from 0.0093 to 0.0116, as shown below in Figure 2.4.12-1. The monthly horizontal groundwater gradient in the Upper Chesapeake unit was calculated by staff using observed heads at well OW-301 (at the CCNPP Unit 3 Reactor Building location) and well OW-703A (located to the north, near Branch 2), which corresponds to one of the subsurface transport pathways analyzed in COL FSAR Section 2.4.13. The observed groundwater gradient between these wells varied from 0.0141 to 0.0178, as shown in Figure 2.4.12-1.

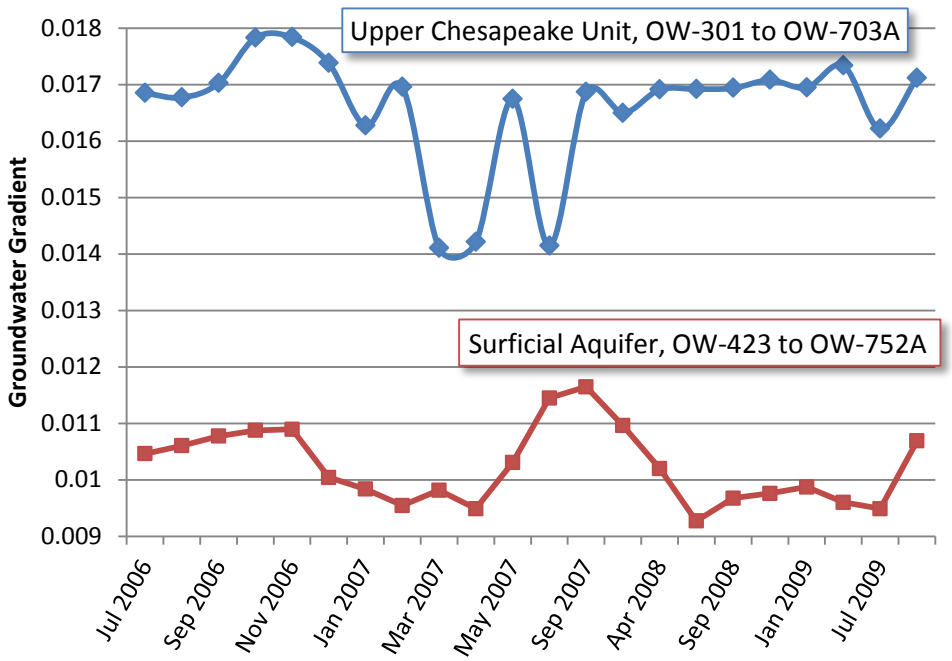

Figure 2.4.12-1. Monthly Groundwater Gradient Estimated between Observation Wells OW-301 and OW-703A in the Upper Chesapeake Unit and OW-423 and OW-752A in the Surficial Aquifer

The staff evaluated the CCNPP Unit 3 site groundwater hydraulic head data and the associated contour maps of head in the surficial aquifer and the Upper and Lower Chesapeake units presented in the COL FSAR. The staff finds the contour maps consistent with the site data and appropriate for evaluating subsurface pathways at the CCNPP Unit 3 site. The staff finds the COL applicant's June 12, 2009, response to RAI 101, Question 02.04.12-6, acceptable. Accordingly, the staff considers RAI 101, Question 02.04.12-6 resolved.

2.4.12.4.4 Subsurface Pathways

Information Submitted by COL Applicant

The COL applicant reported physical and hydraulic properties for the uppermost water-bearing units based on the results of laboratory and field testing conducted as part of the CCNPP Unit 3 field investigations. The COL applicant reported minimum, maximum, and geometric mean values for saturated hydraulic conductivity in COL FSAR, Table 2.4-41, "CCNPP Unit 3 Observation Wells – Hydraulic Conductivities from Slug Tests," which are given below in Table 2.4.12-2 (the arithmetic mean is given for the Lower Chesapeake unit). The COL applicant stated in COL FSAR, Revision 6, Table 2.4-38, "CCNPP Unit 3 Aquifer Unit Geotechnical Parameters," that porosity values were calculated from laboratory geotechnical data and that effective porosity was assumed to be 80 percent of porosity. The resulting values calculated by the COL applicant are given in Table 2.4.12-2 of this report. In COL FSAR revisions provided in Enclosure 1 of the COL applicant's September 10, 2010, response to RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11, and subsequently incorporated in COL FSAR Revision 7, the COL applicant deleted COL FSAR Table 2.4-38 and described a different methodology for estimating effective porosities based on median grain size and information from

literature sources. The revised effective porosity values provided by the COL applicant are given below in Table 2.4.12-2.

Table 2.4.12-2. Physical and Hydraulic Properties of CCNPP Water-Bearing Units and Estimated Groundwater Velocities Reported by the COL Applicant

PARAMETER, UNITS		Water-Bearing Unit		
		Surficial aquifer	Upper Chesapeake	Lower Chesapeake
Average Effective Porosity (COL FSAR Revision 6)		0.341	0.370	0.412
Average Effective Porosity (COL FSAR Revision 7)		0.139	0.145	0.156
Hydraulic Conductivity, m/d (ft/d)	**Min**	0.012 (0.039)	0.036 (0.118)	0.006 (0.020)
	Max	5.30 (17.39)	4.18 (13.71)	0.028 (0.092)
	Avg	0.277 (0.909)	0.226 (0.741)	0.0137 (0.0449)
Hydraulic Gradient		0.011	0.017	0.014
Groundwater Velocity, m/d (ft/d)		0.022 (0.072)	0.026 (0.085)	0.0012 (0.0039)

Average groundwater velocities were computed by the COL applicant using average hydraulic gradients, hydraulic conductivities, and the effective porosities. Groundwater velocities as calculated by the COL applicant in Enclosure 1 of the COL applicant's September 10, 2010, response to RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11, are shown below in Table 2.4.12-2.

The COL applicant identified five potential subsurface pathways based on the observed groundwater heads. The staff determined that these pathways characterize the potential groundwater transport under existing conditions (i.e., prior to construction). The COL applicant calculated travel times for all pathways, which are shown in Table 2.4.12-3 of this report. The COL applicant calculated surficial aquifer travel time from the center of the groundwater divide located in the proposed nuclear power block area to the estimated discharge point at Branch 3. The COL applicant calculated Upper and Lower Chesapeake unit travel times from the center of the nuclear power block area to the COL applicant's estimated discharge points at Branch 2 and the Chesapeake Bay shoreline, respectively. Two additional pathways in the Upper Chesapeake unit were considered by the COL applicant, from a point south of the nuclear power block area to discharge points in Branch 1 and the Chesapeake Bay. The minimum travel time for all potential pathways considered by the COL applicant was 15 years for the Upper Chesapeake pathway with discharge to Branch 2.

Table 2.4.12-3. Travel Distance and Estimated Travel Time for Subsurface Pathways Reported by the COL Applicant for Existing Conditions

Pathway	Discharge Point	Travel Distance, m (ft)	Travel Time, yr
Surficial aquifer	Branch 3	400.8 (1,315)	50
Upper Chesapeake	Branch 2	143.3 (470)	15
	Branch 1	338.3 (1,110)	35
	Chesapeake Bay	513.6 (1,685)	53
Lower Chesapeake	Chesapeake Bay	469.4 (1,540)	1,050

The COL applicant also calculated travel times for potential groundwater transport pathways that characterize post-construction conditions. These pathways are discussed in Section 2.4.13 of this report and were based on hydraulic gradients the COL applicant computed from the five-layer groundwater model described in Section 2.4.12.4.6 of this report.

The Staff's Technical Evaluation

The staff reviewed the hydraulic and geotechnical data presented in COL FSAR Sections 2.4.12 and 2.5.4, and in the COL applicant's geotechnical report (Schnabel Engineering 2007). The method to estimate effective porosity described in Enclosure 1 of the COL applicant's September 10, 2010, response to RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11, was based on results from a calibrated numerical flow and transport code using data reported for a single field study that was not a well-controlled experiment. The staff concluded that extrapolating these results to the CCNPP site is not justified. However, the effective porosity values reported in the COL applicant's September 10, 2010, response to RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11, were approximately 40 percent of the values given in COL FSAR, Revision 6, Table 2.4-38, "CCNPP Unit 3 Aquifer Unit Geotechnical Parameters." The staff noted that the lower values of effective porosity increase the groundwater velocities and therefore provide conservative estimates of advective travel times along the groundwater pathways. The staff finds the methods used to evaluate the laboratory and field data appropriate and the physical and hydraulic property values presented in COL FSAR Section 2.4.12, Revision 7, consistent with the data.

The staff evaluated the potential subsurface pathways from the nuclear power block area based on the site hydraulic head data and head contour maps presented in the COL FSAR. The staff concluded that the pathways described by the COL applicant are plausible representations of conditions prior to construction of Unit 3. The staff also concluded that groundwater velocities and travel times calculated by the COL applicant may not be bounding because of the use of hydraulic conductivity values based on averages computed from all boreholes within a hydrogeologic unit. However, the COL applicant used a more conservative approach to calculate travel times for the radionuclide transport analysis in COL FSAR Section 2.4.13. These pathways and travel times are evaluated by the staff in Section 2.4.13 of this report.

To evaluate the potential for an alternative transport pathway to the Piney Point-Nanjemoy aquifer, in RAI 101, Question 02.04.12-5, the staff requested that the COL applicant provide a description of the water budget at the CCNPP Unit 3 site, including a three-dimensional conceptual description of groundwater flow within and between the surficial aquifer, the Chesapeake units, and the Piney Point-Nanjemoy aquifer. In a June 12, 2009, response to

RAI 101, Question 02.04.12-5, the COL applicant provided information about the vertical hydraulic gradients between the water-bearing units at the CCNPP site, a general water budget for the site, and the potential for a vertical transport pathway to the Piney Point-Nanjemoy aquifer. The COL applicant's estimate of recharge to the surficial aquifer was about 13 cm/yr (5 in./yr), based on calibration of a groundwater model of the aquifer discussed in COL FSAR Section 2.4.12.5, "Site Characteristics for Subsurface Hydrostatic Loading and Dewatering." This value is about 11 percent of the annual average precipitation of 112 cm/yr (44 in./yr). The COL applicant estimated vertical flow between water-bearing units using groundwater heads observed in adjacent well pairs completed at different depths. These wells indicate generally downward groundwater flow.

The COL applicant also provided estimates for the vertical hydraulic conductivity of the Chesapeake unit in the June 12, 2009, response to RAI 101, Question 02.04.12-5, and in COL FSAR Section 2.4.12.3.2.2, "Chesapeake Group." These estimates ranged from 2.6×10^{-6} meters per day (m/d) to 7.6×10^{-3} m/d (8.6×10^{-6} ft/d to 2.5×10^{-2} ft/d). Using an average value of 3×10^{-5} m/d (10^{-4} ft/d) for the vertical hydraulic conductivity of the Chesapeake unit confining layers, the COL applicant estimated vertical fluxes across these units, concluding that there is a potential pathway from the surficial aquifer to the Piney Point-Nanjemoy aquifer with an estimated flux of 5.6×10^{-6} m/d (1.7×10^{-6} ft/d) between the Lower Chesapeake unit and the Piney Point-Nanjemoy aquifer (i.e., the COL applicant estimated that about 1.6 percent of the recharge to the surficial aquifer reaches the Piney Point-Nanjemoy aquifer). The COL applicant estimated that this flux across the CCNPP site would be diluted 5.5 to 11 times by the groundwater flow in the Piney Point-Nanjemoy aquifer.

The staff reviewed the COL applicant's estimate of the site water budget and vertical flow through the Chesapeake unit. Drummond (2007) used a range of 1.5×10^{-5} m/d to 6.1×10^{-4} m/d (5×10^{-5} ft/d to 2×10^{-3} ft/d) for the vertical hydraulic conductivity of the Chesapeake unit. The staff determined that the high end of this range is inconsistent with recharge estimates and the observed head loss across the Chesapeake unit at the CCNPP site and likely representative of the more permeable portions of the unit. Based on the reasons given above, the staff finds that a vertical hydraulic conductivity of approximately 1.8×10^{-4} m/d (6×10^{-4} ft/d) is an acceptable, alternative (conservative) value for the confining materials of the Chesapeake unit.

Using this value and information provided in COL FSAR Section 2.4.12 and the June 12, 2009, response to RAI 101, Question 02.04.12-5, the staff evaluated dilution factors and travel times for a vertical pathway at the CCNPP Unit 3 site. The staff conservatively assumed instantaneous mixing over the depth of the water-bearing units. In calculating dilution factors, the staff assumed the contaminant to be mixed across the entire nuclear power block area. The staff used a value of 0.06 for the effective porosity of the confining materials of the Chesapeake unit, as assumed by the COL applicant in COL FSAR Section 2.4.12.3.3, "Ground Water Flow and Transport." The staff estimated that a contaminant introduced in the surficial aquifer would be diluted by a factor of 34 in the Piney Point-Nanjemoy aquifer with a travel time of 277 years. A contaminant introduced in the Upper Chesapeake unit would be diluted by a factor of 12 in the Piney Point-Nanjemoy aquifer with a travel time of 274 years. The staff concluded that the COL applicant's estimate of dilution for a vertical subsurface pathway (as documented in the June 12, 2009, response to RAI 101, Question 02.04.12-5) is bounding and that the horizontal subsurface pathway in the Upper Chesapeake unit is the bounding pathway with respect to travel time. Based on this conclusion, the staff considers RAI 101, Question 02.04.12-5 resolved.

2.4.12.4.5 Groundwater Monitoring and Safeguard Requirements

Information Submitted by COL Applicant

The COL applicant initially provided a brief, non-specific discussion of groundwater monitoring plans during construction and operation. In RAI 101, Question 02.04.12-7, the staff requested that the COL applicant provide specific details of the anticipated groundwater monitoring programs during CCNPP Unit 3 construction and operation, including monitoring objectives, monitoring locations, what quantities will be measured, and the frequency of monitoring. In a July 2, 2009, response to RAI 101, Question 02.04.12-7, and the September 10, 2010, response to RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11, the COL applicant described the ongoing groundwater-surface-elevation monitoring in the 47 existing monitoring wells (through October 2009).

The COL applicant stated that of the 47 existing wells, 38 will be sealed (plugged) and abandoned prior to construction, because they are located in areas that will be re-graded during construction of CCNPP Unit 3. The COL applicant identified the nine wells that will continue to be monitored quarterly during construction to evaluate changes in water-surface elevations. The COL applicant stated that 29 new wells would be completed prior to operation of CCNPP Unit 3: 11 well pairs installed in the surficial aquifer and the Upper Chesapeake unit; 2 well pairs installed in the Upper and Lower Chesapeake units; and 1 well triplet monitoring all three water-bearing units. The COL applicant stated that quarterly groundwater-surface-elevation monitoring will be conducted during CCNPP Unit 3 operation, and some of the wells will be used to monitor groundwater quality. The COL applicant provided a map showing the locations of existing wells to be retained during construction, the proposed post-construction locations of new wells to be constructed, and wells designated for radiological monitoring in the CCNPP Unit 3 radiological environmental monitoring program. The COL applicant updated COL FSAR Section 2.4.12.4, "Monitoring or Safeguard Requirements," to reflect the additional information contained in the response to RAI 101, Question 02.04.12-7.

The Staff's Technical Evaluation

The staff reviewed the information provided in the July 2, 2009, response to RAI 101, Question 02.04.12-7, regarding groundwater monitoring programs. The staff recognizes that groundwater monitoring is an ongoing activity, and that monitoring wells will need to be closed and new wells installed because site access conditions change during construction. The staff agrees that further evaluation and the installation of new wells will be necessary to ensure that groundwater-surface elevations will be adequately monitored as site conditions change. Based on the description in the July 2, 2009, response to RAI 101, Question 02.04.12-7, the staff finds the proposed location of new wells and the frequency of monitoring during operation of CCNPP Unit 3 a reasonable approach for monitoring the effects of construction and natural variations in groundwater-surface elevations during plant operation. The staff finds the COL applicant's July 2, 2009, response to RAI 101, Question 02.04.12-7, acceptable. Accordingly, the staff considers RAI 101, Question 02.04.12-7 resolved.

The COL applicant stated in COL FSAR Section 3.8.4.6.1, "Materials," that groundwater from the surficial aquifer in the nuclear power block area is considered aggressive to concrete because of its low pH. In RAI 144, Question 03.08.04-12, the staff requested a description of the inservice inspection program for below-grade concrete foundations subjected to aggressive soil conditions. In a July 23, 2010, response to RAI 144, Question 03.08.04-12, the COL

applicant described a monitoring system of risers and drain sumps inside the waterproofing materials to confirm that groundwater is removed from around the concrete structures. The COL applicant also stated that groundwater in the surrounding nuclear power block area will be monitored to ensure that unprotected concrete is not exposed to low pH groundwater. In response to RAI 265, Question 02.04.12-13, the COL applicant included this monitoring information in COL FSAR Section 2.4.12.5, "Site Characteristics for Subsurface Hydrostatic Loading and Dewatering."

2.4.12.4.6 Effects of Groundwater on Plant Structures

Information Submitted by COL Applicant

The COL applicant stated that the completed surface-grade elevation at the site is expected to range from 21.9 to 25.9 m (72 to 85 ft). The COL applicant indicated in COL FSAR Figure 2.4-7 that the surface elevation around the buildings in the nuclear power block area ranges from about 24.4 m to 25.8 m (80 ft to 84.7 ft). The U.S. EPR FSAR Revision 4 requires that the maximum groundwater-surface elevation be at least 1.0 m (3.3 ft) below grade for safety-related structures. The COL applicant developed and applied groundwater flow models of the CCNPP Unit 3 site to evaluate post-construction groundwater-surface elevations. The COL applicant used results from a single-layer model of the surficial aquifer as the basis for the discussion of site characteristics for subsurface hydrostatic loading and dewatering in COL FSAR Section 2.4.12.5, Revision 6.

In response to staff RAIs (described below), the COL applicant provided a description of a new groundwater flow model that included the surficial aquifer, the Upper and Lower Chesapeake units, and the intervening low-permeability layers of the Chesapeake confining unit (the Upper and Middle Chesapeake aquitards). This model also provided a more detailed representation of the building structures and foundations. The model was documented in Bechtel Calculation No. 25237-000-30R-GEK-00002, Revision 000, "Groundwater Model for the Calvert Cliffs Nuclear Power Plant Unit 3 Site," (ML101300094 and ML101300095) and was included in COL FSAR, Revision 7, Section 2.4.12.5.

The Staff's Technical Evaluation

In the May 20, 2009, response to RAI 101, Question 02.04.12-8 , the COL applicant provided an electronic copy of the Visual MODFLOW input files used in the single-layer groundwater modeling discussed in COL FSAR, Revision 6, Section 2.4.12.5. Using these input files, the staff reviewed the single-layer groundwater model of the surficial aquifer. As a result of that review, the staff's earlier concerns were addressed. The staff finds the COL applicant's May 20, 2009, response to RAI 101, Question 02.04.12-8, acceptable. Accordingly, the staff considers RAI 101, Question 02.04.12-8 resolved. Results from the staff's evaluation of the single-layer model showed that the predicted depth to groundwater was less than 1 m (3.3 ft) below ground surface in parts of the nuclear power block area and was very close to the U.S. EPR FSAR Revision 4 maximum groundwater-surface-elevation requirement (within 1 m) over a relatively large area. The staff subsequently requested that the COL applicant provide additional information regarding its groundwater modeling approach in RAI 101, Question 02.04.12-8. Specifically, in RAI 101, Question 02.04.12-9, the staff requested that the COL applicant provide details regarding the degree of conservatism of the model results and the reliability of meeting the U.S. EPR FSAR Revision 4 requirement for maximum groundwater-surface elevation. Next, in RAI 101, Question 02.04.12-10, the staff requested that the COL applicant explain model

results that showed areas around the nuclear power block where the saturated thickness of the surficial aquifer was zero, and to account for building foundations that could potentially raise the water table. Lastly, in RAI 101, Question 02.04.12-11, the staff requested that the COL applicant describe the post-construction effects on the Upper Chesapeake unit groundwater-surface elevations.

In a May 3, 2010, response to RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11, the COL applicant described a new five-layer groundwater flow model documented in Enclosure 5 of the response (Bechtel Calculation No. 25237-000-30R-GEK-00002, Revision 000, "Groundwater Model for the Calvert Cliffs Nuclear Power Plant Unit 3 Site" (ML101300094 and ML101300095). Using results from this model, the COL applicant concluded that the post-construction water table elevation in the nuclear power block area was less than 18.3 m (60 ft). The COL applicant also determined that the subsurface pathway for a radionuclide release from the bottom of the Nuclear Auxiliary Building depended on the value of the structural fill hydraulic conductivity. For a hydraulic conductivity of 10^{-3} cm/s (2.8 ft/d), travel time through the Upper Chesapeake unit to the Chesapeake Bay was more than 22 years. For a hydraulic conductivity of 10^{-2} cm/s (28 ft/d), travel time through the fill material to Branch 2 was less than 1 year.

The staff reviewed the groundwater modeling report provided in the May 3, 2010, response to RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11. The staff reviewed the implementation of the model using the model input files provided by the COL applicant. The staff verified that the model files reproduced the results provided in the groundwater modeling report. The staff concluded that the groundwater model provided an improved representation of the hydrogeology, the building foundations, and surface conditions (including recharge). In follow-up RAI 265, Question 02.04.12-13, the staff requested additional information about the technical basis for omitting from the groundwater model the lean concrete described in COL FSAR Section 2.5.4.5.2, "Extent of Excavations, Fills, and Slopes," and the waterproofing materials described in the July 23, 2010, response to RAI 144, Question 03.08.04-4. Also, in RAI 265, Question 02.04.12-13, the staff requested that the COL applicant provide a reference supporting the values of hydraulic conductivity used for the structural fill material in the groundwater model.

In a November 12, 2010, response to RAI 265, Question 02.04.12-13, the COL applicant clarified the extent of the foundation mud mat (lean concrete) and waterproofing features. Since these features are located adjacent to the concrete foundations and do not extend significantly beyond the foundations, the staff concluded that the lean concrete and waterproofing materials would have little to no effect on the post-construction groundwater flow model results. In the November 12, 2010, response to RAI 265, Question 02.04.12-13, the COL applicant also provided the data and methodology used to establish the range of fill material hydraulic conductivity used in the groundwater modeling. The staff verified the methodology and accepted the result. Accordingly, the staff considers RAI 265, Question 02.04.12-13 resolved.

As described in COL FSAR Section 2.5.4.5, "Excavation and Backfill," and as shown in the excavation profiles of COL FSAR Figures 2.5-149 to 2.5-153, excavation in the nuclear power block area is to Stratum IIb, which corresponds to the Upper Chesapeake unit and the Middle Chesapeake aquitard as shown in Table 2.4.12-1 of this report. In the five-layer groundwater model provided by the COL applicant, exposure of the relatively high-conductivity Upper Chesapeake unit in the excavation resulted in a significant lowering of the estimated groundwater-surface elevations beneath and surrounding the nuclear power block area

buildings. As noted previously, the geotechnical investigation identified three layers within the Stratum IIb unit, the uppermost of which was of relatively high density. As illustrated in the excavation profiles (COL FSAR Figures 2.5-149 to 2.5-153), it is the uppermost layer that is in direct contact with the fill materials. The staff was concerned that the relatively high density of the uppermost layer could indicate greater cementation and lower hydraulic conductivity than assumed for the Upper Chesapeake unit, and that this could have an effect on groundwater-surface elevations in the nuclear power block area.

The staff completed confirmatory simulations to evaluate the effect on groundwater-surface elevations of the hydraulic connection between the excavation fill materials and the Upper Chesapeake unit. The staff modified the COL applicant's groundwater model to include a sublayer in the top of the Upper Chesapeake unit. This sublayer was assigned a hydraulic conductivity of 4×10^{-5} cm/s (0.1 ft/d), the minimum hydraulic conductivity measured in the Upper Chesapeake unit, as shown in COL FSAR Table 2.4-41. Results from this simulation showed that the maximum water table elevation adjacent to buildings within the excavation area was approximately 21.3 m (70 ft) and occurred at the Turbine Building. (Recall that the surface elevation around the buildings in the nuclear power block area ranges from about 24.4 m to 25.8 m((80 ft to 84.6 ft).) Maximum groundwater-surface elevation must be at least 1.0 m (3.3 ft) below grade for safety-related structures. The water table was below the foundation of Essential Service Water Building 4 (ESWB 4) and less than 18.3 m (60 ft) at all safety-related buildings. A more conservative instance of this simulation was also completed by reducing the hydraulic conductivity of the Upper Chesapeake unit sublayer by an order of magnitude in a region beneath and surrounding the excavation. The staff noted that results were similar; the groundwater-surface elevation was less than 18.3 m (60 ft) at all safety-related buildings.

The staff also modified the COL applicant's groundwater model to include a continuous layer of Upper Chesapeake aquitard material (Stratum IIa) between the fill materials of the excavation and the Upper Chesapeake unit (Stratum IIb). This approach was conservative in that the hydraulic conductivity used for the clay/silt material of Stratum IIa, 5×10^{-8} cm/s (1×10^{-4} ft/d), was three orders of magnitude smaller than the minimum hydraulic conductivity measured in the Upper Chesapeake unit. Under these conditions, the model produced groundwater-surface elevations in the excavation that were no higher than 22.9 m (75 ft) adjacent to the Turbine Building. The highest groundwater-surface elevation at a safety structure occurred at ESWB 4 at an elevation of about 21.3 m (70 ft). The staff also evaluated the additional effect on groundwater-surface elevations of reducing the hydraulic conductivity of the fill material from 10^{-3} cm/s (2.8 ft/d) to 10^{-4} cm/s (0.28 ft/d). With this lower hydraulic conductivity and the presence of a low-conductivity layer between the fill and the Upper Chesapeake unit, the maximum groundwater-surface elevation at a safety structure occurred at ESWB 4 at an elevation of 21.6 m (71 ft).

The confirmatory simulations indicated that groundwater-surface elevations in the excavated area are sensitive to the hydraulic conductivity of the materials in the uppermost layer in contact with the excavation fill materials. With sufficiently low hydraulic conductivity in the uppermost layer, the staff noted that groundwater-surface elevations in the excavation exceeded the COL applicant's site characteristic value of 18.3 m (60 ft). However, this required hydraulic conductivity values that were lower than is consistent with the Upper Chesapeake unit site characterization data provided in the COL FSAR. With hydraulic conductivity in the uppermost sublayer set to plausible low values (the minimum measured value over most of the sublayer and one-tenth of the minimum under and surrounding the excavation), heads in the excavation were less than 18.3 m (60 ft) adjacent to all safety-related structures. In all cases evaluated by

the staff, groundwater-surface elevations in the excavation were below the U.S. EPR FSAR Revision 4 requirement for maximum groundwater-surface elevation.

Based on the evaluation of site characterization data and the confirmatory analyses described above, the staff concluded that 18.3 m (60 ft) is a conservative estimate of the maximum post-construction groundwater-surface elevation adjacent to safety-related structures in the nuclear power block area. Accordingly, the staff considers RAI 101, Questions 02.04.12-9, 02.04.12-10, and 02.04.12-11 resolved.

Regarding seismic effects, NUREG-0800, Section 2.4.12 states that seismic criteria should be evaluated to determine whether they should be used in postulating worst-case groundwater effects at a site. In COL FSAR Section 2.5, "Geology, Seismology, and Geotechnical Engineering," the COL applicant submitted information about seismic risks and the potential effects of earthquakes on structures and foundations. In COL FSAR Section 2.5.4.6, "Groundwater Conditions," the COL applicant discussed groundwater conditions in relation to construction and foundation stability.

The COL applicant did not submit specific information about the potential effects of seismic effects on worst-case groundwater conditions. The staff reviewed the available literature on seismic effects on groundwater-surface elevations (e.g., Montgomery and Manga 2003; Wang and Manga 2010; Roeloffs 1996; Bredehoeft 1967) and considered CCNPP Unit 3 site-specific conditions. Groundwater-surface elevations in the nuclear power block area are determined by the water table elevation of the uppermost, unconfined aquifer. The unconfined aquifer is relatively thin (the combined thickness of the surficial aquifer and Upper Chesapeake unit is about 23 m (75 ft)) and the porosity is not small (minimum estimated porosity is at least 0.15), both factors that tend to reduce the response of an unconfined aquifer to seismic waves. The staff considered the design earthquakes given in COL FSAR Table 2.5-21, "Mean Magnitudes and Distances from Deaggregations," and used the idealized unconfined aquifer analysis of Bredehoeft (1967) to estimate a maximum increase in groundwater-surface elevation of 12 cm (0.8 in.). The staff concluded that no plausible scenarios present conditions in which seismic events could have significant effects on groundwater-surface elevations at this site.

2.4.12.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.12.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR Revision 4. On the basis of its review, the staff confirmed that the COL applicant addressed the required information related to groundwater. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to groundwater incorporated by reference in the COL FSAR will be documented in the staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will update Section 2.4.12 of this report to reflect the final disposition of the design certification application.

2.4.13 Accidental Releases of Radioactive Liquid Effluent in Ground and Surface Waters

2.4.13.1 *Introduction*

This section provides a characterization of the attenuation, retardation, dilution, and concentrating properties governing transport processes in the surface-water and groundwater environment at the site. This section's goal is not to provide an assessment of the effects of a specific release scenario but to provide a suitable conceptual model of the hydrological environment for other assessments. Because it would be impractical to characterize all the physical and chemical properties (e.g., hydraulic conductivities, porosity, mineralogy) of a time-varying and heterogeneous environment, the section characterizes the environment in terms of the projected transport of a postulated release of radioactive waste. The accidental release of radioactive liquid effluents in ground and surface waters is evaluated using information about existing uses of groundwater and surface water and their known and likely future uses as the basis for selecting a location to summarize the results of the transport calculation. The source term from a postulated accidental release is reviewed under NUREG-0800, Section 11.2, following the guidance in Branch Technical Position (BTP) 11-6, "Postulated Radioactive Releases Due to Liquid-containing Tank Failures." The source term is determined from a postulated release from a single tank outside of the containment. The results of a consequence analysis are evaluated against Standard Review Plan (SRP) Section 11.2 and BTP 11-6 guidance and effluent concentration limits (ECLs) of Table 2, Column 2 in 10 CFR Part 20, Appendix B, as SRP acceptance criteria. Under SRP guidance, the effluent concentration limits of 10 CFR Part 20, Appendix B are applied as acceptance criteria only for the purpose of assessing the acceptability of the results of the consequence analysis and are not intended for demonstrating compliance with ECLs.

The following specific areas are reviewed by the staff: (1) Alternative conceptual models of the hydrology at the site that reasonably bound the site's hydrogeological conditions to the degree that these conditions affect the transport of radioactive liquid effluent in the groundwater and surface-water environment; (2) a bounding set of plausible surface and subsurface pathways from potential points of an accidental release to determine the critical pathways that may result in the most severe effect on existing uses and known and likely future uses of groundwater and surface-water resources in the vicinity of the site; (3) the ability of the groundwater and surface-water environments to delay, disperse, dilute, or concentrate accidentally released radioactive liquid effluents during transport; and (4) the assessment of scenarios wherein an accidental release of radioactive effluents is combined with potential effects of seismic and non-seismic events.

2.4.13.2 *Summary of Application*

COL FSAR Section 2.4.13, "Pathways of Liquid Effluents in Ground and Surface Waters," incorporates by reference U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.13, "Pathways of Liquid Effluents in Ground and Surface Waters." This section of the COL FSAR addresses the accidental release of radioactive liquid effluents in ground and surface waters.
In addition, in COL FSAR Section 2.4.13, the COL applicant addressed COL Information Item 2.4-14 identified in U.S. EPR FSAR Tier 2, Revision 4, Table 1.8-2 and U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.13.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-14

> A COL applicant that references the U.S. EPR design certification will provide site-specific information on the ability of the groundwater and surface water environment to delay, disperse, dilute, or concentrate accidental radioactive liquid effluent releases, regarding the effects that such releases might have on existing and known future uses of groundwater and surface water resources.

The applicant addressed this COL information item by including in the COL FSAR the following information as a supplement to the U.S. EPR FSAR Revision 4:

- The COL applicant described the accident scenario and resulting source term. Rupture of a Reactor Coolant Storage Tank in the Nuclear Auxiliary Building was selected as the postulated source, because these tanks contain the largest volume of reactor coolant water. The tank was postulated to instantaneously release 80 percent of its volume to the Upper Chesapeake unit.

- The COL applicant described the conceptual model for groundwater transport and identified six alternative groundwater transport pathways for consideration. Pathway travel times were estimated using site data and the results of the five-layer groundwater model discussed in Section 2.4.12 of this report. Pathways with the shortest travel time were through the Upper Chesapeake unit and through the excavation fill material, with each of these pathways discharging to Branch 2, as shown in Table 2.4.13-1 of this report.

- The COL applicant described a radionuclide transport analysis, including the calculation of radionuclide concentrations and doses, and comparison with acceptance criteria based on 10 CFR Part 20. The COL applicant's transport analysis was primarily based on a method of characteristics solution to the one-dimensional advection-dispersion equation with first-order decay and linear equilibrium adsorption. The transport analysis was conducted using five stages. The initial stage of the transport analysis considered advection and radioactive decay only. The second stage of the COL applicant's analysis included the effect of radionuclide decay and adsorption in computing radionuclide concentrations at the groundwater discharge locations. The third stage of the COL applicant's analysis included the effect of decay, adsorption, and dilution of groundwater discharged to the creeks. The fourth stage of the applicant's transport analysis included the effect of longitudinal and lateral dispersion in groundwater using a two-dimensional analytical solution.

- The "sum of fractions approach" described in 10 CFR Part 20, Appendix B, Table 2, was evaluated by the COL applicant using the minimum concentration resulting from the application of each stage. The COL applicant concluded that the 10 CFR Part 20 requirements were satisfied for the pathways to Branch 1, Branch 3, and Johns Creek. The sum of fractions exceeded 1.0 for the pathways to Branch 2 and the Chesapeake Bay. A fifth stage of the transport analysis was completed by the COL applicant, in which the radionuclide concentrations from the fourth stage, without the surface-water dilution, were used to evaluate potential biological uptake and human ingestion of fish, crustaceans, and mollusks harvested in Chesapeake Bay at the point of

groundwater discharge to the Chesapeake Bay and near the discharge of Branch 2. The COL applicant estimated that the annual radiological dose to humans from the ingestion of contaminated biota ranged from 12.4 to 74.0 mrem/yr (0.12 to 0.74 mSv/yr) for the three relevant pathways.

- The COL applicant considered release to surface water and concluded that because of the location of tanks and building design features, the accident scenario excluded a direct release of radioactivity to surface water.

2.4.13.3 *Regulatory Basis*

The relevant requirements of NRC regulations for the pathways of liquid effluents in ground and surface waters, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.13.

The applicable regulatory requirements for evaluating accidental release of radioactive liquid effluents in ground and surface waters are set forth in the following:

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

The staff also used the acceptance criteria identified in NUREG-0800, Section 2.4.13:

- Alternate Conceptual Models: Alternate conceptual models of hydrology in the vicinity of the site are reviewed.

- Pathways: The bounding set of plausible surface and subsurface pathways from the points of release are reviewed.

- Characteristics that Affect Transport: Radionuclide transport characteristics of the groundwater environment with respect to existing and known and likely future users should be described.

- Consideration of Other Site-Related Evaluation Criteria: The COL applicant's assessment of the potential effects of site-proximity hazards, seismic, and non-seismic events on the radioactive concentration from the postulated tank failure related to accidental release of radioactive liquid effluents to ground and surface waters for the proposed plant site is needed.

- BTP 11-6 provides guidance in assessing a potential release of radioactive liquids after the postulated failure of a tank and its components, located outside of containment, and effects of the release of radioactive materials at the nearest potable water supply, located in an unrestricted area, for direct human consumption or indirectly through animals, crops, and food processing.

The staff used best current practices to analyze groundwater transport of radioactive liquid effluents.

In addition, the hydrologic characteristics should conform to appropriate sections from RG 1.113, "Estimating Aquatic Dispersions of Effluents from Accidental and Routine Reactor Releases for the Purpose of Implementing Appendix I."

The regulatory basis of the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.13.4 *Technical Evaluation*

The staff reviewed COL FSAR Section 2.4.13, Revision 9, and checked the applicable sections of the U.S. EPR FSAR Revision 4 to ensure that the combination of the information represents the complete scope of required information related to this review topic. The review confirmed that the information contained in the COL application and incorporated by reference addressed the required information related to this section. U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.13 has been reviewed by the staff under Docket No. 52-020. The staff's technical evaluation of the information incorporated by reference related to U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.13 has been documented in the staff FSER on the design certification application for the U.S. EPR. The FSER on the U.S. EPR FSAR is not yet complete. The staff will update Section 2.4.13 of this report to reflect the final disposition of the design certification application.

The staff reviewed the resolution of the COL-specific items related to the accidental release of radioactive liquid effluents in ground and surface waters included under COL FSAR, Revision 9, Section 2.4.13.

The staff's review of the information contained in the COL FSAR is discussed below.

Information Provided by COL Applicant

The COL applicant considered the possible direct release of liquid effluents to a surface-water pathway and concluded that such releases were unlikely. As a result, the COL applicant did not postulate or analyze any accident scenarios for the release of liquid effluents directly to surface water.

The COL applicant developed a conservative analysis of the accidental release of liquid effluents to groundwater. The COL applicant selected rupture of a reactor coolant storage tank in the Nuclear Auxiliary Building as the postulated source, because these tanks contain the largest volume of reactor coolant water. Source term radionuclide activities were stated by the COL applicant to represent maximum values observed in two reactor coolant analyses.

The tank selected by the COL applicant for the source term is located at a floor elevation of approximately 13.7 m (45 ft) and has a volume of 125 m^3 (4,410 ft^3). The COL applicant postulated the tank to instantaneously release 80 percent of its volume (100 m^3 (3,530 ft^3)) to the Upper Chesapeake unit. The COL applicant noted that this assumes instantaneous flow past the building containment structure and sump collection system. In the excavation description of COL FSAR Section 2.5.4, the COL applicant stated that the Stratum IIa Chesapeake Clay/Silt (referred to in COL FSAR Section 2.4.12 as the Upper Chesapeake

aquitard) will be completely excavated below the Nuclear Auxiliary Building with structural fill placed beneath the building foundation.

Using the results of the five-layer groundwater flow model discussed in COL FSAR 2.4.12 and described in detail in the May 3, 2010, response to RAI 101, Question 02.04.12-11, the COL applicant evaluated subsurface pathways from the Nuclear Auxiliary Building to surface-water discharge points under post-construction groundwater conditions. The COL applicant stated that the surficial aquifer cannot be affected by the postulated radionuclide release because of the aquifer's termination elevation (18.6 m (61.0 ft)) being well above the postulated release elevation and the observation of lower groundwater heads in the Upper Chesapeake unit than in the surficial aquifer, which precludes upward flow between these units. The COL applicant stated that the groundwater pathway from the Nuclear Auxiliary Building is north-northeast toward a discharge point in Branch 2, and eventual discharge is to the Chesapeake Bay via this surface stream. In RAI 104, Question 02.04.13-4, the staff requested that the COL applicant provide a discussion of the technical basis for concluding that the postulated groundwater pathway was conservative, including discussion of possible alternative pathways. In a June 30, 2010, response to RAI 104, Question 02.04.13-4, the COL applicant identified five other potential transport pathways. Four of these pathways were through the Upper Chesapeake unit with discharge to Chesapeake Bay, Branch 1, Johns Creek, and Branch 3. The fifth pathway was through the structural fill with discharge to Branch 2.

Travel times for the six potential pathways identified by the COL applicant were provided in the June 30, 2010, response to RAI 104, Question 02.04.13-4, and were included in COL FSAR Table 2.4-46, "Calculated Hydraulic Gradients, Groundwater Velocities, and Travel Times." The COL applicant used hydraulic gradients based on groundwater head results from the five-layer groundwater flow model discussed in Section 2.4.12.4 above, except for the pathway through the structural fill, where the COL applicant used the model-estimated travel time directly. The COL applicant's hydraulic gradient values are given in Table 2.4.13-1 of this report. The COL applicant used the maximum observed hydraulic conductivity and the effective porosity (COL FSAR Revision 7 value), both given in Table 2.4.12-2 of this report, for the Upper Chesapeake unit pathways. The structural fill pathway travel time was computed by the COL applicant from groundwater model results using the upper limit of the estimated range for the hydraulic conductivity of the structural fill (10^{-2} cm/s (30 ft/d)) and an estimated effective porosity of 0.082. The COL applicant's groundwater velocities are given in Table 2.4.13-1 of this report. Straight-line distances were used by the COL applicant to compute travel times. Travel distances and travel times as calculated by the COL applicant are given in Table 2.4.13-1 of this report. The pathways with the shortest travel time identified by COL applicant were the Upper Chesapeake unit pathway with discharge to Branch 2 and the pathway through the structural fill. The staff noted that these travel times are more conservative (i.e., shorter) than the travel times calculated for the existing conditions (given in Table 2.4.12-3 of this report).

Table 2.4.13-1. Travel Distance and Estimated Travel Time for Subsurface Pathways Evaluated by the Applicant under Post-Construction Conditions

	Subsurface Pathway					
	Upper Chesapeake Unit					Structural Fill
Discharge Point	Branch 2	Chesapeake Bay	Branch 1	Johns Creek	Branch 3	Branch 2
Travel Distance, m (ft)	176.8 (580)	400.8 (1,315)	461.8 (1,515)	883.9 (2,900)	765.0 (2,510)	164.6 (540)
Hydraulic Gradient	-0.057	-0.042	-0.018	-0.009	-0.008	NA
Groundwater Velocity, m/d (ft/d)	1.64 (5.38)	1.20 (3.94)	0.51 (1.67)	0.25 (0.82)	0.24 (0.79)	1.43 (4.69)
Travel Time, yr	0.3	0.9	2.5	9.7	8.7	0.3

The COL applicant evaluated radionuclide transport through all six potential pathways. The COL applicant's groundwater radionuclide transport analysis was primarily based on a method of characteristics solution to the one-dimensional advection-dispersion equation with first-order decay and linear equilibrium adsorption. The COL applicant conducted the transport analysis using four stages. At each of the first three stages, the COL applicant considered in the following stage only radionuclides whose concentrations were greater than 0.01 times the applicable concentration. The fourth stage, which considered the effect of dispersion on radionuclide transport, was provided in the June 30, 2010, response to RAI 104, Question 02.04.13-4, and was included in COL FSAR Section 2.4.13.1.4, "Radionuclide Transport Analysis."

The initial stage of the COL applicant's transport analysis considered advection and radioactive decay only. A summary of the COL applicant's results was provided in the June 30, 2010, response to RAI 104, Question 02.04.13-4, included as COL FSAR Table 2.4-51, "Summary of Results for the Transport Analysis Considering Advection, Radioactive Decay," and Tables 2.4-61 to 2.4-66 (transport analysis results for each pathway). The COL applicant stated that the pathways to Branch 2 (via the Upper Chesapeake unit and the structural fill) had the greatest number of radionuclides with concentrations exceeding 0.01 times the 10 CFR Part 20, Appendix B, "Annual Limits on Intake (ALIs) and Derived Air Concentrations (DACs) of Radionuclides for Occupational Exposure; Effluent Concentrations; Concentrations for Release to Sewerage," Table 2 limits.

The second stage of the COL applicant's analysis included the effect of advection, radionuclide decay, and adsorption in computing radionuclide concentrations at the groundwater discharge locations. The COL applicant obtained site-specific distribution coefficients (K_d values) for this analysis for Mn, Fe, Co, Zn, Sr, Ru, Cs, and Ce using site groundwater, 9 soil samples obtained from the surficial aquifer, and 11 soil samples from the Upper Chesapeake unit. The COL applicant provided distribution coefficient values in COL FSAR Table 2.4-48, "Summary of the Radionuclide K_d Values for 20 Soils and Averages." For each radionuclide, the COL applicant used the minimum measured distribution coefficient value in the transport analysis. For the pathway through the fill material, the COL applicant used the minimum measurement from the surficial aquifer samples, while for the remainder of the pathways the COL applicant used the minimum from the Upper Chesapeake unit samples. The COL applicant assumed distribution

coefficients to be zero for radionuclides without site-specific measurements. The COL applicant computed retardation factors using bulk densities of 1.53 g/cm^3 (95.5 lb/ft^3) for the Upper Chesapeake unit and 2.24 g/cm^3 (140 lb/ft^3) for the structural fill, and provided them in the June 30, 2010, response to RAI 104, Question 02.04.13-4, and in COL FSAR Table 2.4-52, "Retardation Factors Calculated Using Site-Specific Distribution Coefficients." The COL applicant assumed the effective porosity of the fill was 0.082. The COL applicant provided a summary of the transport results in the June 30, 2010, response to RAI 104, Question 02.04.13-4, and included it as COL FSAR Table 2.4-53, "Summary of Results for the Transport Analysis Considering Advection, Radioactive Decay, and Adsorption," and Tables 2.4-61 to 2.4-66 (transport analysis results for each pathway). The COL applicant included radionuclides for which the resulting concentration was greater than 0.01 times the 10 CFR 20, Appendix B, Table 2 value in the next stage of the analysis.

The third stage of the COL applicant's analysis included the effect of advection, decay, adsorption, and dilution of groundwater discharged to the creeks (Branches 1, 2, and 3 as well as Johns Creek). The stream discharges used by the COL applicant for dilution were 100-year low annual flow rates estimated from flow observations in other small watersheds in the region (using an inverse-distance relationship and extrapolating low-flow estimates to the 100-year return period). The COL applicant assumed no dilution for the pathway discharging directly to Chesapeake Bay. The COL applicant provided flow rates in the June 30, 2010, response to RAI 104, Question 02.04.13-4, and included them in COL FSAR Table 2.4-56, "Calculation of Effluent Discharge Rates and Dilution Factors." The COL applicant's estimated flow rate in Branch 2 was 2.3×10^{-3} m^3/s (0.08 ft^3/s), a small percentage of the estimate cited in the wetlands report (Tetra Tech NUS 2007, ML100040304), based on a visual examination of the stream.

The COL applicant computed the flow rate of contaminated groundwater along each pathway by assuming the volume of liquid released from the tank (100 m^3 (3,530 ft^3)) occupied the entire thickness of the Upper Chesapeake unit (6.52 m (21.4 ft)) or structural fill and took the shape of a square area in plain view. The discharge area under these assumptions was equal to the product of the width of the contaminated zone normal to groundwater flow and the saturated thickness. The COL applicant multiplied this discharge area by the Darcy velocity to compute a contaminated groundwater flow rate. The COL applicant took the ratio of the contaminated groundwater flow rate and the 100-year low stream flow as the dilution factor. Dimensions of the contaminant slug and the resulting dilution factor for each pathway were provided by the COL applicant in the June 30, 2010, response to RAI 104, Question 02.04.13-4, and included in COL FSAR Tables 2.4-55, "Dimensions of the Contaminant Slug," and 2.4-56, "Calculation of Effluent Discharge Rates and Dilution Factors." A summary of the transport results was provided by the COL applicant in the June 30, 2010, response to RAI 104, Question 02.04.13-4, and included in COL FSAR Table 2.4-57, "Summary of Results for the Transport Analysis Considering Advection, Radioactive Decay, Adsorption, and Dilution in Surface Water," and Tables 2.4-61 to 2.4-66 (transport analysis results for each pathway). Radionuclides for which the resulting concentration was greater than 0.01 times the 10 CFR Part 20, Appendix B, Table 2 value were included by the COL applicant in the next stage of the analysis. This included 15 radionuclides for the pathway through the Upper Chesapeake unit to Branch 2 and 12 radionuclides for the pathways to Chesapeake Bay and to Branch 2 via the structural fill.

The fourth stage of the COL applicant's transport analysis included the effect of longitudinal and lateral dispersion in groundwater using a two-dimensional analytical solution. The longitudinal dispersivities used in the analysis were provided by the COL applicant in the June 30, 2010,

response to RAI 104, Question 02.04.13-4, and included in COL FSAR Table 2.4-58, "Estimated Longitudinal Dispersivities." Lateral dispersivity was assumed by the COL applicant to be one-tenth of the longitudinal value(s). Higher dispersivity values resulted in lower concentrations except for three specific radionuclide-pathway combinations. A summary of the transport results was provided by the COL applicant in the June 30, 2010, response to RAI 104, Question 02.04.13-4, and included in COL FSAR Table 2.4-59, "Summary of Results for the Transport Analysis Considering Advection, Radioactive Decay, Adsorption, and Dilution," and Tables 2.4-61 to 2.4-66 (transport analysis results for each pathway). For the two pathways to Branch 2, H-3 and I-131 exceeded the 10 CFR 20, Appendix B, Table 2 limits. For the pathway to Chesapeake Bay, the limits were exceeded by H-3.

A fifth stage of the transport analysis was provided by the COL applicant, in which the radionuclide concentrations from the fourth stage, without the surface-water dilution, were used by the COL applicant to evaluate potential biological uptake and human ingestion of fish, crustaceans, and mollusks harvested in Chesapeake Bay at the point of groundwater discharge or near the discharge of Branch 2. The COL applicant estimated that the annual human radiological dose from the ingestion of contaminated biota ranged from 12.4 to 74.0 mrem/yr for the three relevant pathways. Results were presented by the COL applicant in COL FSAR Tables 2.4-61 to 2.4-66 (transport analysis results for each pathway).

The "sum of fractions approach," (unity rule) described in 10 CFR Part 20, Appendix B, Table 2, was evaluated by the COL applicant using the minimum concentration resulting from the application of each stage. The resulting sum for each pathway was provided by the COL applicant in COL FSAR Table 2.4-60, "Sum of Radionuclide Activity Concentration/ECL Ratios for Each Pathway." The COL applicant concluded that the 10 CFR Part 20 requirements were satisfied for the pathways to Branch 1, Branch 3, and Johns Creek. The sum of fractions exceeded 1.0 for the remaining pathways. The COL applicant noted the use of conservative values for hydraulic conductivities and effective porosities, that additional dilution of the radionuclides would occur downstream in Branch 2 and the Chesapeake Bay, and that neither Branch 2 nor the Chesapeake Bay are used as a potable water supply.

The Staff's Technical Evaluation

The staff reviewed the COL applicant's accidental release scenario. In RAI 104, Question 02.04.13-1, the staff requested that the COL applicant provide confirmation of and the technical basis for, the use of the Reactor Coolant Storage Tank as the source tank with the greatest inventory for the purposes of the accidental release analysis. In a July 15, 2009, response to RAI 104, Question 02.04.13-1, the COL applicant stated that total activity released from this tank bounds the releases from the other liquid sources considered, confirming the Reactor Coolant Storage Tank as the source tank with the greatest inventory for the purposes of the accidental release analysis. Based on the analysis discussed above, the staff finds the COL applicant's July 15, 2009, response to RAI 104, Question 02.04.13-1, acceptable. Accordingly, the staff considers RAI 104, Question 02.04.13-1 resolved. In the July 15, 2009, response to RAI 104, Question 02.04.13-1, the COL applicant also evaluated the release of Mn-56 and Tc-99 for consistency with NRC Interim Staff Guidance (ISG) Design Certification (DC)/COL-ISG-013. The COL applicant concluded that the inclusion of these radionuclides in the source release would not affect the results.

In RAI 104, Question 02.04.13-2, the staff requested that the COL applicant provide a reference for the radionuclide activities used as the source in the accidental release analysis. In a July 15,

2009, response to RAI 104, Question 02.04.13-2, the COL applicant provided additional information about the technical basis for the radionuclide activities used as the source in the accidental release analysis. The COL applicant described the basis for the source term activities, with reference to ANSI/ANS-18.1-1999, "Radioactive Source Term for Normal Operation of Light Water Reactors" (ANS 1999), the Dose Equivalent Iodine-131 Technical Specification, and the range of burnup considered in determining bounding activities.

The staff compared the COL applicant's source term to U.S. EPR FSAR Tier 1, Revision 1, Table 5.0-1, "Inventory of Radionuclides Which Could Potentially Seep Into the Groundwater," and determined that, although the COL applicant's source was more conservative, it was inconsistent with supporting information presented in U.S. EPR FSAR Tier 2, Revision 4, Table 11.1-2 and U.S. EPR FSAR Tier 1, Revision 1, Table 5.0-1. The event scenario stated that the assumed radionuclide concentrations were based on 0.25 percent defective fuel fraction, but the concentrations presented in COL FSAR, Revision 6, Table 2.4-44, "Reactor Coolant Storage Tank Radionuclide Inventory," were much higher for 21 radionuclides; factors ranged from about 1.2 to 5 times higher. The discussion in COL FSAR Section 2.4.13.1.1 did not explain the source of such differences. A review of the COL applicant's responses to RAI 104, Questions 02.04.13-1 and 02.04.13-2, did not provide a technical basis for the assumed higher concentrations for 21 radionuclides. Therefore, in RAI 104, Question 02.04.13-5, the staff requested that the COL applicant provide the technical basis for the assumed higher concentrations and revise the discussion in COL FSAR Section 2.4.13.1.1, COL FSAR Table 2.4-43, and U.S. EPR FSAR Tier 1, Revision 1, Table 5.0-1, to include the basis for the higher concentrations. The COL applicant was requested to provide enough details in its response for the staff to conduct an independent evaluation and confirm compliance with the acceptance criteria in NUREG-0800, Section 11.2 and BTP 11-6.

In an April 27, 2010, response to RAI 104, Question 02.04.13-5, the COL applicant provided additional information and made reference to two RAI responses prepared on associated topics for the U.S. EPR design certification. One RAI response (U.S. EPR, RAI 292, Question 14.03.07-34) removed the list of radionuclides and concentrations given in U.S. EPR FSAR Tier 1, Table 5.0-1 (Sheet 3), because this information was the same as that presented in U.S. EPR FSAR Tier 2, Table 2.1-2. The other RAI response (U.S. EPR, RAI 86, Question 09.01.02-21) presented a justification for the given radionuclides and added tritium to the list presented in a proposed revision of U.S. EPR FSAR Tier 1, Table 5.0-1. While U.S EPR FSAR Tier 1, Table 5.0-1 was ultimately deleted, as noted above, the proposed revisions were carried forward and included in an updated version of U.S. EPR FSAR Tier 2, Table 2.1-2, and endorsed by the COL applicant in its analysis. In addition, as part of the June 30, 2010, response to RAI 104, Question 02.04.13-4, the COL applicant updated the Reactor Coolant Storage Tank inventory and included this information in COL FSAR Table 2.4-47, "Reactor Coolant Storage Tank Radionuclide Inventory." The staff compared the COL applicant's source term to U.S. EPR FSAR Tier 2, Revision 4, Table 11.1-2, "RCS Design Basis Source Term," and noted that the COL applicant's source was more conservative. Based on this, and considering the information provided by the COL applicant in the responses to RAI 104, Questions 02.04.13-1 and 02.04.13-2, the staff concluded that the Reactor Coolant Storage Tank was the most conservative radionuclide source with respect to volume and activity.

With respect to demonstrating compliance with NUREG-0800, Sections 2.4.13 and 11.2 and BTP 11-6, the COL applicant noted that the information and results of the consequence analysis were presented as part of the response to RAI 104, Question 02.04-13-4. Given the information

discussed above, the staff subsumed the issues raised in RAI 104, Question 02.04.13-5, into RAI 104, Question 02.04.13-4, and closed out RAI 104, Questions 02.04.13-5 and 02.04.13-2.

The COL applicant did not specify whether the source tank would contain chelating agents that might affect transport. Therefore, in RAI 104, Question 02.04.13-3, the staff requested that the COL applicant provide additional information about the presence or absence of chelating agents in the tank used for the source in the accidental release analysis and also discuss the planned use of any chemical agents anywhere at the CCNPP site that could modify the radionuclide transport characteristics of the subsurface region. In a July 15, 2009, response to RAI 104, Question 02.04.13-3, the COL applicant stated that no chelating agents are used in the reactor coolant treatment system or are present in the reactor coolant during normal operations. The staff finds the COL applicant's response acceptable and, therefore, considers RAI 104, Question 02.04.13-3 resolved.

In RAI 104, Question 02.04.13-4, the staff requested that the COL applicant provide the technical basis for concluding that the groundwater transport analysis is conservative, including discussion of the following specific issues: (1) The assumption that a transport analysis that does not consider hydrodynamic dispersion is conservative for a constituent subject to decay; (2) consideration of the possible combination of radionuclides at the boundary of the unrestricted area due to variation in K_d values; (3) consideration of possible alternative pathways (e.g., to the underlying aquifer or to Johns Creek and Branch 3); (4) the effect of site construction (including excavation and fill) on possible alternative transport pathways; and (5) consistency of the transport analysis with COL FSAR Sections 2.4.12 and 2.5.4.

The staff reviewed the COL applicant's June 30, 2010, response to RAI 104, Question 02.04.13-4. The staff conducted an independent confirmatory analysis that verified the COL applicant's calculations for the case with dispersion. Based on the results of the confirmatory analysis and the COL applicant's results presented in the June 30, 2010, response to RAI 104, Question 02.04.13-4, the staff concluded that the COL applicant's transport analysis without hydrodynamic dispersion was conservative. The staff also concluded that the methodology used by the COL applicant to evaluate the effect of dispersion had an adequate technical basis and was appropriate for use in this analysis as part of a staged approach that proceeded from a conservative, screening-level analysis to a more realistic analysis.

The staff evaluated the COL applicant's analysis of adsorption and concluded that the assignment of adsorption coefficients was conservative and the evaluation of radionuclide mixtures was completed without regard to travel times. The staff concluded that this approach maximized the sum of fractional concentrations and the estimated dose, and was therefore conservative.

The staff reviewed the methods for estimating radionuclide concentrations presented in the COL applicant's June 30, 2010, response to RAI 104, Question 02.04.13-4, and noted that the applicant had an adequate technical basis and were appropriate for this analysis. The staff conducted an independent calculation of radionuclide concentrations for the bounding pathway (Upper Chesapeake unit to Branch 2) and verified the COL applicant's calculations.

Using the results of the five-layer groundwater flow model documented in the COL applicant's May 3, 2010, response to RAI 101, Question 02.04.12-11, the COL applicant identified two transport pathways from the Nuclear Auxiliary Building as being the most plausible: (1) A pathway through the Upper Chesapeake unit to Branch 2, and (2) a pathway through the

Upper Chesapeake unit discharging directly to Chesapeake Bay. The COL applicant considered three alternative pathways through the Upper Chesapeake unit (to Branch 1, Branch 3, and Johns Creek) and one alternative pathway through the structural fill materials (to Branch 2). The COL applicant concluded that the three alternative pathways through the Upper Chesapeake unit were not supported by the five-layer groundwater flow model. The COL applicant nonetheless evaluated transport along these pathways as described above. The pathway through the structural fill was supported by the groundwater model when the structural fill was assigned the maximum hydraulic conductivity value of its potential range. As described in Section 2.4.12.4 above, the staff evaluated the five-layer groundwater flow model, which was the basis for the pathways and travel times used by the COL applicant in COL FSAR 2.4.13. Based on the reasons given above, the staff finds that the model provides an adequate technical basis for the evaluation of conservative pathways and travel times, and that the model provides an adequate representation of post-construction conditions and is consistent with information in COL FSAR Section 2.5.4. On the basis of the analysis described above, the staff considers RAI 104, Question 02.04.13-4 resolved.

In RAI 266, Question 02.04.13-6, the staff requested that the COL applicant address the following issues related to the dose calculations presented in the June 30, 2010, response to RAI 104, Question 02.04.13-4. In Tables 2.4-206 and 2.4-211 of the June 30, 2010, response to RAI 104, Question 02.04.13-4, the basis of the stated consumption rates for fish (5.4 kg/yr (12 pounds (lb)/yr)) and mollusk/crustacean (0.9 kg/yr (2 lb/yr)) were not attributed to specific references. The staff requested that the COL applicant identify the references for the assumed consumption rates. In addition, the staff questioned the use of RESRAD code data and the basis for the stated consumption rates. The RESRAD code was developed for the purpose of evaluating sites after the decommissioning of nuclear facilities. Regarding the consumption rates of fish, mollusks, and crustaceans, the staff requested that the COL applicant consider the guidance and consumption rates given in RG 1.109, Tables E-4 and E-5. The concern with the RESRAD data was that they may not be sufficiently conservative in assessing the effects of a radioactive waste tank failure. The objective was to use a common set of data and references in applying fish, mollusk, and crustacean consumption rates. The staff also questioned the use of freshwater site bioaccumulation factors in assessing the transfer of radioactivity from water to fish, crustaceans, and mollusks in a saltwater environment. The staff noted that the dose assessments applied in the COL Environmental Report and COL FSAR Section 11.2 for radioactive waste liquid effluent releases were based on parameters that characterize a saltwater site. The COL applicant was requested to update its assessment using bioaccumulation factors for a saltwater site. For guidance and supporting information, the COL applicant was referred to RG 1.109, Table A-1, or GENII Computer codes, Appendix D, Tables D.11 to D.13. The objective was to use a common set of data and references in applying bioaccumulation factors for saltwater and freshwater site conditions.

In a November 12, 2010, response to RAI 266, Question 02.04.13-6, the COL applicant provided an updated dose consequence analysis and a proposed revision of the information presented in COL FSAR Sections 2.4.13.1.4.7, 2.4.13.1.4.12, and 2.4.13.1.5. In its response, the COL applicant revised the model input parameters used in modeling the consumption of fish, mollusks, and crustaceans, and applied bioaccumulation factors for saltwater biota. The consumption rates were based on RG 1.109, Table E-4 and the bioaccumulation factors were based on GENII code, Version 2. Tables D.11 to D.13, software manual. The radionuclide distributions and source term concentrations used in the COL applicant's analysis were based on the U.S. EPR design certification and given in COL FSAR Table 2.4-47. The revised consequence analysis presented results for releases into the Chesapeake Bay via the

six pathways discussed above. Concentrations in the Chesapeake Bay were recalculated for all six discharge pathways.

The results for three transport pathways showed that the sum of radionuclide concentrations as ECL ratios did not exceed 1.0, as shown in COL FSAR Table 2.4-60. For these three pathways, the sum of fractions was 0.3 for the pathway to Branch 1, 0.01 for the pathway to Johns Creek, and 0.02 for the pathway to Branch 3. For the other three release pathways, the ECL ratios and unity rule were exceeded. The sum of radionuclide concentrations as ECL ratios was 8.1 for the pathway to Branch 2, 5.6 for the pathway to Chesapeake Bay, and 5.2 for the pathway to Branch 2 through the excavation fill materials. For these release pathways, the COL applicant calculated doses and compared dose results to the limit of 100 mrem (1 mSv), as established in 10 CFR 20.1301 for members of the public. The COL applicant applied a dose limit as a surrogate SRP acceptance criterion in assessing the radiological effect of the releases given that, although water from the Chesapeake Bay is not potable, human exposure could occur via the consumption of fish and shellfish.

Using the concentrations presented in COL FSAR Tables 2.4-61, 2.4-62, and 2.4-66, and information provided in the proposed revision of COL FSAR Section 2.4.13, the staff confirmed the resulting doses and verified that they were less than the dose limit of 100 mrem (1 mSv) in unrestricted areas. The resulting doses were 33 mrem (0.33 mSv) for the Branch 2 pathway, 12 mrem (0.12 mSv) for the Chesapeake Bay pathway, and 74 mrem (0.74 mSv) for the Branch 2 pathways flowing through the fill materials. The staff noted that the COL applicant's updated analysis incorporated an adequate level of conservatism, with the major conservative factor being that the calculations excluded the effect of water dilution in the Chesapeake Bay upon discharge. In light of the above, the staff finds the revised dose calculations and revised COL FSAR Sections 2.4.13.1.4.7, 2.4.13.1.4.12, and 2.4.13.1.5; and Tables 2.4-61, 2.4-62, and 2.4-66 acceptable. Accordingly, the staff considers RAI 266, Question 02.04.13-6 resolved in light of the update of COL FSAR, Revision 7, Section 2.4.13.

2.4.13.4.1 Technical Specifications

The staff's evaluation of the COL applicant's exclusion of the requirement specifying controls and limits of radioactivity levels in outdoor radioactive waste storage tanks, presented in COL FSAR Chapter 16, "Technical Specifications," and COL application, Part 4: Technical Specifications and Bases, is addressed in Sections 11.2 and 11.16 of this report.

2.4.13.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.13.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR Revision 4. On the basis of its review, the staff confirmed that the COL applicant addressed the required information related to the accidental release of radioactive liquid effluent in ground and surface waters. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to the accidental release of radioactive liquid effluent in ground and surface waters incorporated by reference in the COL FSAR will be documented in the staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will

update Section 2.4.13 of this report to reflect the final disposition of the design certification application.

2.4.14 Technical Specifications and Emergency Operation Requirements

2.4.14.1 *Introduction*

COL FSAR Section 2.4.14, "Technical Specification and Emergency Operation Requirements," describes the technical specifications and emergency operation requirements as necessary. The requirements described implement protection against floods for safety-related facilities to ensure that an adequate supply of water for shutdown and cool-down purposes is available.

This section of the report provides an evaluation of the following specific areas: (1) The control of hydrological events, which are determined in the hydrology sections of the COL FSAR, to identify bases for emergency actions required during these events; (2) the amount of time available to initiate and complete emergency procedures before the onset of conditions during controlling hydrological events that may prevent such action; (3) the organization responsible for the review of issues related to technical specifications reviews the technical specifications related to all emergency procedures necessary to ensure adequate plant safety from controlling hydrological events; (4) potential effects of seismic and non-seismic information about the postulated technical specifications and emergency operations for the proposed plant site; and (5) any additional information requirements prescribed in the "Contents of Application" sections of the applicable subparts of 10 CFR Part 52.

2.4.14.2 *Summary of Application*

This section of the COL FSAR addresses technical specifications and emergency operation requirements. COL FSAR Section 2.4.14 incorporates by reference U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.14. In this section, the COL applicant provided site-specific supplemental information to address COL-specific information identified in U.S. EPR FSAR Tier 2, Revision 4, Section 2.4.14.

The COL applicant addressed the issues as follows:

COL Information Item 2.4-15

> A COL applicant that references the U.S. EPR design certification will describe any emergency measures required to implement flood protection in safety-related facilities and to verify there is an adequate water supply for shutdown purposes.

The COL applicant provided additional information in COL FSAR Section 2.4.14 to address U.S. EPR FSAR \Tier 2, Revision 4, Table 1.8-2, "U.S. EPR Combined License Information Items," COL Information Item 2.4.15, which describe emergency measures required to implement flood protection in safety-related facilities. The COL applicant stated that based on prior sections of the COL FSAR, no emergency protective measures would be required to minimize the effect of hydrology-related events on safety-related structures. The COL applicant further stated that CCNPP Unit 3 would be designed in a manner that does not need technical specifications or Emergency Operating Procedures (EOPs) to protect the plant from hydrologic events.

2.4.14.3 *Regulatory Basis*

The relevant requirements of NRC regulations for technical specifications and emergency operation requirements, and the associated acceptance criteria, are specified in NUREG-0800, Section 2.4.14.

The applicable regulatory requirements for technical specifications and emergency operation requirements are set forth in the following:

- 10 CFR 50.36, "Technical specifications," as it relates to identifying technical specifications related to all emergency procedures required to ensure adequate plant safety from controlling hydrological events by the organization responsible for the review of issues related to technical specifications.

- 10 CFR 52.79(a)(1)(iii), as it relates to identifying hydrologic site characteristics with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

- 10 CFR Part 100, as it relates to identifying and evaluating hydrological features of the site. The requirement to consider physical site characteristics in site evaluations is specified in 10 CFR 100.20(c).

- 10 CFR 100.23(d), as it sets forth the criteria to determine the siting factors for plant design bases with respect to seismically induced floods and water waves at the site.

The staff also used the appropriate sections of the following RGs for the acceptance criteria identified in NUREG-0800, Section 2.4.14:

- RG 1.59, as supplemented by the current best practices, provides guidance for developing the hydrometeorological design bases,

- RG 1.102 describes acceptable flood protection to prevent the safety-related facilities from being adversely affected.

The regulatory basis of the information incorporated by reference will be addressed in the staff's FSER related to the U.S. EPR design certification application.

2.4.14.4 *Technical Evaluation*

The staff reviewed the information in COL FSAR Section 2.4.14.

The staff's review of the information contained in the COL FSAR is discussed as follows:

COL Information Item 2.4-15

The staff reviewed COL Information Item No. 2.4-15 from U.S. EPR FSAR Tier 2, Revision 4, Table 1.8-2 included under COL FSAR Section 2.4.14.

A COL applicant that references the U.S. EPR design certification will describe any emergency measures required to implement flood protection in safety-related facilities and to verify there is an adequate water supply for shutdown purposes.

The staff reviewed the COL applicant's supplemental information about the technical specifications and EOPs. The staff's review of the COL application is summarized below.

Information Provided By COL Applicant

In COL FSAR Section 2.4.14, based on prior sections of the COL FSAR, the COL applicant concluded that no emergency protective measures would be required to minimize the effect of hydrology-related events on safety-related structures. The COL applicant further stated that CCNPP Unit 3 would be designed in a manner that would eliminate the need for technical specifications or EOPs to protect the plant from hydrologic events.

The Staff's Technical Evaluation

As described in Section 2.4.2 of this report, the staff determined that the nuclear power block would not flood. In Section 2.4.10 of this report, the staff stated that UHS MWIS would be subject to inundation and that the DBA flooding event for the UHS MWIS would be the PMSS. The DBA low-water event would also be a PMSS. Based on the engineered features of the UHS MWIS intake (e.g., invert elevation of the intake, water-tight features of the UHS pump housing), the staff determined that neither technical specifications nor EOPs would be required.

Based on its review of the COL applicant's information in the COL FSAR, the staff concluded that the COL applicant appropriately considered flood-causing phenomena and their combinations that are relevant for the CCNPP Unit 3 site. Therefore, based on the reasons given above, the staff finds that COL applicant has met the requirements of 10 CFR 50.36, 10 CFR 52.79, 10 CFR 100.23(d), and 10 CFR 100.20(c), as they relate to identifying and evaluating emergency procedures and hydrological features of the site. Since the staff agrees that the combinations of flood-causing phenomena considered by the COL applicant are appropriate for the CCNPP Unit 3 site, and based on the reasons given above, the staff finds that the requirements of 10 CFR 52.79(a)(1)(iii) are met, as they relate to the determination of hydrologic characteristics of the proposed site with appropriate consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area and with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

2.4.14.5 *Post Combined License Activities*

There are no post COL activities related to this subsection.

2.4.14.6 *Conclusions*

The staff reviewed the COL application and the referenced U.S. EPR FSAR Revision 4. On the basis of its review, the staff confirmed that the COL applicant addressed the required information related to technical specification and emergency operations requirements. The staff is reviewing the information in the U.S. EPR FSAR on Docket No. 52-020. The results of the staff's technical evaluation of the information related to technical specification and emergency operations requirements incorporated by reference in the COL FSAR will be documented in the

staff FSER on the design certification application for the U.S. EPR. The staff notes that the FSER on the U.S. EPR is not yet complete. The staff will update Section 2.4.14 of this report to reflect the final disposition of the design certification application.