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MAR 0 3 2015

U. S. Nuclear Regulatory Commission
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SUSQUEHANNA STEAM ELECTRIC STATION
FLOOD HAZARDS REEVALUATION REPORT
PLA-7287

Docket Nos. 50-387
and 50-388

References:

1. *NRC Letter, Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident, dated March 12, 2012*
2. *PPL Letter (PLA-6867), Response to Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding the Flooding Aspects of Recommendations 2.1 and 2.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, dated June 11, 2012*

The United States Nuclear Regulatory Commission (NRC) issued Reference 1 on March 12, 2012, pursuant to Title 10 of the Code of Federal Regulations (CFR), Section 50.54(f), related to the implementation of Recommendations 2.1, 2.3, and 9.3 from the Near-Term Task Force, a portion of which called for performing flood hazard reevaluations at all nuclear power plants in the United States. In Reference 2, PPL Susquehanna, LLC indicated plans to comply with the requested response date of March 12, 2015 for flood hazard evaluation. The enclosure to this letter transmits the required Flood Hazard Reevaluation Report for the Susquehanna Steam Electric Station, Units 1 and 2.

There are no new or revised regulatory commitments contained in this submittal.

If you have any questions regarding this submittal, please contact Mr. Jeffery N. Grisewood, Manager, Nuclear Regulatory Affairs, at (570) 542-1330.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on: 3/3/15

T. S. Rausch
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Enclosure: Susquehanna Steam Electric Station Flood Hazards Reevaluation Report

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Enclosure to PLA-7287

Susquehanna Steam Electric Station Flood Hazards Reevaluation Report

SUSQUEHANNA STEAM ELECTRIC STATION FLOOD HAZARD REEVALUATION REPORT

12-4834

FEBRUARY 20, 2015

SUBMITTED TO:

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SUSQUEHANNA STEAM ELECTRIC STATION FLOOD HAZARD REEVALUATION REPORT

PROJECT NO. 12-4834
REVISION 1
FEBRUARY 20, 2015

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APPROVALS

Project No.:	12-4834
Report Name:	Susquehanna Steam Electric Station Flood Hazard Reevaluation Report
Date:	February 20, 2015
Revision No.:	Revision 1

Approval by the responsible manager signifies that the document is complete, all required reviews are complete, and the document is released for use.

Originator:	Jeffrey A. Oskamp, Engineering Associate, RIZZO Associates Jeffrey A. Oskamp, E.I.T. Engineering Associate	February 20, 2015 Date
Originator:	Tom Edwards, Engineering Associate, RIZZO Associates Tom Edwards Engineering Associate	February 20, 2015 Date
Independent Technical Reviewer:	Daniel J. Barton, Vice President, RIZZO Associates Daniel J. Barton Jr., P.E. Vice President	February 20, 2015 Date
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Principal In Charge:	John Paul Giunta, Senior Program Manager, RIZZO Associates John Paul Giunta, P.E. Principal	February 20, 2015 Date

CHANGE MANAGEMENT RECORD

Project No.: 12-4834

Report Name: Susquehanna Steam Electric Station
Flood Hazard Reevaluation Report

REVISION NO.	DATE	DESCRIPTIONS OF CHANGES/AFFECTED PAGES
Draft A	November 21, 2014	Draft A Submittal for PPL Comment
Draft B	January 14, 2015	Draft B Submittal incorporating resolutions to PPL Comments on Draft A
0	January 20, 2015	Revision 0 Submittal subsequent to second round of PPL review. No change from Draft B
1	February 20, 2015	Minor editorial updates based on PPL Comments

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SUSQUEHANNA STEAM ELECTRIC STATION FLOOD HAZARD REEVALUATION REPORT

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The United States Nuclear Regulatory Commission (NRC) issued a letter on March 12, 2012, pursuant to Title 10 of the Code of Federal Regulations (CFR), Section 50.54(f), related to the implementation of Recommendations 2.1, 2.3, and 9.3 from the Near-Term Task Force, a portion of which calls for performing flood hazard reevaluations at all Nuclear Power Plants in the United States (NRC, 2012a).

On behalf of PPL Susquehanna LLC (PPL), this Flood Hazard Reevaluation Report (FHRR) for the Susquehanna Steam Electric Station (SSES) Units 1 and 2 provides the information required to address NRC Recommendation 2.1 with due consideration of recent guidelines and regulations.

1.2 SITE BACKGROUND

The SSES site is licensed for the operation of two General Electric Boiling Water Reactors (BWRs) rated at approximately 1,300 megawatts electrical, each. The NRC issued the operating license for Unit 1 on July 17, 1982, and on March 23, 1984 for Unit 2. On November 24, 2009, the operating licenses for Units 1 and 2 were renewed, such that the expiration dates for the Units 1 and 2 licenses are July 17, 2042 and March 23, 2044, respectively (NRC, 2014a; NRC, 2014b).

The SSES Units 1 and 2 Final Safety Analysis Report (FSAR) (PPL, 2013) was issued as Revision 66 in February 2013.

1.3 HYDROLOGIC DESCRIPTION OF STUDY AREA

The SSES site is located in Salem Township in western Luzerne County, Pennsylvania (PA), at approximately 41°06' North latitude and 76°09' West longitude (***Figure 1-1***). The site is situated on a relatively flat plain of gently rolling hills on the west bank of the North Branch of the Susquehanna River (NBSR), which is the principal hydrologic feature in the surrounding area.

The Susquehanna River drains a basin of approximately 27,510 square miles (mi^2), covering a large part of Pennsylvania and portions of New York and Maryland (***Figure 1-1***). The river flows approximately 444 miles from its headwaters at Otsego Lake in Cooperstown, New York to Havre de Grace, Maryland, where the river discharges to the Chesapeake Bay (UniStar, 2013; Section 2.4.1.2.1.1).

There are several small tributaries of the Susquehanna River in the vicinity of the site, which drain localized areas into the main river channel. Many of these tributaries are unnamed, with the exception of Walker Run, located to the west of the SSES site (***Figure 1-2***). Walker Run drains a catchment of approximately 4.3 mi^2 (UniStar, 2013; Section 2.4.3) and discharges into the Susquehanna River approximately two miles southwest of the site, as shown on ***Figure 1-3***.

The Susquehanna River flows from the northeast to the southwest in the general location of the site, flowing through the wind gap at Lee Mountain in an almost due south direction along an approximately five mile reach to the east of the site, before returning to its previous course (***Figure 1-2***).

The two closest population centers in the surrounding area are Wilkes-Barre, PA, approximately 21 miles upstream, and Berwick, PA, approximately two miles downstream (***Figure 1-2***), with populations of 41,108 and 10,343 in 2013, respectively, as estimated by the United States Census Bureau (USCB) (USCB, 2014a) and (USCB, 2014b).

The topography of the Middle Susquehanna River subbasin is characterized by high flat-topped plateaus separated by steep sided valleys. Approximately four miles north of the site, the Lee Mountain ridge runs west to east across the Susquehanna River floodplain (***Figure 1-2***). The ground levels fall steeply to the south of the ridge, down to a relatively flat plain with rolling hills, on which the site is situated.

SSES is considered to be a "dry site" (PPL, 2013; Section 2.4.2.1) in accordance with the definitions contained in Regulatory Guide 1.102: Flood Protection for Nuclear Power Plants (NRC, 1976). As defined by the Regulatory Guide, a dry site is a site where the plant is built above the design basis flood level, and, therefore, safety-related Structures, Systems, and Components (SSCs) are not affected by flooding.

The lowest plant grade elevation on the site is approximately 670 feet (ft) Mean Sea Level (MSL). The lowest SSES grade elevation is approximately 175 ft above the Susquehanna River floodplain adjacent to SSES (about one mile east of the site) (PPL, 2013; Section 2.4.1.1).

The vertical datum used in the SSES FSAR is presented as MSL. At the site, the MSL datum is equated with the National Geodetic Vertical Datum of 1929 (NGVD29), which, prior to 1973, was referred to as the Sea Level Datum of 1929 (National Oceanic and Atmospheric Administration [NOAA], NOAA, 2014). Throughout this FHRR, all elevations are provided in NGVD29, unless stated otherwise, in order to facilitate a direct comparison with elevations provided in the SSES FSAR (PPL, 2013).

2.0 FLOOD HAZARDS AT THE SITE

Section 2.0 has been prepared in response to Request for Information Item 1.a. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (NRC, 2012a). This Section documents Current Licensing Basis (CLB) results, as well as pertinent site information related to the applicable flood hazards. Relevant SSCs important to safety and the Ultimate Heat Sink (UHS) are included in the scope of this reevaluation, including pertinent data concerning these SSCs.

2.1 DETAILED SITE INFORMATION

Section 2.1 has been prepared in response to Request for Information Item 1.a.i. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (NRC, 2012a). Relevant site data presented for consideration include the present-day site layout, elevation of SSCs important to safety, and site topography, as well as pertinent spatial and temporal data sets.

2.1.1 Design/As-Built Site Information

Design site information describes characteristics considered for the original licensing basis of the SSES site. Changes to the site layout and SSCs related to flooding protection were evaluated as part of the Near-Term Task Force Recommendation 2.3 Flood Walkdown Submittal Report for SSES (PPL, 2012), (subsequently referred to as the Walkdown Report). These changes were evaluated as part of this hazard reevaluation report with respect to new guidance and methodologies.

The design/as-built topographic mapping and site layout for the SSES site is shown on ***Figure 2-1*** and supplemented by aerial photography (***Figure 1-3***). The locations of the selected safety-related SSCs and other relevant buildings are shown on ***Figure 2-2***, with the design elevations presented in ***Table 2-1***. A list of the existing licensing basis parameters found in the SSES FSAR (PPL, 2013) is presented in ***Table 2-2.***

2.1.1.1 Design/As-Built Site Topography

The SSES FSAR (PPL, 2013; Section 2.4.1.1) states that the lowest plant grade elevation of the SSES site is 670 ft. The site is located upon a raised plateau, which gently slopes from west to east. To the east of the site, the elevation falls quickly towards the Susquehanna River, which is located approximately one mile to the east of the site.

2.1.1.2 Description of Design/As-Built Safety-Related Structures, Systems, and Components

A list of safety-related structures and their elevations is presented in ***Table 2-1***. The locations of these safety-related SSCs and other relevant buildings at the SSES site are shown on ***Figure 2-2***.

2.1.1.3 Description of the Design/As-Built Ultimate Heat Sink

The UHS for the SSES site consists of the Spray Pond, which is located in the northwest corner of the site as shown on ***Figure 2-2***. The Spray Pond is designed to remain functional under the most adverse hydrometeorological and seismic conditions.

The normal operating water level in the Spray Pond of 679 ft (resulting in a water depth of approximately 11 ft) is maintained by (1) rainfall (primary); (2) separate make-up, pumped from the Susquehanna River through a pipe adjacent to the Engineered Safeguards Service Water (ESSW) Pumphouse (secondary); and (3) Cooling Tower blowdown (backup as required) (PPL, 2013; Section 2.4.8.2).

The normal operating water level is controlled by an overflow weir in the ESSW Pumphouse with a crest elevation of 678.5 ft, and an uncontrolled spillway located at the east end of the pond with an elevation of 680.5 ft (PPL, 2013; Section 2.4.8.2). The uncontrolled spillway discharges to a channel that passes beneath the railway embankment via four 6 ft by 3 ft box culverts (***Figure 2-2***) and then discharges into a natural watercourse tributary to the Susquehanna River (***Figure 1-3***).

The SSES FSAR presents the design basis for the Spray Pond, which states that the design basis flood level for the ESSW Pumphouse is 684.8 ft (PPL, 2013; Table 2.4-14, Section 2.4.8.4.1), including the effects of coincident wind-wave activity (Section 2.2.1.1). All safety-related equipment in the pumphouse is located at or above the floor level elevation of 685.5 ft.

2.1.2 Present-Day Site Information

Section 2.1.2 provides a discussion of present-day site information and describes changes to the SSES site that have occurred since the original design of the site. Changes to hydrologically and hydraulically significant plant features may influence the reevaluation of flooding hazards; therefore, the reevaluation is based on present-day site information.

2.1.2.1 Present-Day Site Topography

Detailed 2006 aerial topographic mapping from the Pennsylvania Spatial Data Access (PASDA) was used to represent the present-day topography for the site and surrounding area (i.e., to assign ground surface elevations for modeling purposes). It is noted that the Walkdown Report (PPL, 2012) does not document any major changes to site topography since 2006. An additional plant walkdown was used to confirm that no major changes have been made to the site topography since the 2006 survey. The 2006 survey is, therefore, understood to represent the present-day site conditions. This topographic data is illustrated on ***Figure 2-3***.

The digital map data obtained from PASDA includes a 3.2 ft resolution digital elevation model (DEM) based on Light Detection and Ranging (LiDAR) data. The horizontal datum associated with the DEM is the U.S. Pennsylvania State North 1983. The vertical datum is NAVD88, which was converted to NGVD29 (NOAA, 2013) to facilitate comparison between the reevaluation analysis and the SSES FSAR analysis (PPL, 2013).

The ground elevations on the site generally slope from west to east, before falling more quickly towards the Susquehanna River to the east of the site. A steep hillside is located to the north of the site, and a ditch containing a small tributary of the Susquehanna River flows west to east near the northern boundary of the site (***Figure 1-3***), intercepting runoff flow from the hillside to the north.

A comparison between the design/as-built topography shown on ***Figure 2-1*** and the present-day topography on ***Figure 2-3*** suggests there have been no significant topographic changes in the immediate vicinity of the SSES site.

2.1.2.2 Description of Present-Day Safety-Related Structures, Systems and Components

Changes to the site layout and SSCs related to flooding protection were noted as part of the Walkdown Report (PPL, 2012). Flood barrier drawings were created by PPL to establish external flood ratings around safety-related structures. The drawings were incorporated by reference in the SSES FSAR (PPL, 2013; Section 2.4.2.2).

The list of safety-related buildings and corresponding design floor elevations for the CLB are presented in ***Table 2-1***. The SSES site incorporates flood barriers, which provide passive flood protection of safety-related SSCs (PPL, 2012). ***Table 2-1*** also includes the heights of the flood barriers and the credited flood protection elevation for relevant safety-related SSCs.

2.1.2.3 Description of Present-Day Ultimate Heat Sink

The design criteria for the Spray Pond has not changed (***Section 2.1.1.3***). No changes in the Spray Pond since license issuance are noted in the SSES FSAR (PPL, 2013) or the Walkdown Report (PPL, 2012).

2.2 CURRENT LICENSING BASIS FLOOD ELEVATIONS

Section 2.2 has been prepared in response to Request for Information Item 1.a.ii. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (NRC, 2012a). The CLB flood elevations for all flood-causing mechanisms are summarized in ***Table 2-3***. Corresponding building floor elevations are included in ***Table 2-1***. The following subsections outline the CLB flooding analyses and results.

2.2.1 Probable Maximum Precipitation

For the CLB, effects of Local Intense Precipitation (LIP) were investigated to determine whether the Probable Maximum Precipitation (PMP) event would result in flooding of safety-related buildings.

The SSES FSAR (PPL, 2013; Section 2.4.2.3) determined the all-season 24-hour PMP, and the maximum 6-hour precipitation was then disaggregated into half hour increments. For storms with a duration of less than 30 minutes, National Weather Service (NWS) ratios were used to determine the appropriate rainfall increments.

The grading and natural topography of the SSES site were credited with directing runoff away from safety-related buildings via a system of culverts, drainage channels, and underground storm drains (PPL, 2013; Section 2.4.2.3). However, all culverts and underground drains, except for the culverts beneath the railway that connect to the uncontrolled spillway on the Spray Pond, were assumed to be blocked.

Using the PMP depths and storm durations, rainfall intensities were determined and corresponding flow rates for the SSES site were then calculated using the "rational" method. Cross sections were taken across the site at 'check locations,' where the flow depth was evaluated using Manning's equation, to determine whether the PMP flow could be contained within the drainage channels (PPL, 2013; Section 2.4.2.3).

The SSES FSAR (PPL, 2013; Section 2.4.2.3) states that "pressure resistant doors are provided to prevent flood water from reaching safety-related equipment in the event of localized ponding adjacent to the power block." The SSES FSAR, therefore, concludes that the possibility of flooding of any safety-related facility due to local PMP is precluded; however, no specific design flood levels or depths are provided in the SSES FSAR for locations adjacent to safety-related buildings.

The potential impact of the local PMP event with coincident wind-wave run-up on the Spray Pond is considered in the SSES FSAR (PPL, 2013; Section 2.4.8.4.1), which describes the design basis for the Spray Pond. The wave run-up was calculated for the following three scenarios:

- A sustained wind of 40 miles per hour (mph) and the Probable Maximum Flood (PMF)
- The worst wind of record at Avoca Airport (65 mph) and the Standard Project Flood (SPF), defined as half of the PMF
- A probable maximum gradient wind (85 mph) and the 10-year flood

The maximum run-up elevation resulted from the worst wind of record and the SPF Spray Pond level, which produced run-up elevation levels of 684.8 ft and 684.6 ft at the ESSW Pumphouse and the side of the Spray Pond, respectively (PPL, 2013; Table 2.4-14).

2.2.2 Probable Maximum Flood on Rivers and Streams

The SSES site is considered to be a "dry site" (***Section 1.3***). Conservative assumptions and baseline conditions were adopted to maximize the calculated PMF water surface elevation for the Susquehanna River adjacent to the site (PPL, 2013; Section 2.4.3).

The SSES FSAR used the PMP characteristics derived by a 1965 U.S. Department of Commerce Weather Bureau study of the PMP for the Susquehanna River upstream of Harrisburg, which was based on U.S. Army Corps of Engineers (USACE) data and methodologies (PPL, 2013; Section 2.4.3.1).

The USACE HEC-1 Flood Hydrograph Package computer program was subsequently used to calculate the Susquehanna River PMF flow, adjacent to the site as 1,100,000 cubic ft per second (cfs) (PPL, 2013; Section 2.4.3.4). The alternative method of analysis presented in Regulatory Guide 1.59 (NRC, 1977) was also applied to verify that the calculated PMF flow value of 1,100,000 cfs was appropriate.

Hydraulic backwater curve calculations were then completed using USACE methods to determine the PMF flood level elevation in the Susquehanna River for the specified flow rate. The calculations used discharge records from historic floods and topographic survey of the river channel cross sections in the vicinity of the site. The Manning's "n" roughness values were modified to calibrate the flood level to a historic flood profile (PPL, 2013; Section 2.4.3.5.1).

During a PMF event, it was considered likely that the volume of water flowing downstream would cause any river crossings to be washed away. The Berwick Highway Bridge crosses the Susquehanna River approximately seven miles downstream of the site. The backwater calculations were completed both with and without the presence of the bridge, to ensure that any additional head losses and corresponding increase in flood water level elevations were accounted for within the calculation.

The calculations were completed in accordance with standard procedures defined by the USACE as described in the SSES FSAR (PPL 2013; Section 2.4.3). When the bridge is considered to be washed out by the flood, the maximum water level adjacent to the site is 544.8 ft, and when the bridge remains intact the maximum water level is 545.7 ft (PPL 2013; Section 2.4.3.5.2).

In order to include appropriate conservative assumptions throughout the determination of flood hazards, the SSES FSAR considers the PMF scenario when the Berwick Highway Bridge remains intact with an elevation of 545.7 ft.

The SSES FSAR (PPL, 2013; Section 2.4.3.6) states that the CLB for river flooding includes consideration of coincident wind-wave activity in addition to the PMF stillwater level elevation on the Susquehanna River. The wave setup and run-up effects were estimated assuming a crossriver 5,000 ft fetch, with an average depth of approximately 45 ft and a wind speed of 45 mph along the fetch.

This resulted in a total wave setup and run-up of 2.3 ft, in addition to the PMF stillwater level of 545.7 ft, giving a total PMF elevation of 548 ft, providing a freeboard of approximately 120 ft between the PMF elevation and the plant grade level of 670 ft.

The SSES FSAR (PPL, 2013; Section 2.4.3.6) also includes consideration of coincident wind-wave activity during floods of a more frequent nature, and for the 100-year flood level calculates the maximum supportable wave height, i.e., the breaking wave height and its associated run-up.

The 100-year flood level of 513.6 ft results in a total water depth of approximately 33.6 ft in the Susquehanna River, and a corresponding maximum wave height of 26.2 ft, with associated wave run-up of 5.2 ft. Based on the conservative assumptions applied, the SSES FSAR states that the total water level during the 100-year flood level with coincident wind-wave activity is conservatively estimated as 539.8 ft (PPL, 2013; Section 2.4.3.6).

The 100-year flood level with coincident wind-wave activity is more than 130 ft below the plant grade elevation of the site. The level is also below the PMF level of 548 ft; therefore, the SSES FSAR retains this PMF level as the design basis flood level, which accounts for the effects of coincident wind-wave activity, as the CLB for the SSES site.

2.2.3 Potential Dam Failures (Seismically Induced)

The SSES FSAR considers a range of dam failure scenarios, including singular dam failures and multiple dam failures within the Chemung River, Susquehanna River, and Lackawanna River basins located upstream of the site (***Figure 1-1***). The original analysis assumed instantaneous failure of all fourteen dams located upstream of the SSES site (PPL, 2013; Section 2.4.4). At the time of the original analysis, the dam failures considered included six operational dams, two dams that were under construction, and six dams that were authorized for future construction.

Since the original analysis was completed, the six dams authorized for future construction have been deauthorized and the SSES FSAR states there are no plans to construct any of the deauthorized dams in the foreseeable future. However, the original analysis (including assumed failure of the six dams that have now been deauthorized) has been retained as the most conservative bounding case for dam failure at the SSES site (PPL, 2013; Section 2.4.4).

The SSES FSAR states that multiple dam failures throughout the Chemung River and Susquehanna River basins would result in flow rates of approximately 54 and 56 percent of the PMF flow in the Susquehanna River adjacent to the site. Detailed consideration of multiple dam failures on the Lackawanna River basin is not considered necessary, because their combined storage volume is less than five percent of the storage volume of the Susquehanna River basin dams.

The SSES FSAR does not state the calculated flow due to the postulated simultaneous failure of fourteen dams upstream of the SSES site (both existing and, at the time of the original analysis, authorized for construction). However the SSES FSAR concludes that the resulting water level would be more than 120 ft below the site elevation (PPL, 2013; Section 2.4.4.3).

This demonstrates that the peak flood level resulting from any dam failure scenario, incorporating highly conservative assumptions, remains less than the PMF water levels, i.e., flooding from dam failure is bounded by flooding from rivers and streams.

2.2.4 Coastal Flooding

Storm surge, seiche flooding, and tsunami flooding were screened out as potential flooding mechanisms in the SSES FSAR (PPL, 2013; Sections 2.4.5 and 2.4.6, respectively).

The SSES FSAR states that flooding due to propagation of storm surge to the site (a distance of 165 miles upstream on the Susquehanna River from the Chesapeake Bay) is not applicable. The SSES FSAR also states that consideration of seiche flooding potential is not applicable, as the Susquehanna River is the only major water body in the vicinity of the site (PPL, 2013; Section 2.4.5).

The SSES FSAR also states that tsunami flooding is not applicable to the SSES site (PPL, 2013; Section 2.4.6).

2.2.5 Ice-Induced Flooding

Ice-induced flooding was screened out as a potential flooding mechanism in the SSES FSAR (PPL, 2013; Section 2.4.7). The SSES FSAR considers historic incidents of ice jam related flooding on the Susquehanna River and concludes that maximum flood levels experienced are less than the PMF level.

The SSES FSAR, therefore, concludes that any potential impacts due to ice-induced flooding are bounded by the analyses of LIP, and flooding from rivers and streams (PPL, 2013; Section 2.4.7).

2.2.6 Flooding Resulting from Channel Migration or Diversion

Flooding resulting from channel migration or diversion was screened out as a potential flooding mechanism in the SSES FSAR (PPL, 2013; Section 2.4.9). The SSES FSAR states that the Susquehanna River possesses a stable stream course flowing through well-defined ridge and valley topography, which is not subject to major meandering realignment or diversion.

2.2.7 Cooling Tower Basins Rupture

The SSES FSAR (PPL, 2013; Section 2.4.2.2) states that in the event of a Cooling Tower basin rupture or failure, flooding would occur on the western side of the Turbine Building. This flooding could potentially impact safety-related areas including the Control Structure and the western side of the Reactor Building (***Figure 2-2***). Additionally, the SSES FSAR analysis conservatively allows for the possibility of water reaching the ESSW Pumphouse and the east side of the powerblock by passing both through and around buildings.

The SSES FSAR (PPL, 2013; Section 2.4.2.2) describes that passive flood protection measures (i.e., flood barriers) prevent water from entering safety-related features during postulated external flooding events, including Cooling Tower basin rupture and PMP event scenarios, as discussed in ***Section 2.5***.

Peak external flooding levels and ponding depths resulting from Cooling Tower basin rupture are not specifically presented within the SSES FSAR (PPL, 2013).

2.3 FLOOD-RELATED CHANGES TO THE LICENSING BASIS

As noted in ***Section 2.1.2.2***, flood barrier drawings were created to establish external flood ratings around safety-related buildings. These flood barrier drawings establish conservative maximum external flood ratings around safety-related buildings. The drawings are now referenced in the SSES FSAR (PPL, 2013; Section 2.4.2.2). There are no known flood-related changes that have been identified for the SSES site since determination of the CLB outlined in the SSES FSAR (PPL, 2013).

2.4 Changes to the Watersheds and Local Site Area

The CLB assessment of dam failure (***Section 2.2.3***) includes consideration of a total of fourteen dams. Six of these dams have subsequently been deauthorized for construction; however, the original analysis has been retained as a conservative bounding case for dam failure.

Changes to the local site area include modification of land use due to plant activities (e.g., some additional areas have been paved and a VBS has been installed). These changes are accounted for in the updated flooding analyses documented in this report (***Section 3.0***).

2.5 Current Licensing Basis Flood Protection and Pertinent Flood Mitigation Features

Section 2.5 has been prepared in response to Request for Information Item 1.a.v of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (NRC, 2012a). Relevant site data to be considered include CLB flood protection and pertinent flood mitigation features at the site.

The SSES site flood protection features are identified in the Walkdown Report (PPL, 2012), and condition reports were generated for those flood protection features (e.g., penetrations/doors) that were found to be degraded during the flooding walkdowns. Note that flood barrier drawings discussed in the Walkdown Report (PPL, 2012) were subsequently referenced in the SSES FSAR (PPL, 2013; Section 2.4.2.2). All flood barriers credited in the CLB are also credited for the purposes of this flood hazard reevaluation analysis (***Section 3.0***).

The SSES site flood protection features are classified as "Incorporated Barriers" based on the guidance contained in Nuclear Energy Institute (NEI) Guidelines for Performing Verification Walkdowns of Plant Flood Protection Features (NEI, 2012). These include permanently installed features, which protect safety-related SSCs from the effects of external flooding events.

In addition, "Exterior Passive" features, as defined in the NEI (2012) guidelines, such as normally closed external flood doors, exterior wall penetrations, and openings within the exterior walls of safety-related structures are credited as external flood protection features. The features are considered to be passive features, since they are not required to function (i.e., doors are designed to remain normally closed, and walls are designed to remain in place) during flood events.

External flood barriers are located in the Unit 1 and Unit 2 Reactor Buildings, the Common Diesel Generator Building, Common Diesel Generator 'E' Building, and the ESSW Pumphouse (***Figure 2-2***). The height of the flood barriers and the credited flood protection level at relevant safety-related SSCs are presented in ***Table 2-1***. Further detailed descriptions of each feature are included in the Walkdown Report (PPL, 2012) and design drawings, which state the barrier heights and credited flood protection levels.

2.6 ADDITIONAL SITE DETAILS

The SSES incorporates a VBS around the perimeter of the site (***Figure 2-3***) to prevent unauthorized vehicular access. The VBS consists of a combination of concrete Jersey Barriers, continuous concrete walls, and removable railroad blocks, which are tied in to high ground (PPL, 2003). The effects of the VBS are considered in the flood hazard reevaluations presented herein (***Section 3.0***).

3.0 FLOOD HAZARD REEVALUATION ANALYSIS

In response to the NRC 10 CFR 50.54(f) letter (NRC, 2012a) requesting information from all nuclear power reactor licensees, ***Section 3.0*** provides the results of the flood hazard reevaluation for the SSES site, addressing each applicable flood-causing mechanism, as detailed in Enclosure 2, Information Item 1.b. The basis for all inputs, assumptions, methodologies and models used, including input and output files, are summarized in this report and included in detail with the supporting analyses.

The flood-causing mechanisms potentially impacting the site include Cooling Tower basin ruptures and LIP (including coincident wind-wave activity). All other potential flooding mechanisms are screened out as credible sources of flooding at the SSES site, with appropriate justification provided for each screening. The sources of flooding that are screened out include river flooding, dam breaches and failures, storm surge, seiche, tsunami, channel migration or diversion, ice-induced flooding, and combined effects flooding.

3.1 SUMMARY OF RECOMMENDATION 2.1

To respond to Phase 1 of NRC Recommendation 2.1 (NRC, 2012a) and the 2012 Consolidated Appropriations Act (CAA) (CAA, 2012; Section 402), the NRC requested that each licensee provide a reevaluation of all appropriate external flooding sources, including the effects of LIP on the site, PMF on rivers and streams, storm surges, seiches, tsunamis, dam failures, ice-induced flooding, channel migration or diversion, and combined effects flooding. It was requested that the reevaluation should apply present-day regulatory guidance and methodologies being used for Early Site Permit and Combined Operating License (COL) reviews, and that the reevaluation should employ current techniques, software, and methods used in present-day standard engineering practice to develop the flood hazard reevaluation analysis.

3.2 SOFTWARE USED

The following software was used to perform the flood hazard reevaluation analyses: FLO-2D Pro (FLO-2D, 2012), ArcGIS 10.1 (ESRI, 2012), and USACE HEC-SSP 2.0 (USACE, 2010a) and HEC-RAS 4.1 (USACE, 2010b).

The FLO-2D Pro software is a volume conservation model that routes fluid flow in one-dimensional (1D) channel flow and/or two-dimensional (2D) overland flow, including interaction between 1D and 2D flow (FLO-2D, 2012). The FLO-2D Pro software is an "effective tool for delineating flood hazards or designing flood mitigation" (FLO-2D, 2012). The FLO-2D basic model is the other model available from FLO-2D Software, Inc., and includes fewer features than the Pro model.

The basic model has been approved by the Federal Emergency Management Agency (FEMA) for use in Flood Insurance Studies (FIS). The Pro model, though not specifically approved by FEMA, includes all the features of the basic model, along with some additional features (e.g., storm drain interface with surface water using the Environmental Protection Agency's SWMM program, parallel processing capabilities, and expanded capabilities for simulating sediment transport [FLO-2D, 2012]).

The ArcGIS (ESRI, 2012) interface is used for analysis of topographic data and cartographic mapping, in addition to preparation and processing of the FLO-2D input and output files, respectively. At the time of the writing of this report, the use of ArcGIS is deemed acceptable to the NRC as it is referenced repeatedly in their design basis guidelines (e.g., NRC, 2011).

HEC-SSP (USACE, 2010a) is designed to perform standard statistical analyses of hydrologic data. The software program allows the user to undertake data analysis through application of numerous statistical distributions, to generate outputs including annual peak flood frequency curves and exceedance frequency relationships. The program is used for statistical analyses of data for screening of the river flooding and combined effects flooding scenarios.

HEC-RAS (USACE, 2010b) is designed to perform 1D hydraulic calculations for a full network of natural and constructed channels. It was used to simulate the outflow from Cooling Tower basins during failure scenarios, and develop boundary conditions for input to the FLO-2D model.

All software used to perform the flood hazard reevaluation analyses has been Verified and Validated (V&V) and commercially dedicated in accordance with a quality assurance program that meets the requirements of 10 CFR, Part 50 (10 CFR 50) Appendix B, through compliance with the Basic and Supplementary Requirements established in Part I of American Society of Mechanical Engineers (ASME) NQA-1-2008 and NQA-1a-2009 Addenda (ASME, 2009), as well as the following Subparts that apply to the previously identified software:

- Subpart 2.7, “Quality Assurance Requirements for Computer Software for Nuclear Facility Applications.”
- Subpart 2.14, “Quality Assurance Requirements for Commercial Grade Items and Services.”

3.3 FLOOD-CAUSING MECHANISMS

NRC NUREG/CR-7046 (NRC, 2011) recommends using a Hierarchical Hazard Assessment (HHA) method for evaluating the safety of SSCs. The HHA method is a progressively refined process of stepwise estimation of site-specific hazards that starts with the most conservative plausible assumptions consistent with available data. The HHA process proceeds for each flood-causing mechanism to be reanalyzed. This method can be summarized as follows (NRC, 2011):

1. Develop a conservative estimate of the hydrologically relevant site-related parameters using simplifying assumptions for the flood-causing mechanism and estimate new flood elevations using the appropriate modeling approach.
2. Compare the reevaluated flood hazard elevation (from step 1) with the original design flood elevation for the selected flood-causing mechanism. If the newly-calculated flood elevation is lower, it is used for comparison against the CLB for the reevaluation of this causal mechanism.
3. If not lower, determine if the parameterization of site hydrology can be further refined. If yes, perform reevaluation (repeat step 2). If not, use the flood elevation from the previous step for this causal mechanism for comparison of reevaluation against the CLB.
4. If all flood-causing mechanisms have not been addressed, select another flood-causing mechanism and proceed to step 1.

For each flood-causing mechanism, the final flood elevations from the hazard reevaluation were compared with the CLB flood elevations to determine whether the CLB flood bounds each reevaluated hazard.

The methodology described above was used to evaluate the potential flooding effects resulting from each potential flood-causing mechanism relevant to the SSES site using present-day methodologies and regulatory guidance. Details regarding the considerations and results of the analyses for each flood-causing mechanism are presented in the following subsections of this report:

- Section 3.3.1 – Local Intense Precipitation
- Section 3.3.2 – Probable Maximum Flood on Rivers and Streams
- Section 3.3.3 – Dam Breaches and Failures
- Section 3.3.4 – Coastal Flooding
- Section 3.3.5 – Ice-Induced Flooding
- Section 3.3.6 – Flooding Resulting from Channel Migration or Diversion
- Section 3.3.7 – Combined Effects Flooding
- Section 3.3.8 – Cooling Tower Basin Rupture

3.3.1 Local Intense Precipitation

Sections 3.3.1.1 through 3.3.1.4 address the effects of LIP at the SSES site. A flow chart of the HHA screening methodology for the LIP flooding analysis, based on guidance developed in NUREG/CR-7046 (NRC, 2011), is shown on ***Figure 3-1***.

3.3.1.1 Local Intense Precipitation Hyetograph

NUREG/CR-7046 (NRC, 2011; Section 3.2) states that "Local Intense Precipitation is a measure of the extreme precipitation at a given location." The LIP is "deemed equivalent to the 1-hour, 2.56-km^2 (1 mi^2) PMP at the location of the site" (NRC, 2011; Section 3.2).

An LIP hyetograph was developed based on the 1-hour 1 mi^2 PMP depth of 17.5 inches documented in the proposed Bell Bend Nuclear Power Plant (BBNPP) FSAR (UniStar 2013, Table 2.4-18), which is located approximately one mile southwest of the site. The BBNPP FSAR utilized the HMR-52 methodology to determine the PMP value. This methodology is the recommended approach specified by NRC (2011). The HMR-52 methodology is unable to distinguish between locations less than one mile apart, therefore, it is appropriate to utilize the PMP value from the BBNPP for the SSES Site.

Three individual LIP temporal distributions were developed by varying the timing of the peak rainfall intensity as follows: (a) at the start of the LIP event; (b) in the middle of the LIP event; and, and (c) after two-thirds of the LIP event. Flooding simulations were completed for all three temporal distributions (***Section 3.3.1.2***), which demonstrated that the two-thirds (c) distribution resulted in the highest flood depth at safety-related SSCs. The 1-hour LIP hyetograph for the two-thirds (c) distribution is shown on ***Figure 3-2***.

3.3.1.2 Effects of Local Intense Precipitation

In accordance with the guidance presented in NRC NUREG/CR-7046 (NRC, 2011), the effects of LIP have been evaluated at the SSES site. The analysis addressed the following LIP flooding characteristics:

- Depth of Flooding
- Duration of Flooding
- Maximum Velocities
- Hydrodynamic and Hydrostatic Loads
- Sedimentation
- Debris Loading

Each of these considerations was evaluated based on the results of 2D flow modeling in FLO-2D to simulate runoff from the local SSES site. The output of the FLO-2D model includes water surface elevations, water depths, maximum water velocities, and the duration of flooding. FLO-2D also computes the hydrostatic and hydrodynamic forces that the floodwater could exert on obstacles (e.g., buildings) within flooded areas. The results from FLO-2D are used to evaluate LIP flooding effects in accordance with the guidance presented in NRC NUREG/CR-7046 (NRC, 2011). The potential for sedimentation and debris loading was qualitatively evaluated, based on the interpretation of FLO-2D output of depths, maximum velocities, and flow directions.

The FLO-2D model boundaries (***Figure 3-3***) were established at a sufficient distance from the power block area and safety-related SSCs to prevent boundary conditions from affecting the evaluated flood levels within the power block area, and to ensure the stability of the model. The domain of the FLO-2D model was developed to represent site conditions reflected in site aerial topographic mapping. The boundaries of the FLO-2D domain were primarily established along drainage divides (e.g., the centerlines of roads, berms, and ridges).

Consistent with established FLO-2D methodology, boundary conditions include mechanisms through which water enters or leaves the model domain. These mechanisms include lateral outflow through the model boundaries, rainfall applied directly to the FLO-2D grid cells, and infiltration that removes water from the model domain. Lateral outflow conditions along all boundaries allow simulation of runoff from the FLO-2D domain without distorting the flow patterns within the model. The rainfall hyetographs discussed in ***Section 3.3.1.1*** were applied as direct rainfall in FLO-2D, and for the more refined simulations where infiltration was characterized for pervious areas surrounding the power block, the Soil Conservation Service (SCS) Curve Number method (FLO-2D, 2012) was applied, based on site-specific soil properties and land use classifications.

The FLO-2D model characterized topographic and man-made features that affect runoff from the SSES site, including the VBS. Buildings, tanks, and other structures were characterized within FLO-2D as flow obstructions. The VBS was characterized as a "levee" (i.e., a wall with a specified crest elevation). As a conservative assumption, the seven pedestrian access points in the VBS (***Figure 2-3***) were blocked (i.e., water was not allowed to flow through the access points). This blocking effect was intended to simulate blocking by potential debris carried away from the power block by runoff during the LIP simulation. Thus, any potential backwater effects of debris jams during the LIP event were accounted for in the FLO-2D model. A sensitivity analysis described below evaluated the significance of blocking the pedestrian access points in the VBS with respect to flood levels near safety-related SSCs. All culverts on the SSES site were considered blocked for this analysis (including the box culverts at the uncontrolled spillway for the Spray Pond [***Figure 2-2***]).

The HHA methodology for evaluating LIP flooding is shown on ***Figure 3-1***. As noted in ***Section 3.3***, the HHA methodology consists of iterative calculations starting with conservative modeling assumptions and progressively refining the inputs and assumptions. Five basic cases were developed for the SSES site, and additional subcases were evaluated to determine the sensitivity of model results to the time of peak rainfall intensity and unblocking the VBS pedestrian access points. The results of the subcases were reviewed to determine appropriate model refinements for the subsequent cases.

All of the cases used a computational grid cell size of 10 ft by 10 ft to ensure consistent representation of topographic features on the site. The cases and subcases are summarized as follows (additional simulation details are contained in the supporting analyses):

- Case 1 was a steady state simulation (i.e., constant rainfall at the peak intensity) with assumed high Manning's roughness coefficients (within published recommended ranges), no infiltration losses, and blockage of the VBS pedestrian access points.

- Case 2 included infiltration losses and a time-varying LIP distribution. Three subcases were considered (2a, 2b, and 2c), which varied the timing of the peak rainfall intensity as follows: (2a) at the start of the LIP event; (2b) in the middle of the LIP event; and, and (2c) after two-thirds of the LIP event. The high Manning's roughness coefficients were retained.
- Case 3 applied the LIP distribution that resulted in the highest flood depths on site (Case 2c), and represented existing site conditions by unblocking the seven VBS pedestrian access points. The high Manning's roughness coefficients were retained.
- Case 4 applied a lower, more representative set of Manning's roughness coefficients (within published recommended ranges), and considered two subcases (4a and 4b), which represented the VBS access points as blocked and unblocked, respectively.
- Case 5 considered the VBS access points to be blocked, and conservatively represented the three vehicular access doors on the west side of the Turbine Building as fully open, in order to calculate the inflow volume to the building during an LIP event.

The results of the sensitivity analyses were reviewed, and it was determined that the LIP distribution with peak rainfall intensity occurring at the two-thirds point of the 1-hour LIP event (Case 2c) resulted in the highest flood depths on the SSES site. The variation in maximum flood depth due to the timing of peak intensity was approximately 0.6 ft at safety-related SSCs. The variation in maximum depth due to changes in Manning's roughness coefficients was approximately 0.2 ft at safety-related SSCs, and it was determined that the lower valued coefficients (Case 4a) were more representative of site topography and land cover.

The impact of debris blockages at the VBS pedestrian access points was determined to be negligible, with no discernible change in maximum flood depths. As it is reasonable that debris from the site could cause blockage of the access points during an LIP event, the most refined simulation (Case 5) assumed that the access points were blocked, in conjunction with peak rainfall intensity occurring at the two-thirds point in the 1-hour LIP event, and the lower, more representative roughness coefficients.

Case 5 is used as the most refined simulation for reporting results near safety-related SSCs. It should be noted that Case 5 represented the vehicular access doors to the Turbine Building in the open position. Although the Turbine Building is not safety-related, these doors represent one potential path for water to access safety-related SSCs; i.e., if the inflow to the Turbine Building were to exceed the available storage, floodwater could be conveyed to the Reactor Buildings.

Case 5 represented the vehicular access doors on the west side of the Turbine Building in the open position in order to calculate the volume of inflow to the Turbine Building. It should be noted that opening the doors to the Turbine Building does not affect flood levels near safety-related SSCs, because the Turbine Building doors are on the opposite side of the plant from the safety-related SSCs. The negligible impact of flow through these doors on water levels near safety-related SSCs was verified by comparing the model results of Case 4a with the results of Case 5 (two simulations that differ only by whether or not the Turbine Building doors are open). This comparison indicated negligible impact on flood levels near safety-related SSCs due to opening the Turbine Building doors.

The localized maximum ponding depths reported for the most refined simulation (Case 5) are shown on ***Figure 3-3*** and peak flood levels near structures are presented in ***Table 3-1***. The maximum external ponding depths adjacent to safety-related structures range from 1.61 ft at the east side of the Unit 1 and Unit 2 Reactor Buildings, to 0.31 ft at the south side of the ESSW Pumphouse. The maximum LIP flood depths do not result in internal flooding at any safety-related SSCs, due to the existing flood barriers (***Table 2-1***) as described in the Walkdown Report (PPL, 2012). ***Table 3-1*** also presents the freeboard between the maximum LIP flood level and the flood barrier level, which shows the available freeboard ranges from a minimum value of 0.64 ft at the Unit 1 Reactor Building, to a maximum value of 9.09 ft at the ESSW Pumphouse. Due to the shallow depths near safety-related SSCs, hydrostatic and hydrodynamic forces are small (i.e., maximum values at safety-related SSCs of 81 and 26 pounds/ft, respectively), and no detailed structural analysis of safety-related SSCs is required.

The Turbine Building is not a safety-related structure; however, this building could potentially convey flood water to the (safety-related) Reactor Buildings, if the inflow to the building were to exceed the available storage volume within the Turbine Building. As noted in ***Table 3-1***, approximately 9.9 million gallons of storage is available within the Turbine Building before water could rise to a sufficient elevation and convey flow into the Reactor Buildings. The maximum inflow to the Turbine Building during the LIP event is calculated as approximately 2.5 million gallons, which can be contained within the building and would, therefore, not impact the Unit 1 and Unit 2 Reactor Buildings.

The maximum still water level in the Spray Pond during the LIP event was calculated as 682.02 ft as presented in ***Table 3-1***, which is below the operating floor level of the ESSW Pumphouse of 685.5 ft (PPL, 2013; Section 2.4.8.4.1).

3.3.1.3 Wind-waves and Run-up Coincident with Local Intense Precipitation

Wave run-up is the process whereby wind-generated waves impinge on a structure or embankment and cause intermittent flow of water up the side of the structure or embankment. In general, the impact of wave action increases with the speed of the wind, the depth of the water over which it acts, and the length over which the wind blows (i.e., the "fetch").

Wave run-up effects were initially considered for all areas of the SSES site inundated during an LIP event. The potential hazard was subsequently screened out qualitatively for all areas except the Spray Pond, based on the shallow water depths, short duration of flooding, and presence of large buildings, tanks and structures throughout the site, which limit available fetch lengths, and channel the wind direction to be parallel with the building walls.

Two locations on the Spray Pond were identified for determination of wind-wave and run-up impacts; the concrete embankment surrounding the Spray Pond, and the ESSW Pumphouse. The longest fetch that approaches each of these areas was determined as the longest straight line approaching the area of interest from inside the Spray Pond. The fetches defined for the concrete embankment and the ESSW Pumphouse are shown on ***Figure 3-4*** as Fetch No. 1 and Fetch No. 2, respectively.

Wind-waves associated with the two-year wind speed were considered to be simultaneous with an LIP event at the SSES site, following procedures defined in the USACE Coastal Engineering Manual (CEM) (USACE, 2008) for the calculation of wind speed, wind-wave generation, and wave run-up. The analysis was based on the two-year fastest-mile wind speed over land of 50 mph, recently computed for the nearby BBNPP site (UniStar, 2013; Table 2.4-33), which was converted to overwater wind speeds (using procedures outlined in the USACE CEM [USACE, 2008]) of 51.76 mph and 52.06 mph for Fetch No. 1 and Fetch No.2, respectively.

The maximum water level was calculated by combining the maximum LIP water level in the Spray Pond of 682.02 ft, with the calculated wave run-up and wind setup values for the two locations. The resulting wave run-up levels were determined as 683.3 ft on the Spray Pond embankment, and 684.3 ft on the ESSW Pumphouse.

The Spray Pond embankment is protected by a concrete lining up to elevation 685.5 ft (PPL, 2013; Section 2.4.8.4.1); therefore, the maximum wave run-up level does not result in overtopping of the embankment. The operating floor level of the ESSW Pumphouse is 685.5 ft

(PPL, 2013; Section 2.4.8.4.1); therefore, the maximum wave run-up level does not result in internal flooding of the pumphouse. There are no additional impacts associated with wind-waves coincident with LIP at any other safety-related SSCs on the SSES site.

3.3.1.4 Sedimentation and Debris Loading Coincident with Local Intense Precipitation

Sedimentation and debris loading on safety-related SSCs during an LIP event were screened out qualitatively as hazards at the SSES site. This screening was based on sources of sediments and debris in conjunction with flow depths, flow velocities, and flow directions predicted by the FLO-2D model for the Spray Pond and power block area. While it is understood that some localized sediment erosion, transport, and deposition could occur, as well as transport of debris, flow depths and velocities near safety-related structures were generally small, and these phenomena are not considered credible hazards to safety-related SSCs. Additionally, simulated flow directions were away from safety-related SSCs, which are surrounded by predominantly paved areas sloped away from buildings, precluding any impact on the SSCs from sedimentation or debris loading. While it is not expected that debris could impact safety-related structures, the potential for debris blockage of the VBS pedestrian access points was conservatively accounted for to maximize the impact of local flooding on the site (see ***Section 3.3.1.2***).

3.3.2 Probable Maximum Flood on Rivers and Streams

River flooding hazards at the SSES site were evaluated based on the HHA method presented in NRC NUREG/CR-7046 (NRC, 2011). The flood hazard reevaluation identified the Susquehanna River and Walker Run watercourses (***Figure 1-2***) for evaluation of flooding hazards due to river flooding. ***Sections 3.3.2.1 and 3.3.2.2*** present the screening of river flooding during the PMF for these two watercourses, respectively. Note that the unnamed tributary of the Susquehanna River that is shown on ***Figure 1-3*** is accounted for in the LIP analysis because it lies within the FLO-2D model domain.

3.3.2.1 Screening Out of Susquehanna River

The evaluation of flooding from the Susquehanna River was based on a conservative statistical analysis of United States Geological Survey (USGS) gage data at Danville, PA, located approximately 30 miles downstream of the SSES site (***Figure 1-2***). The PMF analysis completed for the proposed BBNPP located approximately 1 mile south west of the site (***Figure 1-2***) was also reviewed for comparison.

The statistical analysis was completed using HEC-SSP (USACE, 2010a) to fit the USGS gage data to a Log Pearson Type III distribution to develop a flood flow frequency curve (including confidence intervals). The 95 percent curve was then used to estimate flow rate in the Susquehanna River for the one-million year return period event. The analysis determined this flow rate to be less than 1,000,000 cfs.

To reduce the uncertainty in the statistical estimate, the BBNPP PMF analysis of the Susquehanna River was reviewed. The BBNPP FSAR determined the PMF flow rate to be 1,130,000 cfs (UniStar, 2013; Section 2.4.3). As this is more than 13 percent greater than the one-million year event flow estimated from statistical analysis, it is concluded that the BBNPP FSAR PMF analysis is conservative and applicable for screening out the effects of PMF on the Susquehanna River at the SSES site.

The 1,130,000 cfs PMF discharge calculated for the BBNPP site resulted in peak water levels on the Susquehanna River of 548.7 ft (UniStar, 2013, Section 2.4.3). This flood level is approximately 121 ft below the SSES site grade elevation of 670 ft. Consequently, PMF river flooding on the Susquehanna River is not considered a credible hazard to the SSES site.

3.3.2.2 Screening Out of Walker Run

The BBNPP FSAR analysis (UniStar, 2013, Section 2.4.3) indicates that the peak PMF elevation in Walker Run near the BBNPP site is approximately 675.7 ft. However, inspection of digital topographic data (PASDA, 2014) indicated that the lowest topographic divide between the Walker Run watershed and the SSES site is above 720 ft, at least 44.3 ft above the peak PMF level in Walker Run.

It was concluded that the intervening topographic features would prevent flooding on Walker Run from affecting the SSES site. Consequently, the SSES site would not be affected by the PMF and associated wind waves on Walker Run.

3.3.3 Dam Breaches and Failures

The potential flooding of the SSES site due to dam breaches and failures was evaluated using the Volume Method outlined in the NRC JLD-ISG-2013-01, "Guidance for Assessment of Flooding Hazards Due to Dam Failure" (NRC, 2013a). It is important to note the conservative nature of the methods outlined in JLD-ISG-2013-01, and that the analysis undertaken provides a conservative screening of the dam failure flood hazard.

The Volume Method consists of calculating the theoretical flood elevation that would be obtained if the storage volume of all upstream dams were to be placed on top of the 500-year flood along the main watercourse starting from a cross section located as close to the site as possible. This is a conservative assessment and represents a condition with all upstream dams breached simultaneously and the resulting floodwater transported to a point near the site without attenuation. The dam break hazard can be screened out if the results of the Volume Method show that the flood level does not reach the site grade (NRC, 2013a).

The locations and storage volumes for each dam upstream of the SSES site were obtained from the USACE National Inventory of Dams (NID) (USACE, 2013). The total storage volume for the 489 dams upstream of the SSES site is approximately 1,317,000 acre-ft of water. The 500-year flood level (used as a conservative baseline for the postulated dam failure scenario) adjacent to the SSES site was determined from a USACE study, which was originally completed for FEMA insurance studies (USACE, 2014).

Using digital topographic data (PASDA, 2014) in ArcGIS (ESRI, 2012), it was determined that the flood elevation associated with the total upstream storage volume superimposed on the 500-year flood level was approximately 611 ft. This level is approximately 59 ft below the SSES site grade elevation of 670 ft.

Since the calculated maximum water levels were 59 ft below the plant grade elevation, the potential impacts of wind-wave effects, debris and sediment loads, and hydrostatic and hydrodynamic loads associated with dam failure of all upstream dams were qualitatively dismissed.

Based on the 59 ft difference between dam failure flood levels and the site grade elevation, it was concluded that the SSES is not affected by flooding due to upstream dam failure, which is consequently screened out as a flood hazard. It should be noted that this conclusion is consistent with the BBNPP FSAR (UniStar, 2013, Section 2.4.4).

3.3.4 Coastal Flooding

The FHRR analysis of coastal flooding included consideration for the potential impacts of storm surge, seiche, and tsunami flooding.

As discussed in ***Section 1.3***, the SSES site lies on a relatively flat upland river terrace, approximately one mile west of the Susquehanna River, and the lowest plant grade elevation of the site is approximately 670 ft. The SSES site is located approximately 110 miles north of Chesapeake Bay, which would provide the most direct potential approach for storm surge (***Figure 1-1***). The distance to the Atlantic Ocean in the New York City area is also approximately 100 miles, but with higher intervening topography. The approximate length of the Susquehanna River channel between Chesapeake Bay and the SSES site is 165 miles. There are no other large bodies of water in the immediate vicinity of the site.

Coastal flooding (including storm surge, seiche, and tsunami) was not considered to be a credible hazard to the SSES site. As the site is located a sufficient distance inland from any coastal water bodies, and at a sufficiently high elevation, the risk of coastal flooding (or low water effects) was screened out qualitatively. The more detailed considerations described in NUREG/CR-7134 (NRC, 2012b) and JLD-ISG-2012-06 (NRC, 2013b) are not applicable for the SSES site. This is consistent with the conclusions of the BBNPP FSAR (UniStar, 2013; Sections 2.4.5 and 2.4.6).

3.3.5 Ice-Induced Flooding

The risk of ice-induced flooding, which could adversely impact safety-related structures at the SSES site, was assessed in accordance with the applicable guidelines of the NRC. According to guidance in NRC NUREG CR-7046 (NRC, 2011), ice-induced flooding was only considered in the context of whether a collapse of an ice jam could cause water to propagate to the site or whether an ice jam could cause flooding via backwater effects. The analysis screened out ice-induced flooding at the SSES site based on the conservative dam failure analysis outlined in ***Section 3.3.3*** (i.e., the flood level due to an ice jam would be less than or equal to the conservative multiple dam failure scenario evaluated). Ice accumulation on the Spray Pond would not cause flooding at the SSES site, because the most severe potential ice effects (i.e., blockage of the spillway) are less severe than the LIP analysis (***Section 3.3.1***), which effectively blocked the spillway by blocking the associated culverts (***Figure 2-2***) during a PMP event.

Ice-induced flooding was also evaluated in the FSAR for the nearby BBNPP site (UniStar, 2013; Section 2.4.7), with the similar conclusion that ice effects will not cause significant flooding in the area.

3.3.6 Flooding Resulting from Channel Migration or Diversion

The SSES site lies on a relatively flat upland river terrace, and the lowest plant grade elevation of the site is approximately 670 ft, approximately 153 ft above the Susquehanna River 100-year flood level and 121 ft above the PMF level (***Section 3.3.2***).

Based on recent evaluations for the nearby BBNPP site (UniStar, 2013; Section 2.4.9), it is understood that the existing geometry of the Susquehanna River valley near the SSES site does not support the possibility of major landslides. It is noted that several historic landslides have occurred in the vicinity of Schickshinny Mountain (***Figure 1-2***), located approximately 10 miles from the site, and potential future landslides in the area of Schickshinny Mountain could cause localized changes to the river course. However, Lee Mountain (located between SSES and Schickshinny Mountain) only allows for flow through the existing channel, indicating that any new landslides near Schickshinny Mountain would not affect the course of the river near the SSES site.

Additionally, the intervening topographic features between Walker Run and the SSES site preclude the possibility of a flooding hazard due to migration of Walker Run.

Given the topographic and geologic evidence in the region, the possibility of channel migration causing a flood hazard to safety-related SSCs at the SSES site is considered to be negligible. Flooding resulting from channel migration is, therefore, screened out.

3.3.7 Combined Effects Flooding

Both American Nuclear Society (ANS), ANSI/ANS-2.8-1992 (ANS, 1992), and NRC NUREG/CR-7046 (NRC, 2011) indicate that isolated flood-causing events are not adequate as a design basis for power reactors. Consequently, it is appropriate to postulate critical combinations of flood-causing events when reevaluating the flooding hazard at the SSES site.

The NRC NUREG/CR-7046 (NRC, 2011) provides guidance on the combined effects scenarios that require consideration and appropriate evaluation. For the SSES site, many of these scenarios can be qualitatively screened out.

The combined effects associated with coastal flooding, including storm surge, seiche, and tsunami were screened out as the site is located approximately 100 miles from the nearest coastal water body (***Section 3.3.4***). The proposed dam failure combined effect scenarios presented in NRC (2011) were also screened out as less conservative than the reevaluated dam failure analysis described in ***Section 3.3.3***.

The remaining combined effects flood scenarios that required investigation and analysis were Alternative I, II, and III proposed for precipitation floods in ANS (1992), which are listed below:

Alternative I consists of the following effects:

- Mean monthly base flow
- Median soil moisture
- Antecedent or subsequent rain: the lesser of (1) rainfall equal to 40 percent of the PMP and (2) a 500-year rainfall
- The PMP
- Waves induced by a 2-year wind speed applied along the critical direction

Alternative II consists of the following effects:

- Mean monthly base flow
- Probable maximum snowpack
- A 100-year, snow-season rainfall
- Waves induced by a 2-year wind speed applied along the critical direction

Alternative III consists of the following effects:

- Mean monthly base flow
- A 100-year snowpack
- Snow-season PMP
- Waves induced by a 2-year wind speed applied along the critical direction

Given the SSES site plant grade elevation is approximately 121 ft above the PMF level, and in accordance with the HHA approach, a single conservative scenario was postulated to screen out the three alternatives outlined above. The postulated scenario required many conservative assumptions to ensure that all three Alternatives were screened. As a result this bounding scenario was much more conservative than would be required under any NRC guidance. The bounding conservative scenario is outlined and justified below.

The depth of water in the Susquehanna River during the PMF event (i.e., based on [UniStar, 2013; Section 2.4.1.1.3]) was doubled, representing a condition more severe than the alternatives listed above. This condition corresponds to a flood level approximately 54 ft below the plant grade elevation. The same approach was utilized for Walker Run, which resulted in a flood level approximately 28 ft below the topographic divide between the watershed and the SSES site.

The justification for why this approach bounds each of the following alternatives is as follows:

- Doubling the PMF depth bounds Alternative I based on the following:
  - Doubling the PMF flood depth is comparable to doubling the PMP rainfall depth;
  - Doubling the PMP rainfall depth would be a more severe condition than simply incorporating an antecedent storm (e.g., 40 percent of the PMP or the 500-year rainfall). Consequently, doubling the PMF flood depth is more conservative than the antecedent storm postulated in Alternative I. The significant margins even with this conservative scenario indicated that an analysis of wind-waves was not required.
- Doubling the PMF depth bounds Alternative II based on the following:
  - Alternative II is a less severe condition than Alternative III because snow effects are considered in terms of melt water (only a portion of the existing snow pack). Neither the 100-year snow pack nor the probable maximum snow pack would melt entirely during a subsequent rainfall event. Therefore, the more severe rainfall event (Alternative III) provides a more severe flooding scenario than Alternative II. Consequently, if Alternative III is screened (below), Alternative II would be effectively screened out as well.

- Doubling the PMF depth bounds Alternative III based on the following:
  - The 100-year snow pack was computed using the statistical capabilities of HEC-SSP (USACE, 2010a). The USACE "Runoff from Snowmelt" Engineer Manual (USACE, 1998) was then used to estimate the corresponding depths of snow melt that would contribute runoff to the watershed. The equivalent rainfall depth (including melt water) was calculated to be less than double the PMP depth, which was already screened for Alternative I. Consequently, Alternatives II and III were screened out.

On the basis of the reasoning outlined above, Combined Effects flooding was screened out as a potential hazard for the SSES site.

3.3.8 Cooling Tower Basin Rupture

Sections 3.3.8.1 through 3.3.8.3 address the effects of Cooling Tower basin rupture at the SSES site, as required by NRC JLD-ISG-2013-01, "Guidance for Assessment of Flooding Hazards Due to Dam Failure" (NRC, 2013a).

3.3.8.1 Cooling Tower Basin Failure Modes

There are two Cooling Towers on the SSES site, each containing a storage basin beneath the tower with a capacity of approximately 6 million gallons of water (PPL, 2006; Section 2.4.9.6). Although the Cooling Tower basins are situated below the elevation of the surrounding compacted fill (i.e., such that the water level is near ground level), both the Unit 1 and Unit 2 basins are located at higher elevations than safety-related structures at the SSES site. If it is conservatively assumed that the compacted fill material is removed, then a failure of one or both of the basins could cause flooding in critical areas of the site.

The most credible flood-causing failure modes for the Cooling Tower basins were determined to be:

- Mode #1: Collapse of one or more of the 24 ft long concrete panels around the perimeter of one or both of the basins.
- Mode #2: Collapse of the headwall of the Cold Water Outlet Chamber (CWOC) of one or both of the basins. The CWOCs are concrete structures, approximately 70 ft long and between 35 and 60 ft wide (tapering along the length of the CWOC), which protrude from the basins to provide an outlet for cooled water to supply the cooling water circulation pumps.

Construction details and dimensions of the structures were obtained from design drawings, and outflow hydrographs were developed to simulate outflows for each of the above failure modes, using spreadsheet calculations and HEC-RAS software (USACE, 2010b). The FLO-2D model developed for the LIP analysis at the site (***Section 3.3.1***) was used as the basis for the Cooling Tower basin rupture analysis. The model topography was adjusted to represent soil erosion in the areas of failure, and the rainfall inputs were removed.

Sensitivity analyses were completed for the basin perimeter panel collapse failure mode (Mode #1) to assess the impact of varying the location and number of panel failures. An initial assessment based on dominant topographic gradients indicated that failure of the Unit 1 Cooling Tower basin (***Figure 2-2***) in a northerly direction would provide the most severe flooding condition at the ESSW Pumphouse. Other scenarios considered simultaneous failure of the east side of both Cooling Tower basins, which provides the most direct flow toward the Turbine Building and other critical areas of the power block.

Fifteen perimeter panel failure simulations were undertaken, which considered combinations of simultaneous collapse of up to fifteen panels, within one or both basins failing at varying locations. Consideration for the failure of more than fifteen panels was evaluated qualitatively based on dominant topographic gradients and determined not to be the critical scenario for flooding at any one location (i.e., the water would be dispersed over a larger area).

The CWOC headwall collapse failure mode (Mode #2) was evaluated by postulating simultaneous failure of both the Unit 1 Cooling Tower basin and the Unit 2 Cooling Tower basin headwalls. This scenario is conservative, providing more severe flooding than the isolated failure of a single basin.

The results from the Mode #2 simulation and the sensitivity analyses for Mode #1 (i.e., a total of 16 simulations) were used to develop four critical scenarios, which are listed below.

- Scenario 1 (Mode #1): Critical scenario with respect to flooding at the ESSW Pumphouse; simultaneous failure of fifteen perimeter panels along the north side of the Unit 1 basin and erosion of the access road north of the Unit 1 Cooling Tower.
- Scenario 2 (Mode #1): Critical perimeter panel failure scenario with respect to volume of inflow to the Turbine Building; simultaneous failure of four

perimeter panels along the southeast side of the Unit 1 basin and the northeast side of the Unit 2 basin, to direct flows towards the Turbine Building.

- Scenario 3 (Mode #1): A scenario developed to maximize flooding (if any) on the east side of the power block; simultaneous failure of four perimeter panels along the northeast side of the Unit 1 basin and the southeast side of the Unit 2 basin, to direct flows around the Turbine Building towards the east side of the power block.
- Scenario 4 (Mode #2): Critical scenario for CWOC headwall collapse; instantaneous collapse of both headwalls, which provides flow directly towards the Turbine Building.

3.3.8.2 Effects of Cooling Tower Basin Rupture

The results from the critical Cooling Tower basin rupture scenarios (Scenarios 1 through 4, ***Section 3.3.8.1***) demonstrated that the only safety-related structure that would subject to inundation by floodwater for any scenario is the ESSW Pumphouse. Water from the ruptured basins did not reach any other safety-related structures on the SSES site (***Table 3-2***).

The maximum flood elevation on the south side of the ESSW Pumphouse (toward the Unit 1 Cooling Tower) was 686.42 ft for Scenario 1, which corresponds to a maximum localized external flood depth of 1.45 ft. It should be noted that the west, south, and east sides of the ESSW Pumphouse are credited with flood protection barriers to an elevation of 9.3 ft above the finished floor level of 685.5 ft (i.e., a flood barrier elevation of 694.8 ft) (***Table 2-1***), providing significant freeboard above the flood level of 686.42 ft. The maximum flood depths throughout the site for Scenario 1 are shown on ***Figure 3-5***.

The Turbine Building is not a safety-related structure; however, if inflow to the Turbine Building caused internal flood levels to exceed 678 ft, there are pathways through which water could enter the Reactor Building through the Turbine Building. As noted in ***Table 3-2***, approximately 9.9 million gallons of storage is available within the Turbine Building below elevation 678 ft. The maximum inflow to the Turbine Building during the Cooling Tower rupture simulations (3.1 million gallons) occurred for Scenario 4, which can be contained within the building and would not impact the Unit 1 and Unit 2 Reactor Buildings or other safety-related SSCs.

The resulting flood depths throughout the SSES site for Scenario 4 are shown on ***Figure 3-6***, which demonstrates that approximately one to two inches of flood water passed near the Common Diesel Generator 'E' and the Unit 1 Reactor Buildings; however, the flood water did not actually reach the Common Diesel Generator 'E' Building, the Unit 1 Reactor Building, or any other safety-related structures.

The FLO-2D model results were used to evaluate the hydrostatic and hydrodynamic forces on the ESSW Pumphouse and the Turbine Building (the only buildings affected by flood water that could potentially impact safety-related SSCs). The maximum resultant hydrostatic forces exerted on the south side of the pumphouse and west side of the Turbine Building were approximately 66 and 964 pounds/ft, respectively. The maximum hydrodynamic forces were 84 and 337 pounds/ft, respectively.

3.3.8.3 Sedimentation and Debris Loading Coincident with Cooling Tower Basin Rupture

The impacts of sedimentation and debris during Cooling Tower basin ruptures were screened out as hazards to the SSES site based on an assessment of flow velocities, flow directions, and consideration of the available storage volume within the Turbine Building.

Based on 2D modeling results, water approaching the ESSW Pumphouse would pass through a low velocity flow region before reaching the pumphouse. Any sediment and debris carried by the flow would be deposited in this area. Sediment and debris were screened out as hazards for the Turbine Building because there would be more than 6 million gallons of available storage remaining within the building after the volume of inflow report in ***Section 3.3.8.2***. This remaining storage volume is considered to be sufficient to contain even a conservative estimate of the volume of sediment or debris that could enter the Turbine Building.

4.0 COMPARISON OF CURRENT AND REEVALUATED FLOOD LEVELS

Section 4.0 has been prepared in response to Request for Information Item 1.c. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (NRC, 2012a). Item 1.c. requires a comparison of current and reevaluated flood-causing mechanisms at the site, a comparison of the CLB flood elevation to the reevaluated flood elevation for each flood-causing mechanism, and how the findings from Enclosure 4 of this letter (i.e., Recommendation 2.3 flooding walkdowns) support this determination. If the CLB flood bounds the reevaluated hazard for all flood-causing mechanisms, justification should be included for how this finding was determined.

4.1 COMPARISON OF CURRENT AND REEVALUATED FLOOD-CAUSING MECHANISMS

The flood-causing mechanisms evaluated under the CLB were the following:

- Cooling Tower basins rupture (PPL, 2013; Section 2.4.2.2)
- Local Intense Precipitation (PPL, 2013; Section 2.4.2.3)
- Probable Maximum Flood on rivers and streams (PPL, 2013; Section 2.4.3) including coincident wind-wave activity (PPL, 2013, Section 2.4.3.6 and Section 2.4.8.4.1)
- Potential dam failures (seismically induced) (PPL, 2013, Section 2.4.4)
- Probable Maximum Surge and Seiche flooding (PPL, 2013, Section 2.4.5)
- Probable Maximum Tsunami flooding (PPL, 2013, Section 2.4.6)
- Ice effects (PPL, 2013, Section 2.4.7)
- Channel diversions (PPL, 2013, Section 2.4.9)

The flood hazard reevaluation includes the same flooding mechanisms as presented in the CLB. Additionally, Combined Effects flooding has been considered as part of the reevaluated flood hazard.

The conditions for which the flooding analyses were performed and the methods used to perform the analyses varied between CLB analyses and the reevaluation analyses. These differences are summarized in ***Tables 4-1 and 4-2***.

4.2 Assessment of Differences Between Current Licensing Basis and Reevaluated Flood Elevations and Effects

A comparison of CLB and reevaluated flood levels and effects at the SSES site for each flood mechanism is provided in the following subsections of this report. A summary comparison is presented in ***Table 4-3***.

4.2.1 Local Intense Precipitation Flooding

The CLB states that the possibility of flooding of any safety-related facility on the SSES site due to PMP is precluded; however, no specific flood levels or depths are reported for locations adjacent to safety-related SSCs. Pressure resistant doors are credited as passive flood protection features, which protect the site during a PMP event.

The reevaluated LIP analysis determined that the maximum localized ponding depths adjacent to safety-related SSCs vary between 1.61 ft at the Unit 1 and Unit 2 Reactor Buildings, and 0.31 ft at the south side of the ESSW Pumphouse. These maximum depths do not result in internal flooding of any safety-related SSCs due to the presence of credited flood barriers with a minimum height of 2 ft.

The CLB and the reevaluation, therefore, both conclude that internal flooding of safety-related SSCs is precluded during the LIP event. It should be noted that the CLB and FHRR consider different rainfall events within their LIP analyses, which were each in line with the current guidance at the time of the respective analyses.

The impacts of sedimentation and debris loading during a LIP event were screened out qualitatively through consideration of sediment and debris sources, in conjunction with flow depths, velocities, and directions predicted by the FLO-2D model for the Spray Pond and power block area. Hydrostatic and hydrodynamic loadings at safety-related SSCs were determined not to require further structural analysis of buildings, due to the shallow depths and low flow velocities.

The maximum still water level on the Spray Pond during the LIP event was determined and subsequently utilized to assess the impact of the 2-year coincident wind-wave activity on the Spray Pond, in conjunction with a LIP event. The maximum run-up elevations were calculated as 684.3 ft and 683.3 ft for the ESSW Pumphouse and the Spray Pond embankment, respectively. The maximum run-up levels computed for these locations in the FSAR are 684.8 ft and 684.6 ft for the ESSW Pumphouse and Spray Pond embankment, respectively.

It should be noted that the run-up results for the hazard reevaluation are not directly comparable to the critical run-up scenario in the CLB due to the use of different input parameters (i.e., the CLB considers multiple run-up scenarios; the critical scenario is a 65 mph wind speed with the SPF). Regardless of differences in parameters, the reevaluated run-up elevations on the Spray Pond and ESSW Pumphouse are bounded by the critical run-up elevations reported in the CLB. It was therefore concluded that associated hydrostatic and hydrodynamic loading impacts on the ESSW pumphouse and intake structure are also bounded by the design basis for the pumphouse.

4.2.2 Flooding in Rivers and Streams

The CLB indicates that the SSES site is a dry site, and the PMF water level on the Susquehanna River adjacent to the site is 548.0 ft. This level includes an additional 2.3 ft for wave setup and run-up effects. The CLB PMF level is approximately 122 ft below the SSES site grade elevation of 670 ft. The CLB does not consider flooding from Walker Run independently, which is a tributary of the Susquehanna River.

The reevaluated PMF water level for the Susquehanna River is 548.7 ft. A small additional level could be added to this to account for wind-wave effects (similar to the FSAR analysis). However, due to the significant freeboard (approximately 120 ft), coincident wind-wave activity is qualitatively dismissed. The reevaluated PMF level is comparable to the CLB PMF level, considering the large existing freeboard.

The reevaluated PMF water level for Walker Run is approximately 673 ft, which is approximately 47 ft below the topographic divide that lies between the Walker Run watershed and the SSES site.

The conclusions regarding the reevaluated PMF flood levels agree with the conclusions of the CLB, i.e., river flooding is not considered to be a credible hazard at the SSES site.

4.2.3 Dam Breaches and Failures

The CLB considers a postulated scenario of simultaneous and instantaneous failure of fourteen dams located upstream of the SSES site. It should be noted that six of these fourteen dams have since been deauthorized for construction; however, the original analysis has been retained as conservative. The precise flood level for this CLB scenario is not stated; however, the resulting water level is bounded by the CLB PMF level. The dam failure water level is more than 120 ft below site grade elevation, i.e., less than 550 ft.

The reevaluation of dam failures was accomplished using the Volume Method, which is a conservative screening analysis that assumes the volume of water stored behind all upstream dams is instantaneously translated to a point near the site, during a 500-year flood event in the Susquehanna River. The Volume Method results indicated a freeboard of greater than 50 ft between the screening flood level and the plant grade. Thus, a detailed dam failure analysis is not required. This conclusion agrees with the conclusion of the CLB, that dam failure does not constitute a hazard for the SSES site.

4.2.4 Coastal Flooding, Ice-Induced Flooding, and Channel Diversion

Storm surge and seiche, tsunami, ice-induced flooding, and channel diversion were screened out as potential flooding events in both the FSAR (PPL, 2013, Sections 2.4.5, 2.4.6, and 2.4.7) and the hazard reevaluation.

4.2.5 Combined Effects Flooding

Combinations of flooding effects are considered in the CLB for the wind-wave analyses coincident with other flooding effects (e.g., LIP, river flooding, etc.) as discussed above. However, combinations of flooding effects, such as snow pack with rainfall, were not considered. Consistent with ANSI/ANS-2.8-1992 (ANS, 1992) the hazard reevaluation postulated several combined effects flooding scenarios. The postulated scenarios were subsequently screened out using a conservative bounding analysis (i.e., doubling the flow depths associated with the PMF analysis, ***Section 3.3.7***). This analysis showed that (for critical combinations of flooding events) there remains in excess of 54 ft of freeboard for flooding from the Susquehanna River and 28 ft of freeboard for flooding from Walker Run. Consequently, combined effects do not constitute a flooding hazard for the SSES site.

4.2.6 Cooling Tower Basins Rupture

The CLB considers the potential impact of postulated Cooling Tower basin ruptures on the SSES site. A number of scenarios were considered which result in flooding of the western side of the Turbine Building; however, no internal flooding of safety-related SSCs was indicated by this analysis. Passive flood protection measures, such as pressure resistant doors, are credited with preventing internal flooding of any safety-related SSCs (PPL, 2013; Section 2.4.2.2). Specific flood levels and depths are not reported in the FSAR (PPL, 2013), although flood levels are reported in the supporting analysis for the FSAR.

A reevaluated analysis of numerous Cooling Tower basin rupture scenarios has been undertaken to determine the associated hazards (if any) to safety-related SSCs on the SSES site. A sensitivity analysis was completed to determine the worst case rupture scenario, in terms of size and orientation of the basin rupture(s) and effects throughout the plant. The maximum flood level at the ESSW Pumphouse was determined to be 686.42 ft, approximately 8.4 ft below the height of the credited flood barrier. The volume of inflow to the Turbine Building would be contained as internal storage within the Turbine Building and not result in flows being conveyed to adjacent safety-related SSCs (due to the credited flood barriers). The reevaluated analysis determined that no internal flooding of safety-related SSCs would result from Cooling Tower basin ruptures on the site, due to the passive flood protection of the credited flood barriers.

The CLB and the flood hazard reevaluation therefore both conclude that internal flooding of safety-related SSCs is precluded in the event of Cooling Tower basin ruptures. It should be noted that the differences in the assessment methodologies between the CLB Cooling Tower basin rupture analysis and the flood hazard reevaluation analysis preclude a direct comparison. For example, the CLB analysis utilizes a qualitative assessment of all potential cooling tower basins potential flow routes (e.g., including conservative flow paths through buildings), with manual calculations for one-dimensional flow (e.g., Manning's equation). The flood hazard reevaluation analysis presented herein applies a more refined two-dimensional methodology which allows water to flow along natural drainage paths without predefining flow-paths (as required for a one-dimensional analysis) and accounts for the volume of storage in the Turbine Building.

The impacts of sedimentation and debris loading during Cooling Tower basin ruptures were screened out as hazards to safety-related SSCs based on an assessment of flood depths, velocities, flow directions, and the potential volume of sediment and debris storage available within the Turbine Building.

4.3 SUPPORTING DOCUMENTATION

The reevaluated flood levels presented in this report are based on detailed calculations developed in support of the flood hazard reevaluation at the SSES site. The Walkdown Report (PPL, 2012), provides additional information regarding the CLB flood hazard levels, as well as flooding protection and mitigation features.

SSES credits "Exterior Passive" features for flood protection, such as normally closed external pressure resistant doors, exterior wall penetrations, and openings within exterior walls of safety-related structures. These flood protection features have been visually inspected in accordance with NRC guidance, as detailed within the Walkdown Report (PPL, 2012). Degraded conditions discovered during the walkdowns were entered into the SSES corrective action program.

4.3.1 Technical Justification of the Flood Hazard Reevaluation Analysis Approaches

All flood hazard reevaluation analyses described in this report were undertaken considering and implementing the techniques, software, and methods used in present-day standard engineering practice. The technical basis for the various scenarios modeled under the HHA method and the key assumptions utilized in the determination of the reevaluated flooding levels for each flood-causing mechanism are discussed individually in ***Section 3.0*** and are summarized in ***Tables 4-1 through 4-3***.

4.3.2 Technical Justification based on the Recommendation 2.3 Walkdown Results

With respect to the implementation and conclusions of the flooding hazard reevaluation, results from the Walkdown Report (PPL, 2012) have been taken into consideration. Specifically, it was found the flood protection barriers discussed in the Walkdown Report provide protection for the safety-related SSCs with respect to the reevaluated flooding hazards at the SSES site.

4.4 CONCLUSIONS

The CLB states that the SSES site is considered to be a "dry site" because the plant grade elevation is approximately 120 ft above the maximum flood levels on the Susquehanna River. In addition to the Susquehanna River, the reevaluation analysis considers the maximum flood levels in the Walker Run watershed, and concurs that the site is a "dry site," which is elevated significantly above the maximum flood levels on either watercourse.

Both the CLB and the reevaluation analysis determine that during a LIP event or a Cooling Tower basin rupture, there would be no internal flooding of safety-related SSCs due to the presence of passive flood barriers. The reevaluated peak flood levels remain below credited flood barrier levels, therefore no internal building flooding would occur on the SSES site due to the postulated scenarios.

The reevaluated assessment of dam failure flooding (using the Volume Method) screened out dam failure as a potential hazard to the SSES site, which is consistent with the CLB determination.

Both the reevaluation analysis and the CLB dismiss flooding as a result of storm surge and seiche, tsunami, ice effects, and channel migration or diversion.

The conclusions of the reevaluation are consistent with the CLB, i.e., that the SSES site is a "dry site," and no internal flooding of any safety-related SSCs would result from the flood-causing mechanisms assessed.

5.0 INTERIM EVALUATION AND ACTIONS

Section 5.0 has been prepared in response to Request for Information Item 1.d. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (NRC, 2012a): "Provide an interim evaluation and actions taken or planned to address any higher flooding hazards relative to the design basis, prior to completion of the integrated assessment."

5.1 EVALUATION OF THE IMPACT OF THE REEVALUATED FLOOD LEVELS AND EFFECTS ON STRUCTURES, SYSTEMS, AND COMPONENTS

The flooding levels of all potential flood-causing mechanisms, as presented in ***Section 3.0***, did not exceed the elevations of the flood barriers credited with passive protection of safety-related buildings and structures at the site. The most refined simulated conditions resulted in localized ponding of water for limited durations adjacent to safety-related structures; however, no internal flooding would occur under these conditions due to the presence of flood barriers.

5.2 ACTIONS TAKEN TO ADDRESS FLOOD HAZARDS NOT COMPLETELY BOUNDED BY THE CURRENT LICENSING BASIS HAZARD

Request for Information Item 1.d. of NRC Recommendation 2.1 specifies that the flooding reevaluation contain an interim evaluation and actions taken or planned to address any higher flooding hazards relative to the CLB, prior to completion of the integrated assessment, if necessary (NRC, 2012a).

Based on the results of the reevaluated flood hazards, no interim actions are necessary and completion of an integrated assessment is not required.

6.0 ADDITIONAL ACTIONS

Section 6.0 has been prepared in response to Request for Information Item 1.e. of NRC Recommendation 2.1, Enclosure 2 of the 10 CFR 50.54(f) letter (NRC, 2012a): "Provide additional actions beyond Request for Information Item 1.d taken or planned to address flooding hazards, if any."

At this time, there are no additional actions required. As presented in ***Section 5.0***, which addressed the Request for Information Item 1.d. of NRC Recommendation 2.1 (NRC, 2012a), no actions have been taken or are planned to address flooding hazards at the SSES site, and completion of an integrated assessment is not required.

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32. USCB, 2014a, United States Census Bureau (USCB) "State & County QuickFacts: Wilkes-Barre, "Wilkes-Barre (city) Pennsylvania," Website: <http://quickfacts.census.gov/qfd/states/42/4285152.html>. Date Published: July 8, 2014, Date Accessed: October 31, 2014.

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TABLES

TABLE 2-1
LIST OF STRUCTURES AND DESIGN ELEVATIONS

STRUCTURE	SAFETY-RELATED?	DESIGN FLOOR ELEVATION (ft NGVD29)	FLOOD BARRIER HEIGHT (ft)	FLOOD BARRIER ELEVATION (ft NGVD29)
ESSW Pumphouse (south side)	Yes	685.5	9.3	694.80
Common Diesel Generator Building	Yes	677.0	2.0	679.0
Unit 1 Reactor Building	Yes	670.0	2.0	672.0
Unit 2 Reactor Building	Yes	670.0	2.0	672.0
Common Diesel Generator ‘E’ Building	Yes	675.5	2.5	678.0
Turbine Building	No	676.0	N/A	N/A

Note:

Elevations and heights were extracted from SSES flood barrier drawings.

TABLE 2-2
SUMMARY OF CURRENT LICENSING BASIS FLOODING ANALYSIS INPUT PARAMETERS

PARAMETER	INPUT VALUE	SOURCE (PPL, 2013)
Local Intense Precipitation due to Probable Maximum Precipitation	29.72 inches (all season 24-hour PMP depth) 6.70 inches (30-minute PMP depth)	Table 2.4-5 Table 2.4-6
Dam failures (seismically induced)	Simultaneous failure of fourteen upstream dams (8 existing, 6 subsequently deauthorized for construction)	Section 2.4.4
Wind Speed for Coincident Wave Run-up on Susquehanna River	45 miles per hour	Section 2.4.3.6
Wind Speed(s) for Coincident Wave Run-up on Spray Pond	40, 65 and 80 miles per hour	Table 2.4-14
Cooling Tower Basin(s) Rupture	Multiple rupture locations and directions are considered	Section 2.4.2.2

Reference:

PPL, 2013, "Final Safety Analysis Report, Susquehanna Steam Electric Station Units 1 and 2," Revision 66, February 2013.

TABLE 2-3
CURRENT LICENSING BASIS FLOOD ELEVATIONS

FLOODING MECHANISM	FLOOD LEVEL (FT NGVD29)	SOURCE (PPL, 2013)
Local Intense Precipitation due to Probable Maximum Precipitation	N/A[1]	Section 2.4.2.3
Probable Maximum Flood on the Susquehanna River	548.0 (Berwick Bridge intact and coincident wind-wave activity)	Section 2.4.3
Probable Maximum Flood on the Spray Pond	682.3	Table 2.4-13
Spray Pond Critical Run-up Scenario: 65 mph wind-wave activity and Standard Project Flood (i.e., 50% PMF)	684.8 (ESSW Pumphouse) 684.6 (Spray Pond)	Table 2.4-14
Dam Failure (seismically induced)	N/A[2]	Section 2.4.4.3
Storm Surge and Seiche Flooding	N/A[2]	Section 2.4.5
Tsunami Flooding	N/A[2]	Section 2.4.6
Ice Flooding	N/A[2]	Section 2.4.7
Channel Migration/Diversion Flooding	N/A[2]	Section 2.4.9
Cooling Tower Basin(s) Rupture	N/A[1]	Section 2.4.2.3

Notes:

1 Not applicable because the SSES FSAR does not provide specific flood elevations and depths; however, it is stated that flooding of all safety-related SSCs is precluded.

2 Not applicable because this flood-causing mechanism was screened out.

Reference:

PPL, 2013, "Final Safety Analysis Report, Susquehanna Steam Electric Station Units 1 and 2," Revision 66, February 2013.

TABLE 3-1
LOCAL INTENSE PRECIPITATION PEAK FLOOD LEVELS AND EXTERNAL PONDING DEPTHS

STRUCTURE	SAFETY-RELATED?	PEAK FLOOD LEVEL[1] (ft NGVD29)	FREEBOARD BETWEEN PEAK FLOOD LEVEL AND FLOOD BARRIER ELEVATION (ft)
ESSW Pumphouse (south side)	Yes	685.74	9.06
ESSW Pumphouse Valve Chamber[2]	Yes	696.64	0.70
Common Diesel Generator Building	Yes	676.30	2.70
Unit 1 Reactor Building (east side)	Yes	671.36	0.64
Unit 2 Reactor Building (east side)[3]	Yes	670.91	1.09
Common Diesel Generator 'E' Building	Yes	675.27	2.73
Turbine Building[4]	No	677.82	Not Applicable
Spray Pond[5]	Yes	682.02	Not Applicable

Notes:

1 The peak flood level is reported for the most refined (Case 5) local intense precipitation simulation described in ***Section 3.3.1***.

2 The ESSW Pumphouse valve chamber does not include a flood barrier; however, the chamber is raised above the surrounding ground level.

3 There is a small area on the south side of the Unit 2 Reactor Building that is enclosed by a retaining wall where higher flood levels are higher. However, these higher levels are still below the height of the flood protection barriers in that location (681 ft).

4 The Turbine Building can store approximately 9.9 million gallons of water below elevation 678 ft. Above this elevation flows could potentially be conveyed to safety-related SSCs. The inflow to the Turbine Building during Case 5 (the most refined case) is determined to be 2.5 million gallons, which is contained within the building. The Turbine Building does not include flood barriers; therefore, a freeboard value is not applicable.

5 External ponding depth and flood barrier freeboard values are not applicable for the Spray Pond.

TABLE 3-2
COOLING TOWER BASIN RUPTURE PEAK FLOOD LEVELS AND EXTERNAL PONDING DEPTHS

STRUCTURE	SAFETY-RELATED?	PEAK FLOOD LEVEL (ft NGVD29)	FREEBOARD BETWEEN PEAK FLOOD LEVEL AND FLOOD BARRIER LEVEL (ft)
ESSW Pumphouse (south side)[1]	Yes	686.42	8.41
ESSW Pumphouse Valve Chamber	Yes	N/A[3]	N/A[3]
Common Diesel Generator Building	Yes	N/A[3]	N/A[3]
Unit 1 Reactor Building (east side)	Yes	N/A[3]	N/A[3]
Unit 2 Reactor Building (east side)	Yes	N/A[3]	N/A[3]
Common Diesel Generator 'E' Building	Yes	N/A[3]	N/A[3]
Turbine Building[2]	No	680.41	N/A[4]

Notes:

1 Results are presented for Scenario 1, the critical Cooling Tower rupture scenario for the ESSW Pumphouse.

2 Results are presented for Scenario 4, the critical Cooling Tower rupture scenario for inflow to the Turbine Building. The building can store approximately 9.9 million gallons of water before flows could be conveyed to safety-related SSCs. The maximum inflow to the Turbine Building is determined to be 3.1 million gallons, which is contained within the building.

3 Not applicable because flood water does not extend up to these buildings during any of the critical Cooling Tower rupture scenarios considered.

4 The Turbine Building does not include flood barriers; therefore, a freeboard value is not applicable.

TABLE 4-1
COMPARISON OF FLOOD HAZARD REEVALUATION WITH CURRENT LICENSING BASIS: MODELING APPROACHES

MODELING APPROACH	FLOOD HAZARD REEVALUATION	CURRENT LICENSING BASIS (PPL, 2013)
Wind Analysis Data Source	Analysis of SSES meteorological data[1]	Analysis of Avoca Airport and SSES meteorological data
Local Intense Precipitation	HMR-52 (NOAA, 1982)[1]	HMR-33
Local Intense Precipitation Flooding Characterization	A 2D flood routing model (FLO-2D) is used to simulate runoff from the power block and surrounding areas	Rational method calculation to confirm flow capacities at five key locations on the site
Probable Maximum Flood Flow for Susquehanna River	Statistical analysis (HEC-SSP) of USGS gage flow data and BBNPP PMF analysis	NRC Regulatory Guide 1.59 (NRC, 1977)[1]
Probable Maximum Flood Elevation for Susquehanna River	Screening analysis completed based on comparison of PMF flow rates	Hydraulic backwater curves[1]
Probable Maximum Flood Flow for Walker Run	USACE HEC-HMS[1]	No flooding analyses for Walker Run are documented in the current licensing basis
Probable Maximum Flood Elevation for Walker Run	USACE HEC-RAS[1]	No flooding analyses for Walker Run are documented in the current licensing basis
Dam Failure Flooding	The Volume Method was used to determine the water surface elevation	Consideration of instantaneous failure of fourteen dams (existing and proposed) upstream of the site
Combined Effects Flooding	Application of NUREG/CR-7046 and ANSI/ANS-2.8-1992; accounting for recommended combinations of flooding effects by developing a conservative bounding scenario	No combined effects flooding, except for coincident wind-wave activity for PMF/SPF on Susquehanna River/Spray Pond respectively, in accordance with NRC Regulatory Guide 1.59 (NRC, 1977)
Cooling Tower Basin Rupture	Application of NUREG/CR-7046 and JLD-ISG-13-01 methods, using a 2D flood routing model (FLO-2D)	Qualitative assessment of all potential cooling tower basin potential flow routes based on conservative assessments of maximum flood depths with manual calculation of flow routes, flood depths and volumes (e.g., Manning's equation)

Note:

1 Sourced from UniStar, 2013, "Final Safety Analysis Report, Bell Bend Nuclear Power Plant," Revision 4, 2013.

Reference:

PPL, 2013, "Final Safety Analysis Report, Susquehanna Steam Electric Station Units 1 and 2," Revision 66, February 2013.

TABLE 4-2
COMPARISON OF FLOOD HAZARD REEVALUATION WITH CURRENT LICENSING BASIS: ANALYTICAL INPUTS

ANALYTICAL INPUTS	FLOOD HAZARD REEVALUATION	CURRENT LICENSING BASIS (PPL, 2013)
Local Intense Precipitation	17.5 inches (1-hour, 1-mi^2) 13.3 inches (30-min, 1-mi^2) (UniStar, 2013, Table 2.4-18)	29.72 inches (all season 24-hour PMP; Table 2.4-5) 6.70 inches (30-minute PMP; Table 2.4-6)
Spray Pond wind speed for wind-wave run-up	50 mph over land (UniStar, 2013; Table 2.4-33)	40, 65, and 85 mph FSAR (PPL, 2013; Section 2.4.8.4.1)

References:

PPL, 2013, "Final Safety Analysis Report, Susquehanna Steam Electric Station Units 1 and 2," Revision 66, February 2013.

UniStar, 2013, "Bell Bend Nuclear Power Plant, Combined License Application, Part 2: Final Safety Analysis Report," Revision 4, 2013.

TABLE 4-3
COMPARISON OF FLOOD HAZARD REEVALUATION WITH CURRENT LICENSING BASIS: PEAK FLOOD LEVELS

ITEM NO.	FLOOD-CAUSING MECHANISM	FLOOD HAZARD REEVALUATION PEAK FLOOD LEVEL/DEPTH	CURRENT LICENSING BASIS[1] PEAK FLOOD LEVEL/DEPTH
1	Local Intense Precipitation	No internal flooding of any SSCs (peak levels and depths reported in ***Table 3-1***)	No internal flooding of any SSCs (peak levels and depths not reported)
2	Spray Pond with coincident wind-wave activity	Peak levels for LIP + wind-waves = 683.3 ft NGVD29 (Spray Pond embankment) and 684.3 ft NGVD29 (ESSW Pumphouse)	Peak levels for critical scenario (i.e., SPF + wind-waves) = 684.6 ft NGVD29 (Spray Pond embankment) and 684.8 ft NGVD29 (ESSW Pumphouse)
3	River Flooding: Susquehanna River	Screened Out. Peak flood level = 548.7 ft NGVD29 (approximately 121 ft below site grade elevation)	Screened Out. Peak flood level with coincident wind-waves = 548 ft NGVD29 (approximately 122 ft below site grade elevation)
4	River Flooding: Walker Run	Screened Out. Peak flood level = 675.7 ft NGVD29 (approximately 44.3 ft below topographic divide low point)	N/A[2]
5	Combined Effects: Susquehanna River PMF with antecedent rainfall/ snowmelt and coincident wind-wave activity	Screened Out. Peak flood levels approximately 54 ft below site grade elevation when the PMF depth in the river is doubled	N/A[2]
6	Combined Effects: Walker Run PMF with antecedent rainfall/ snowmelt and coincident wind-wave activity	Screened Out. Peak flood levels approximately 28 ft below topographic divide low point when the PMF depth in the river is doubled	N/A[2]
7	Dam Failure Flooding	Screened Out. Peak flood level approximately 59 ft below site grade elevation	Screened Out. Peak flood level more than 120 ft below site grade elevation
8	Coastal Flooding (including storm surge, seiche, and tsunami)	Screened Out	Screened Out
9	Ice-Induced Flooding	Screened Out	Screened Out
10	Channel Migration/Diversion Flooding	Screened Out	Screened Out
11	Cooling Tower Basin Rupture	No internal flooding of any SSCs (peak levels and depths reported in ***Table 3-2***)	No internal flooding of any SSCs (peak levels and depths not reported in the FSAR)

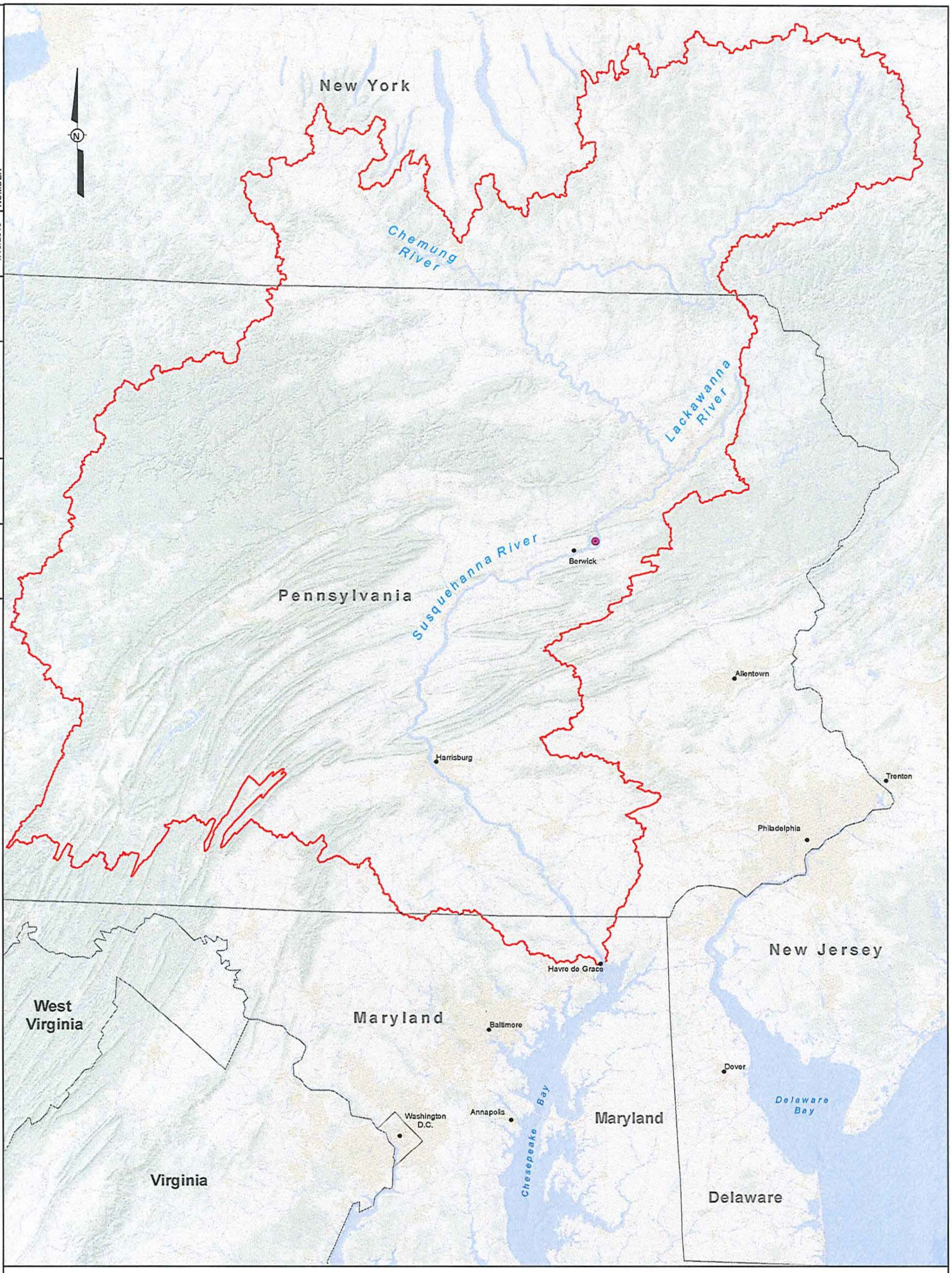
Notes:

1 Reference: PPL, 2013, "Final Safety Analysis Report, Susquehanna Steam Electric Station Units 1 and 2," Revision 66, February 2013.

2 N/A = Not applicable because this flood-causing mechanism is not reported in the FSAR.

FIGURES

DRAWN BY	GJO	20 NOV 2014	CHECKED BY	APPROVED BY	JAO	JAD	1/19/2015	1/19/2015	GIS FILE NUMBER	12-4834-GIS-B001



25 12.5 0 25 50 Miles

Coordinate System: NAD 1983 UTM Zone 18N
Projection: Transverse Mercator

RF: 1:1,490,700

Legend

- Susquehanna River Basin
- Susquehanna Steam Electric Station
- Cities/Towns

Reference:
Background Image: Environmental Systems Research Institute (ESRI), "Ocean/World Ocean Base"
Website: http://goto.arcgisonline.com/maps/Ocean/World_Ocean_Base
Date Accessed: November 20, 2014

Figure 1 - 1

Regional Location Map

Prepared For

Susquehanna Steam Electric Station
Flood Hazard Reevaluation Report

RIZZO ASSOCIATES

DRAWN BY	GJO	20 NOV 2014
CHECKED BY	JAO	1/19/2015
APPROVED BY	JAD	1/19/2015
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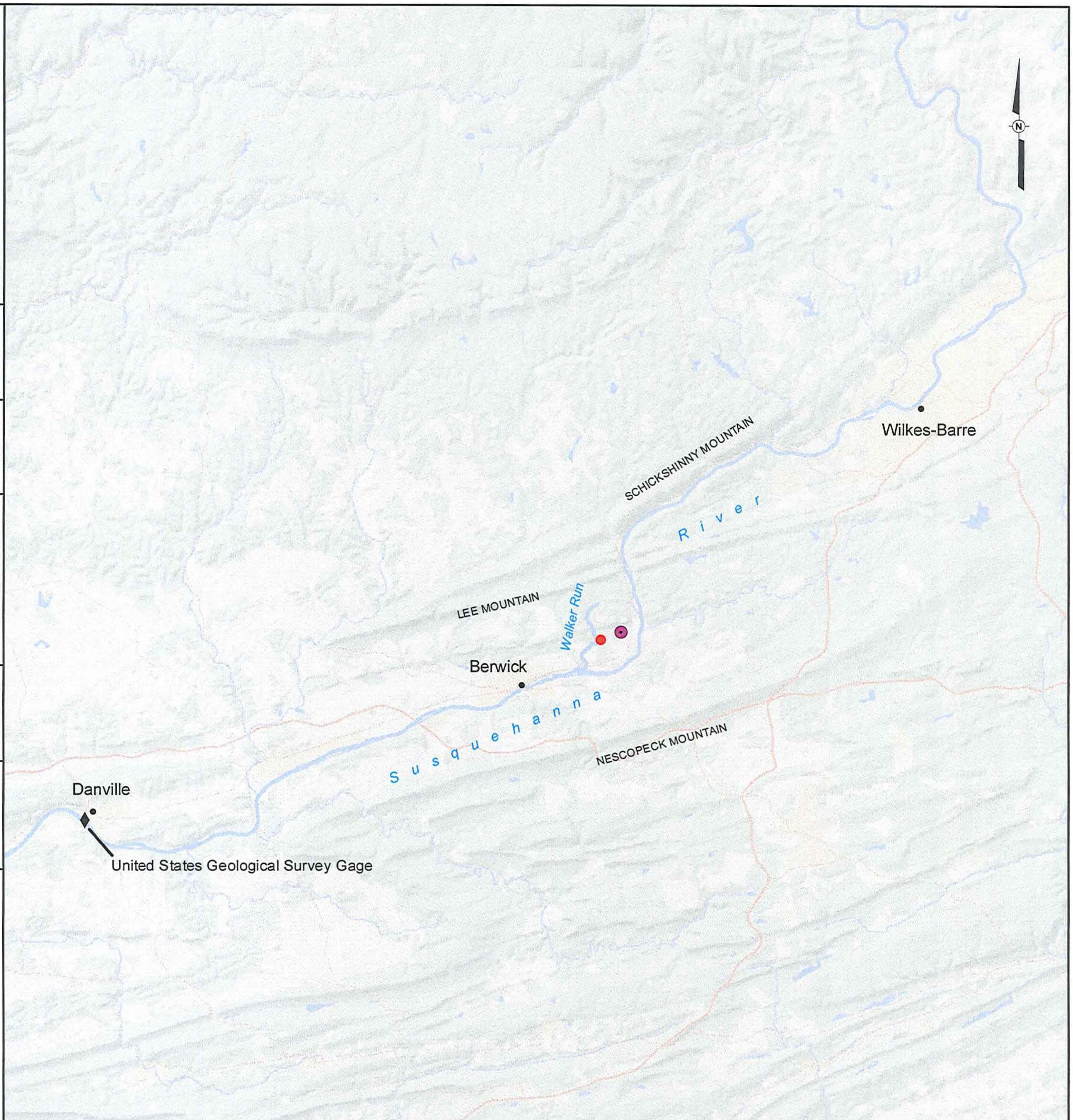


0 5 10 20 Miles

Coordinate System: NAD 1983 UTM Zone 18N
Projection: Transverse Mercator
RF: 1:404,150

Legend

- Susquehanna Steam Electric Station
- Bell Bend Nuclear Power Plant (proposed)
- United States Geological Survey Gage
- Cities/Towns

Reference:
Background Image: Environmental Systems Research Institute (ESRI), "Ocean/World Ocean Base"
Website: http://goto.arcgisonline.com/maps/Ocean/World_Ocean_Base
Date Accessed: November 20, 2014

Figure 1 - 2

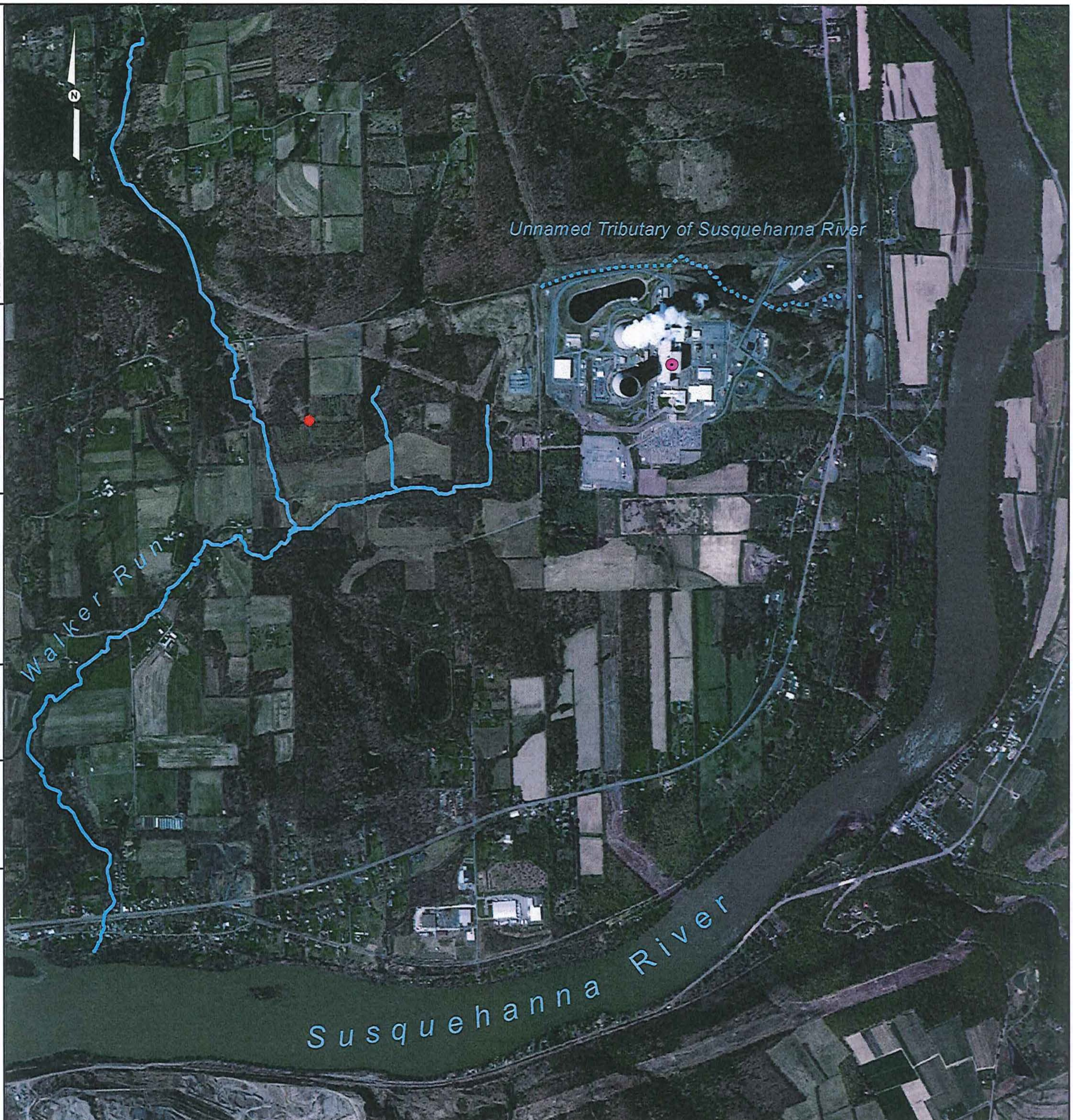
General Location Map of the Site

Prepared For

Susquehanna Steam Electric Station
Flood Hazard Reevaluation Report

RIZZO ASSOCIATES

DRAWN BY	GJO	20 NOV 2014
CHECKED BY	JAO	1/19/2015
APPROVED BY	JAD	1/19/2015
GIS FILE NUMBER	12-4834-GIS-A002	



0 0.1 0.2 0.4 0.6 0.8 1 Miles

Coordinate System: NAD 1983 UTM Zone 18N
Projection: Transverse Mercator
RF: 1:25,000

Legend

- Susquehanna Steam Electric Station
- Bell Bend Nuclear Power Plant (proposed)
- Walker Run
- Unnamed Tributary of Susquehanna River

Reference:
Background Image: Environmental Systems Research Institute (ESRI), "Ocean/World Imagery"
Website: http://goto.arcgisonline.com/maps/World_Imagery
Date Accessed: November 20, 2014

Figure 1 - 3

Site Area Map

Prepared For

Susquehanna Steam Electric Station
Flood Hazard Reevaluation Report

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CHECKED BY	JAO	1/19/2015
APPROVED BY	JAD	1/19/2015
GIS FILE NUMBER	12-4834-GIS-B002	

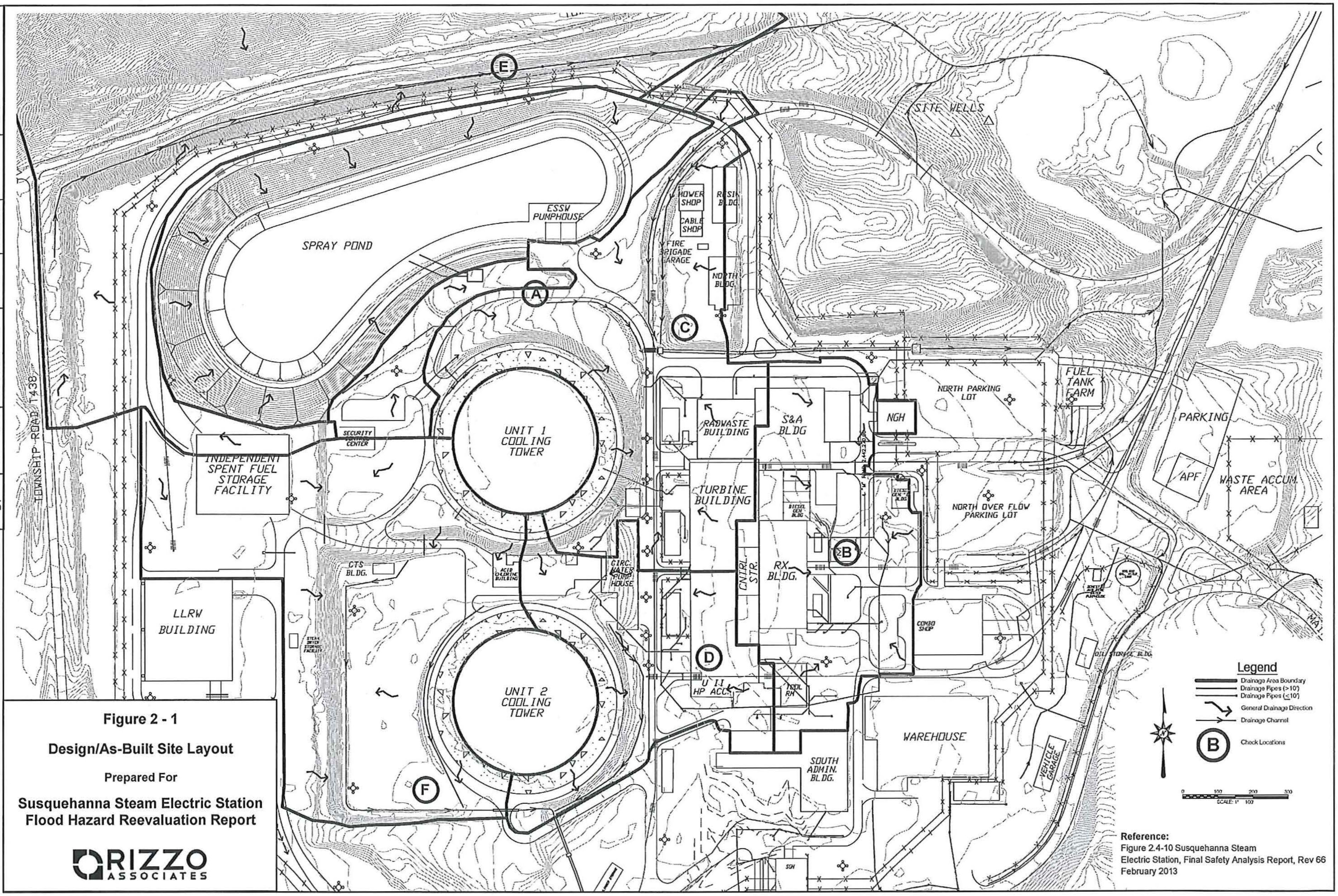

Figure 2 - 1

Design/As-Built Site Layout

Prepared For

Susquehanna Steam Electric Station Flood Hazard Reevaluation Report

RIZZO ASSOCIATES

Reference:
Figure 2.4-10 Susquehanna Steam Electric Station, Final Safety Analysis Report, Rev 66 February 2013

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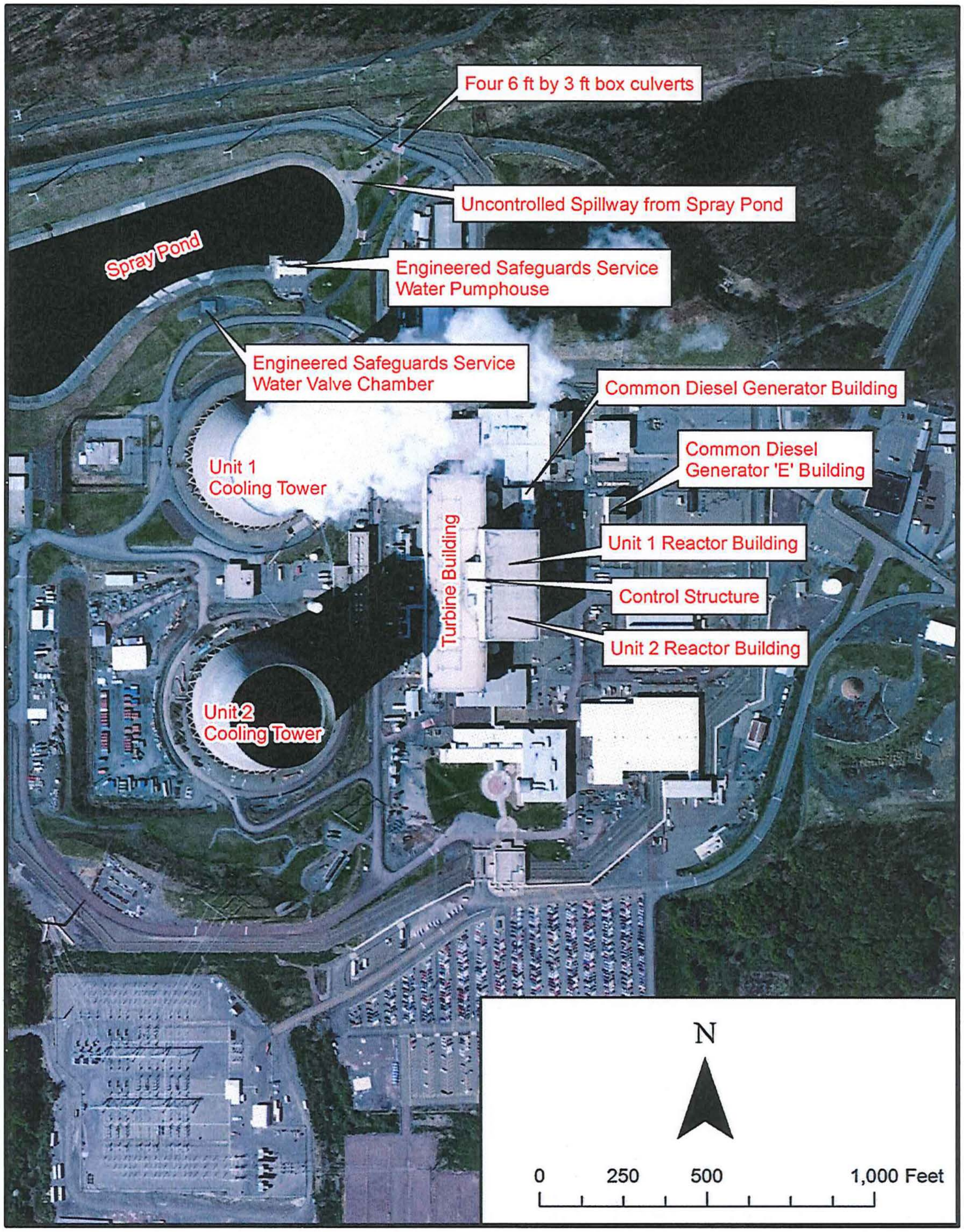

Figure 2 - 2

Locations of Selected Structures

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Reference:
Background Image: Environmental Systems Research Institute (ESRI), "Ocean/World Imagery"
Website: http://goto.arcgisonline.com/maps/World_Imagery
Date Accessed: November 20, 2014

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DRAWN BY	GJO	27 OCT 2014	
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APPROVED BY	JAD	1/19/2015	
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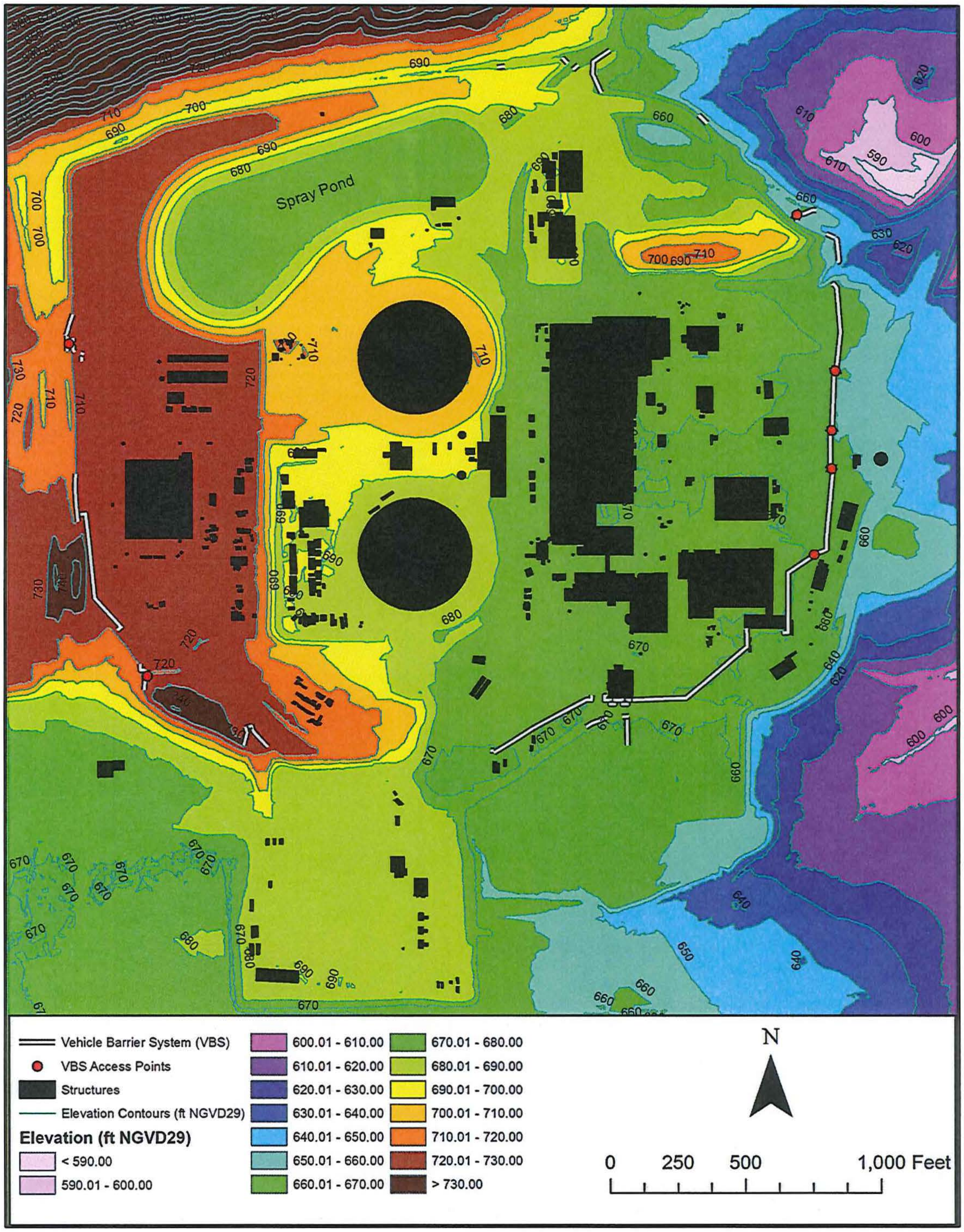

Note: 1. Elevation shown for spray pond reflects water surface elevation at the time of the LIDAR survey (2006).
2. Refer to Figure 2-2 for building and structure names.

Figure 2 - 3

Present-Day Site Layout and Topography

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Reference: Pennsylvania Spatial Data Access, PAMAP Program Digital Elevation Model of Pennsylvania (PASDA, 2014)

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GIS FILE NUMBER 12-4834-GIS-A007

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27 OCT 2014	1/19/2015	1/19/2015

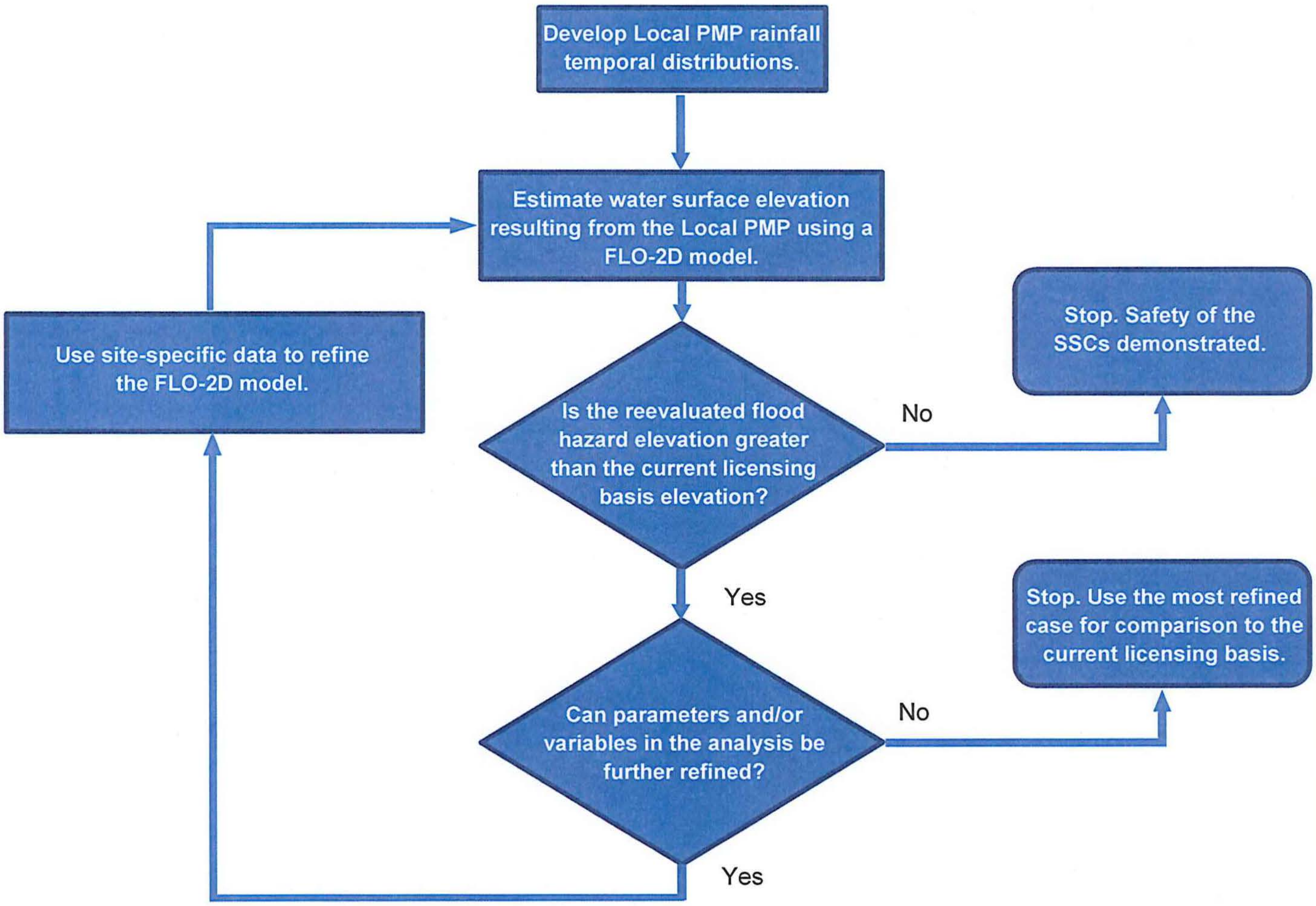

Figure 3 - 1

The Hierarchical Hazard Assessment Diagram for Local Intense Precipitation Flooding Analysis

Prepared For

Susquehanna Steam Electric Station Flood Hazard Reevaluation Report

RIZZO ASSOCIATES

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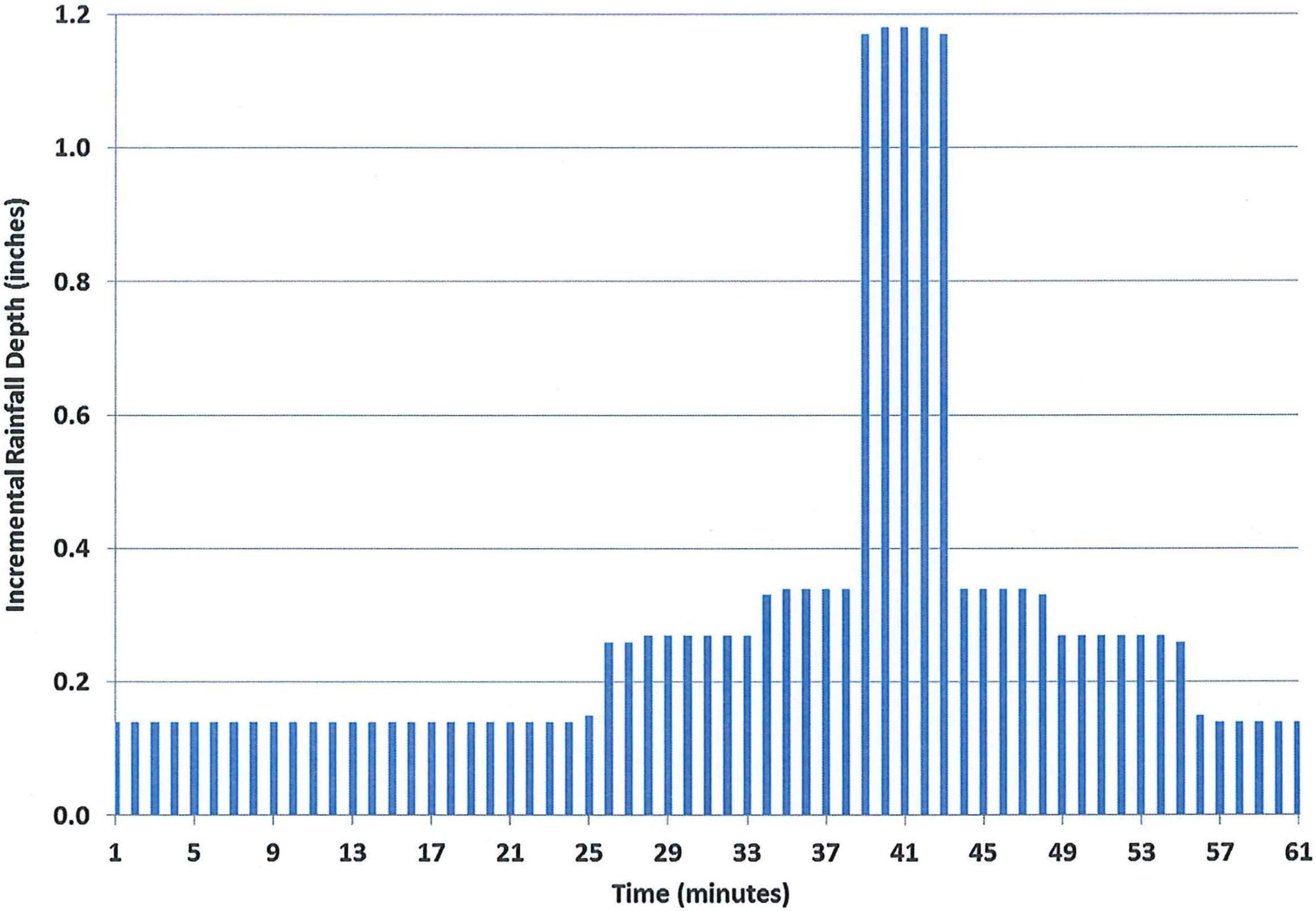

Figure 3 - 2

Critical Local Intense Precipitation Hyetograph

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Flood Hazard Reevaluation Report

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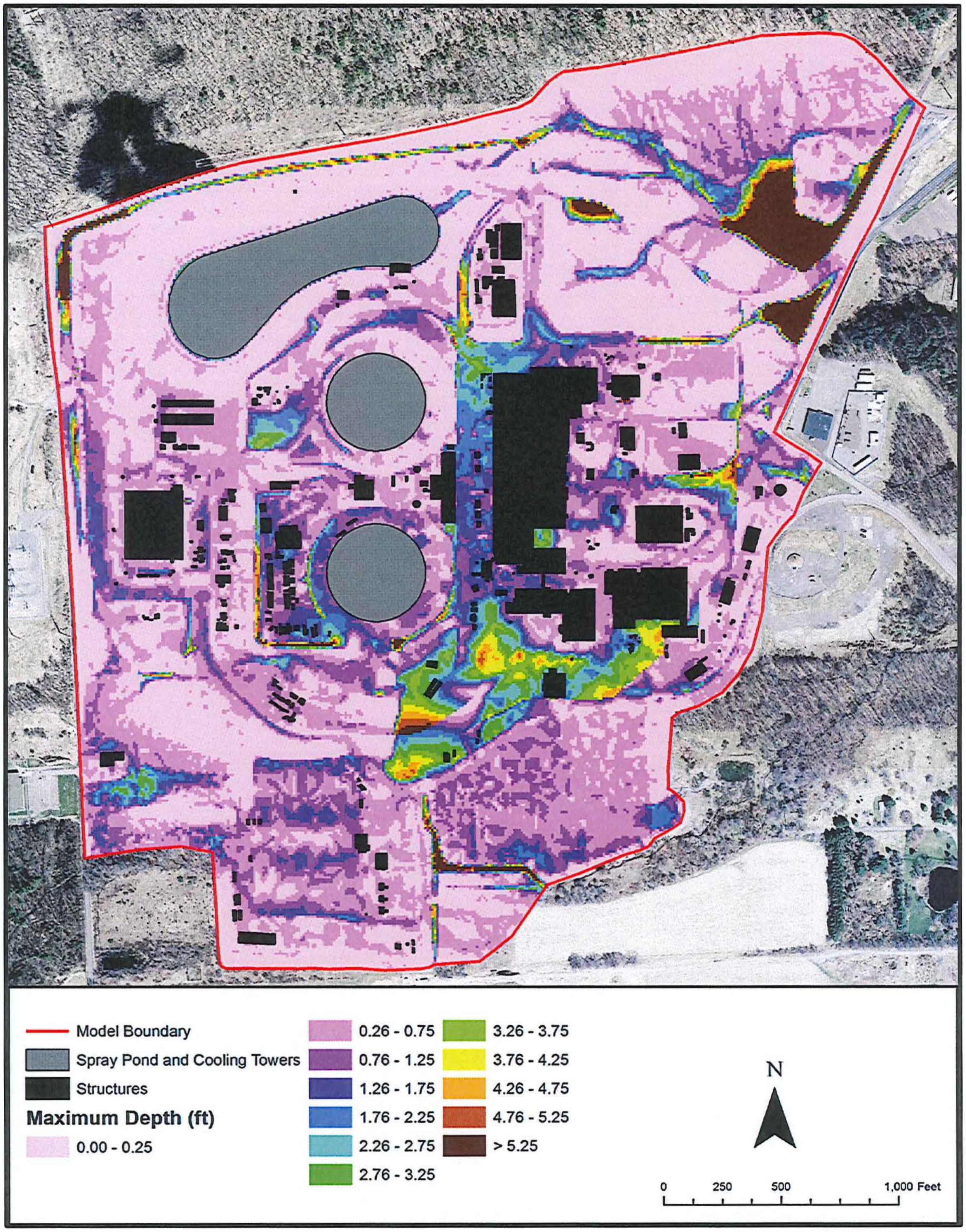

Note: Refer to Figure 2-2 for building and structure names.

Figure 3 - 3

Inundation Map for Local Intense Precipitation

Prepared For

Susquehanna Steam Electric Station Flood Hazard Reevaluation Report

RIZZO ASSOCIATES

Reference:

Background Image: Pennsylvania Spatial Data Access, PAMAP Program Digital Elevation Model of Pennsylvania (PASDA, 2014)

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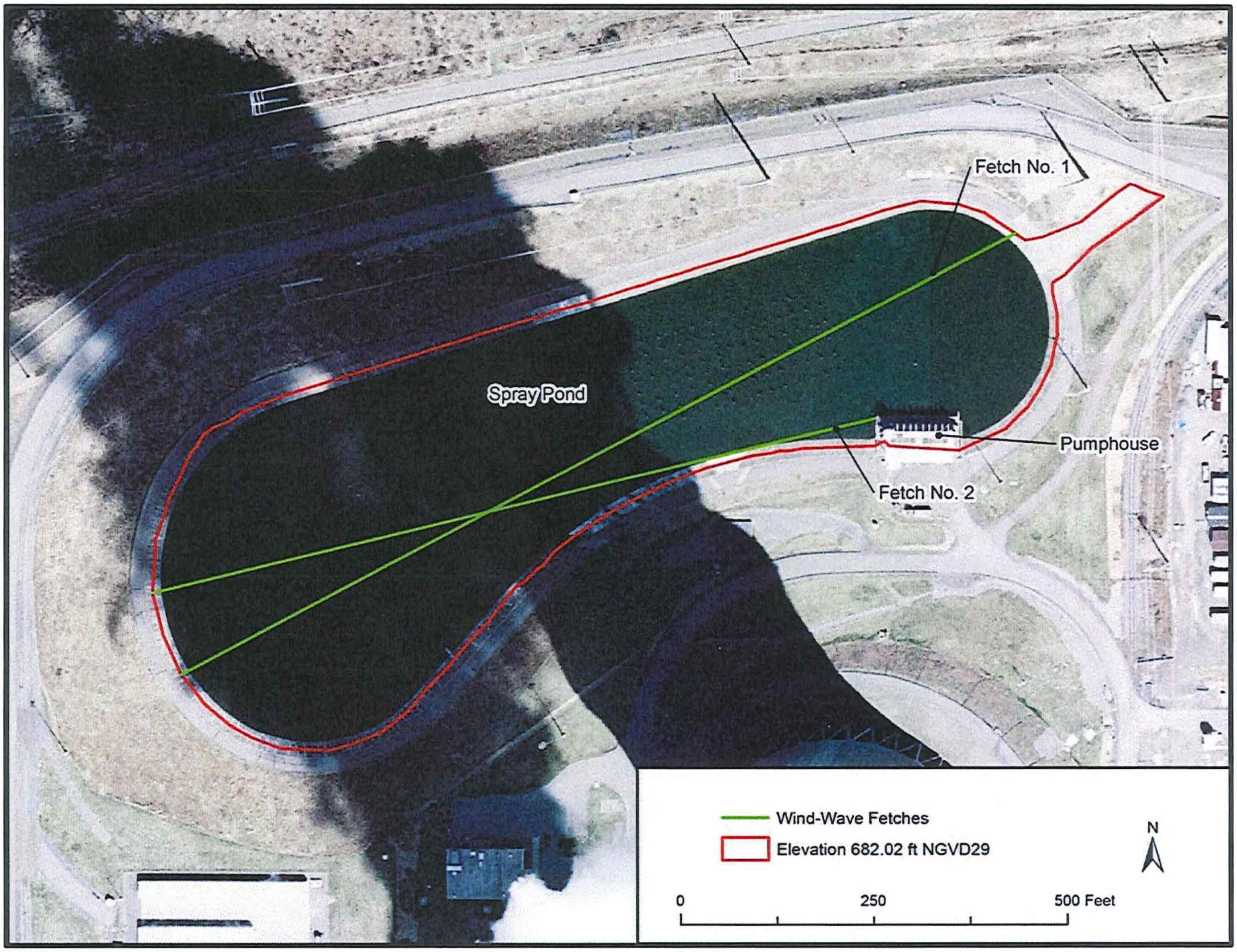

Note: Elevation 682.02 ft NGVD29 is the maximum local intense precipitation water level in the Spray Pond, which was used as the antecedent condition for the coincident wind-wave analysis.

Figure 3 - 4

Fetch Locations for Wind-Wave Activity Coincident with Local Intense Precipitation Flooding

Prepared For

Susquehanna Steam Electric Station Flood Hazard Reevaluation Report

Reference:

Background Image: Pennsylvania Spatial Data Access, PAMAP Program Digital Elevation Model of Pennsylvania (PASDA, 2014)

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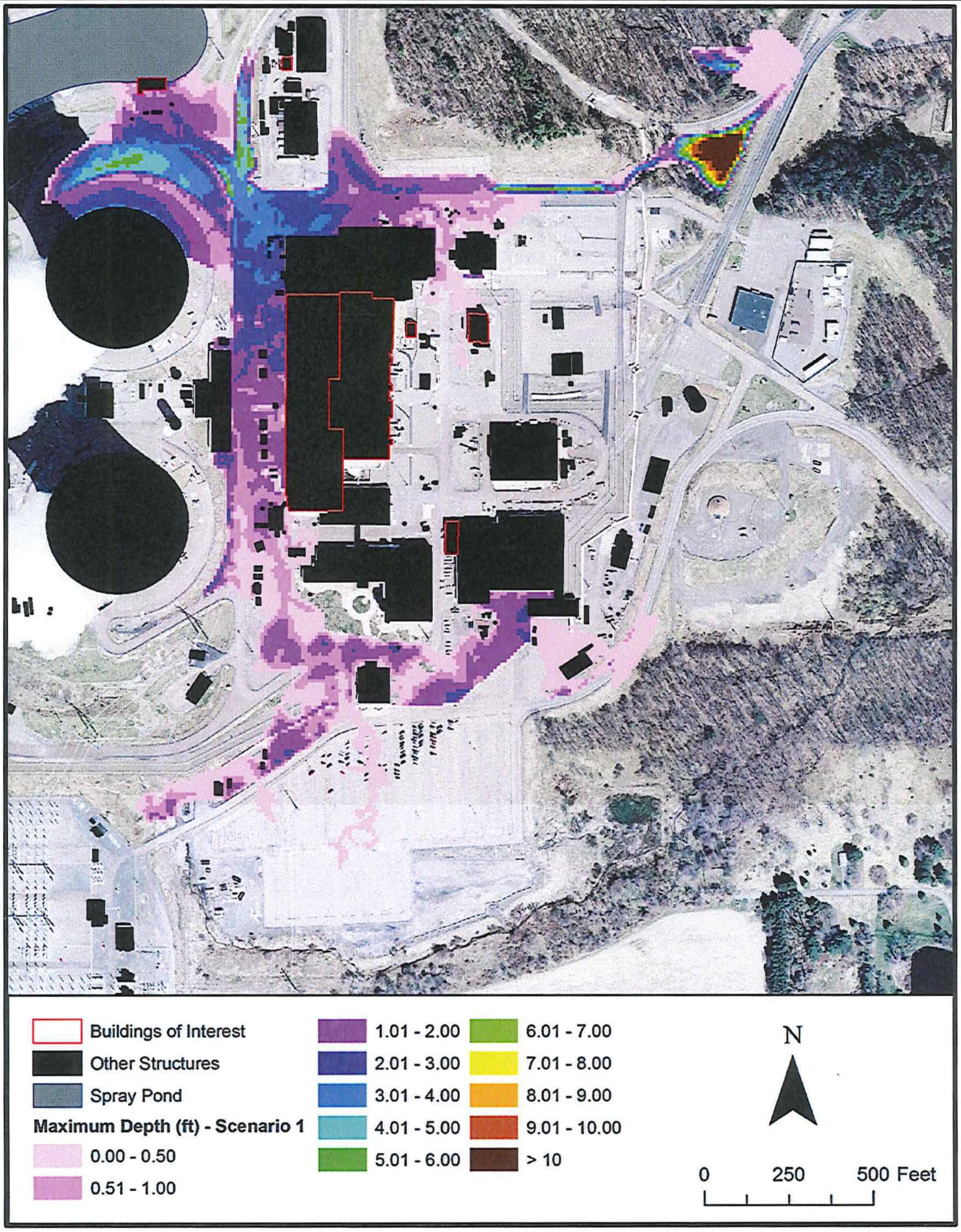


Note: Refer to Figure 2-2 for building and structure names.

Figure 3 - 5

Inundation Map for Cooling Tower Basin Rupture Scenario 1: Critical ESSW Pumphouse Scenario

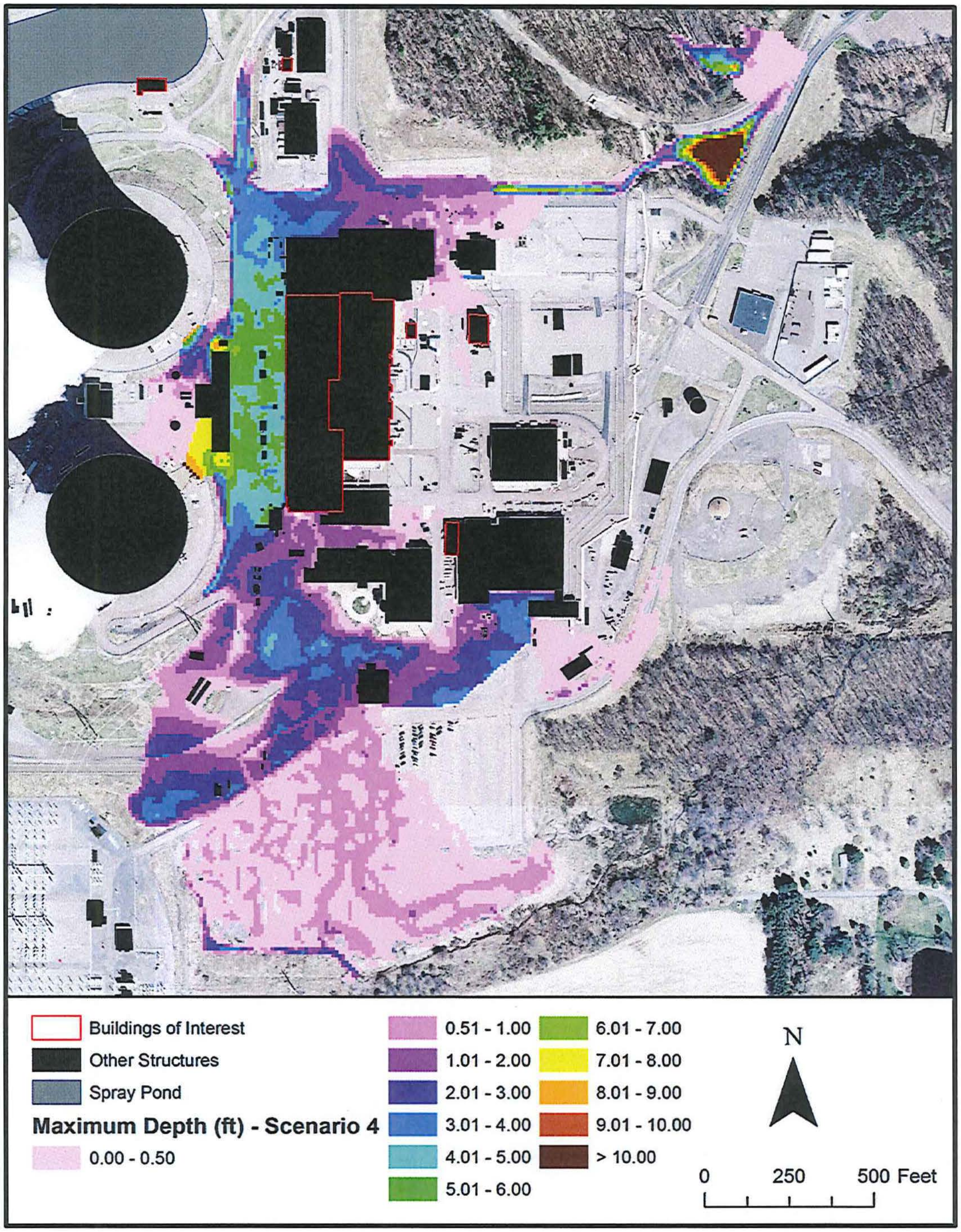
Prepared For

Susquehanna Steam Electric Station Flood Hazard Reevaluation Report

RIZZO ASSOCIATES

Reference:

Background Image: Pennsylvania Spatial Data Access, PAMAP Program Digital Elevation Model of Pennsylvania (PASDA, 2014)



Note: Refer to Figure 2-2 for building and structure names.

DRAWN BY	CHECKED BY	APPROVED BY	GIS FILE NUMBER
GJO 28 OCT 2014	JAO 1/19/2015	JAD 1/19/2015	12-4834-GIS-A013

Figure 3 - 6

Inundation Map for Cooling Tower Basin Rupture Scenario 4: Critical Turbine Building Inflow Volume Scenario

Prepared For

Susquehanna Steam Electric Station Flood Hazard Reevaluation Report

RIZZO ASSOCIATES

Reference:

Background Image: Pennsylvania Spatial Data Access, PAMAP Program Digital Elevation Model of Pennsylvania (PASDA, 2014)