Attachment 3

Proposed Updated Final Safety Analysis Report Text Pages
(Retyped; for Information Only)
(105 pages follow)

(Reference 32). Seventy-three wells were completed in 1983 which were producers of natural gas or natural gas and oil. Exploration for gas and oil may continue in those parts of Lake County that are not developed (Reference 34).

<Figure 2.2-4> depicts locations of gas and oil wells within five miles of the site. No commercial extraction of gravel or sand exists within five miles of the site (Reference 35).

2.2.2.4 Waterways

<Section 2.2.1.3>, Item d, contains information on channels, docks and anchorages within five miles of the plant center. <Figure 2.2-2> shows shipping lanes in the vicinity of Perry. <Table 2.2-6> and <Table 2.2-7> contain information on types and amounts of cargo transported on the Great Lakes in 1982 and the 1982 combined traffic by type of vessel, respectively. The trips and draft of vessels operating in Lake Erie shipping lanes near Perry are presented in <Table 2.2-8>.

The intake structures for PNPP are located approximately 2,600 feet offshore perpendicular from the shoreline. The discharge structure is approximately 1,650 feet offshore. These are located in 20 to 23 feet of water, where the structures have been constructed a minimum of 12 feet below the lake low water datum as provided on (Reference 88).

The closest water intake structure on Lake Erie is located at the now inoperative IRC Fiber plant, about 3.5 miles west-southwest of the plant center.

The closest shipping channel is approximately two miles from the plant center at Perry with a depth of approximately 40 feet. It parallels the Ohio coastline between the ports of Fairport Harbor and Ashtabula (Reference 4) (Reference 36).

the plant site. The unloading area is located on the north side of the storage tank.

The truck unloading area is located such that all drainage from the unloading area drains into the diked area, and the diked area is designed to be capable of containing the complete volume of the tank and the delivery truck, with a one foot freeboard.

The area around the fuel oil tank dike is contoured to ensure that drainage from the area is away from the plant proper and toward the Remnant Minor Stream Channel. Normally, any leakage from the tank will be contained within the dike whose base is one foot below grade level. The only common mode failure that would cause rupture of both the tank and the reinforced concrete dike is the postulated seismic event. The slopes around the dike are such that the spillage resulting from the postulated simultaneous breaks would be directed away from the plant, as shown on <Figure 2.1-3>.

2.2.3.1.3.2 Gas Line

A 20-inch buried natural gas line runs adjacent to the Perry site as shown on <Figure 2.2-3>. It is unlikely that this line will rupture. If such a massive leak with immediate ignition did occur, the fire would continue to burn until the line is isolated and the methane gas in the isolated section of pipe is consumed. Sufficient heat radiation could be given off from such a fire to start a forest fire in the adjacent wooded area during dry seasons. However, the heat radiation at the plant buildings would be less than 200 Btu per hour per square foot which would not be sufficient to affect the plant safety. The forest fire is discussed in <Section 2.2.3.1.3.3>.

2.2.3.1.3.3 Forest Fires

The presence of greater than 350 feet of clear space between forested areas and powerblock structures reduces the potential hazard of forest fire to a minimum (the presence of trees and bushes in landscaped areas does not affect this evaluation). Plant construction is fire resistive with building roof construction of either reinforced concrete or metal deck. The metal decks meet the requirements of Factory Mutual Class I roof decks. Therefore, the hazard from any flying brands is minimized. The yard fire protection system will provide a ready source of water to attack such a forest fire and to apply cooling water to buildings and other structures.

2.2.3.1.3.4 HWC Hydrogen and Oxygen Storage Area

No serious fire exposure hazard is presented by the 9,000 gallon capacity cryogenic liquid hydrogen storage tank or the six 8,350 scf (at 2,400 psi) gaseous hydrogen storage tanks to onsite buildings because of their separation. The hydrogen storage tanks are located approximately 1,240 feet southeast of the fuel handling building, the nearest safety-related plant structure. The cryogenic liquid hydrogen storage tank and gaseous hydrogen storage tanks supply hydrogen for operation of the Hydrogen Water Chemistry (HWC) System.

The location of the hydrogen storage facility complies with the separation distance outline in EPRI NP-5283-SR-A, “Guidelines for Permanent BWR Hydrogen Water Chemistry Installations-1987 Revision”, NFPA 50A (gaseous hydrogen systems at consumer sites) and NFPA 50B (liquefied hydrogen systems at consumer sites), as well as OSHA Standards 29 CFR 1910.103 (hydrogen). This separation distance ensures that the thermal flux from a potential hydrogen gas fireball or the blast overpressure from a potential hydrogen blast will not cause failure of any safety-related structures. The routing and delivery schedule for the hydrogen delivery truck meets the requirements of <Regulatory Guide 1.91> as specified in EPRI NP-5283-SR-A. Delivery of

78. Engineering Division, Crane Company, "Flow of Fluids," Technical Paper No. 410, Crane Company, Chicago, Illinois, 1957.

79. J. J. Duggan, C. H. Gilman, P. F. Fisher, "The Requirements for Relief of Overpressure in Vessels Exposed to Fire," presented at ASME Meeting, June 1943.

80. (Deleted)

81. (Deleted)

82. Personal communication, Robert Archer, Lake County Emergency Management Agency. Confirmation of Uniroyal and Pressure Vessel Service of Ohio ceased operations in Lake County, Ohio, May 15, 2001.

83. Personal communication, Richard Woodworth, Owner, Woodworth Airstrip, Madison, Ohio, May 15, 2001.

84. Personal communication, Bill Pfarr, Manager, Lost Nation Airport, Willoughby, Ohio, May 15, 2001.

85. Calculation 5.6.4, Rev. 3, 5-16-06.

86. Calculation 5.6.2, Rev. 2, 5-17-06.

87. Calculation 5.6.1, Rev. 4, 5-15-06.

88. U.S. Geological Survey quadrangle maps of Perry and Madison, Ohio, PNPP Drawing 736-0201-00000.

surface, the equivalent of a dust devil over water. The latter of these two, known as a common waterspout, is much less intense than a tornado, as the energy available for its formation and maintenance is small. Tornadic waterspouts spawned from clouds are much more intense and can have tornadic intensities, although on the average they are weaker than tornadoes (Reference 9).

There are no pertinent data available concerning the intensity of Lake Erie waterspouts. However, these waterspouts could have tornadic strength if associated with a severe thunderstorm (most commonly accompanying a cold front or squall line). For the reported waterspouts in the Perry site area, there are no damage reports nor are there any reports of waterspouts coming onshore (Reference 8) (Reference 16). Since the waterspout intensity will not exceed the design-basis tornado, there will not be catastrophic damage to safety class surface structures. The water surface below the waterspout can be raised or lowered dependent on which force has the greatest effect, the atmospheric pressure reduction or the wind force. The waterspout does not lift a significant amount of water (i.e., the depth of penetration is relatively small) (Reference 9). Therefore, waterspouts will not have a significant effect on the plant's intake structure, which is located a minimum of 12 feet below the mean low water datum as provided on (Reference 55).

2.3.1.2.3 Extreme Winds

The extreme mile wind speed is defined as the one-mile passage of wind with the fastest speed and includes all meteorological phenomena except tornadoes. Annual fastest mile wind data at Cleveland for the 30-year period from 1948 to 1977 (Reference 3) were used to determine predicted extreme wind speeds for the PNPP site for recurrence intervals of 50 and

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(Reference 25). The PMWP is taken to be the probable maximum 48-hour precipitation during the winter months of December, January and February in the PNPP region. This is based on the assumption that conditions could exist during these months for all of the PMWP to remain on the ground as a live load either as additional snowfall or as liquid precipitation absorbed by the snowpack. For the Perry region, the PMWP is approximately 12 inches of liquid precipitation as determined in (Reference 69) of <Section 2.4>, which corresponds to 62.4 psf at 5.2 psf per inch of precipitable water. These calculations yield a conservative design basis snowload value (ground-level equivalent) for Category I structures of 82.4 psf. The ground-level flooding effects of the PMWP are discussed in <Section 2.4>.

Hail can occasionally occur at the Perry site (associated with well-developed thunderstorms) and at times may be intense. A review of data for the 16-year period, 1962 to 1977, indicates that there were 20 reported cases of hail in Lake County (where the Perry site is located) and in the immediately surrounding counties of Ashtabula, Cuyahoga, Geauga, and Trumbull (Reference 16). Nine of the cases occurred in the Cleveland area. Of the reported cases, the largest hailstones reported were of "tennis ball" size in the Hirmal-Garretsville area approximately 35 miles south of the Perry site. Of the 20 reported cases of hail, eleven recorded hailstones >3/4 inches in diameter (seven cases did not report any hailstone size). Usually during a hailstorm, there is a spectrum of hailstone sizes and there is a tendency to report the largest sizes. The average number of hail days per year in northeast Ohio is approximately two at any one site (Reference 27).

An examination of the 16-year period, 1962 to 1977, indicates that there were nine documented cases of ice storms in Lake County and the immediately surrounding counties (Reference 16). Two of these storms affected the entire state of Ohio while the rest were widespread over northern and northeastern Ohio. All cases were associated with a number of traffic accidents, downed power lines and downed tree limbs.

18. U.S. Department of Commerce, "Extreme Wind Speeds at 129 Stations in the Contiguous United States," NBS Building Science Series 118, March 1979.

19. Chancery, M. J., Historical Extreme Winds for the United States - Great Lakes and Adjacent Regions, <NUREG/CR-2890>, August 1982.

20. Huss, P. O., Relations Between Gusts and Average Wind Speeds for Housing Load Determination, DGAF 140, Daniel Guggenheim Airship Institute, Cleveland, Ohio, 1946.

21. Personal communication, NWS Duty Observer, Cleveland-Hopkins Airport, February 6, 1978.

22. Golde, R. H., "Protection of Structures Against Lightning," Proceedings, The Institute of Electrical Engineers, Vol. 115, No. 10, pp 1523-1529; October 1968.

23. December 1970: Extremes of Snowfall, Weatherwise. Princeton, New Jersey.

24. U.S. Nuclear Regulatory Commission, Winter Precipitation Loads, Site Analysis Branch Position, March 24, 1975.

25. American National Standards Institute, ANSI A58.1-1972; Building Code Requirements for Minimum Design Loads in Buildings and Other Structures. ANSI, Inc., New York, New York.

26. DELETED.

52. Letter From D. R. Davidson, Vice President, Engineering, Perry Nuclear Power Plant, to Director of Nuclear Reactor Regulation, U.S. Nuclear Regulatory Commission, Subject: Docket Nos. 50-440, 50-441, Terrain-Corrected Atmospheric Dispersion Factors, July 2, 1976.

53. Chandler, M. W., J. R. Fleming, S. L. Shipley, M. S. Tapparo, NUSPUF - A Segmented Plume Dispersion Program for the Calculation of Average Concentrations in a Time-Dependent Meteorological Regime, NUS-TM-260, NUS Corporation, Gaithersburg, Maryland, March 1976.

54. Timbre, K., Mazaika, M., Eighth Annual Report of the Meteorological Programs at the Perry Nuclear Power Plant for the Partial Annual Period May 1, 1983, to December 31, 1983, NUS-4536, NUS Corporation, Gaithersburg, Maryland, March 1985.

55. U.S. Geological Survey quadrangle maps of Perry and Madison Ohio, PNPP Drawing 736-0201-00000.

2.4 HYDROLOGIC ENGINEERING

2.4.1 HYDROLOGIC DESCRIPTION

2.4.1.1 Site and Facilities

The Perry Nuclear Power Plant is located in Lake County, Ohio, approximately seven miles northeast of Painesville. The southern plant site boundary line is 3,100 feet from the shoreline of Lake Erie on the west side of the site and 8,000 feet on the east side. Grade elevations in the immediate plant area prior to plant construction varied between 620.0 feet and 623.0 feet based upon the USGS datum. The maximum monthly average level of record for Lake Erie is 575.4 feet (USGS) (Reference 61); therefore, no problems of site flooding exists due to elevated lake levels.

The construction of the plant resulted in changes in local drainage patterns, runoff characteristics and in the modification of two streams located on the plant property. Final topography is shown in <Figure 2.4-3> and an aerial photograph of the site, prior to plant construction, is shown in <Figure 2.1-4>.

Protection is provided for safety-related structures, exterior systems and access equipment against flooding from Lake Erie, surface runoff and local intense precipitation. This is accomplished by location, arrangement and design of these structures, systems and equipment, and flood protection features as discussed in the following sections.

2.4.1.2 Hydrosphere

Lake Erie, the major hydrologic feature of this location, will provide cooling water for the plant. The lake will not influence the surface water characteristics of the site since the mean lake level is in excess of 40 feet below plant grade.

In the vicinity of the site, the coastal watershed is drained by several small streams. These streams have cut deep channels as they approach the lake in the otherwise flat terrain of this region. The width of this coastal watershed in the site area is approximately 4.5 miles with the ground falling away sharply to the south of the ridge into the Grand River Basin.

Topographically, the site lies in the Old Lake-bed Region of northeast Ohio.

Two parallel streams run close to the plant area. The larger stream, called the Major Stream, has a drainage basin of 7.44 square miles (Reference 70) and runs northwestward within 1,000 feet of the southwest corner of the plant. The smaller stream, called the Diversion Stream, which has a drainage area of only 0.59 square mile (Reference 71), borders the plant area to the east. The drainage areas are shown in <Figure 2.4-1>. The smaller stream was originally diverted prior to construction and had been historically referred to as the Minor Stream. The name change to the Diversion Stream coincides with a post-construction modification (Reference 75) to divert the stream further to the east. A portion of the originally engineered channel of the Minor Stream remains located between the plant area and the Diversion Stream. This section of the stream no longer functions as a traditional stream. The "Remnant Minor Stream," as it is now named, functions as a drainage swale located entirely within the Local Intense Precipitation (LIP) domain.

The safety-related structures of the plant are located between the watersheds of the adjacent streams. The site is not subject to flooding from surface runoff originating from either stream's drainage basin. Site drainage ultimately is directed to Lake Erie either directly or via the following notable drainage paths: ESW Swale, Remnant Minor Stream, Major Stream, the Barge Slip, and the Northwest Impoundment.

No recorded data exists for either of the adjacent streams. However, base flow estimates indicate that the average flow for the Major and Diversion streams would be approximately 5 cfs and 0.78 cfs, respectively (Reference 70 and Reference 71). Older local inhabitants living on Lockwood and Center Roads did not recall either the Major or Minor Streams level overtopping Center Road. High water marks on the headwalls of the preexisting Center Road bridge over the major stream indicated that the maximum depth at this point had been approximately four feet. The smaller stream previously passed through a 4-foot by 5-foot rough stone culvert under Center Road.

There are no users of water from the Major or Diversion Streams. Use of groundwater is discussed in <Section 2.4.13>. Possible

contamination of groundwater is also discussed in <Section 2.4.13> and the degree of contamination as assessed in that section will also apply to both streams.

Lake Erie has an area of 9,970 square miles and is only slightly larger than Lake Ontario, which has the smallest surface area of any of the Great Lakes. It is the shallowest of the lakes, with a maximum depth of 210 feet and an average depth of 58 feet. Also, Lake Erie has the smallest volume, 110 cubic miles (Reference 1).

The lake is about 241 miles long with a maximum width of 57 miles. The long axis of the lake lies in a general northeast-southwest direction.

The drainage basin of Lake Erie, including Lake St. Clair, is about 29,650 square miles. The mean annual flow from the Detroit River at the western head of the lake is 178,000 cfs which accounts for about 90 percent of the inflow into Lake Erie. The mean annual outflow into Lake Ontario is about 194,000 cfs through the Niagara River and about 8,000 cfs through the Welland Canal (Reference 1).

Lake Erie level records have been maintained by the Lake Survey Center (NOAA) since 1860 (Reference 61). The average level is 572.3 feet above mean sea level, mean tide, New York City, which is the USGS datum. The 1973 monthly levels were exceptionally high, setting a new maximum monthly mean level of 575.4 feet (USGS) in June, 1973 (Reference 61). The lowest average monthly recorded level was 569.3 feet (USGS) in February, 1936 (Reference 61). Corresponding levels relative to the 1955 International Great Lakes Datum (IGLD 55) are 1.9 feet less. The Lake Erie Low Water Datum (LWD) is at Elevation 570.5 feet (USGS) and Elevation 568.6 feet (IGLD) (Reference 2). Vertical datum conversion values applicable to the site are provided on <Figure 2.4-2>.

Minimum lake levels usually occur in February when precipitation throughout the Great Lakes drainage basin is being stored in the form

of ice and snow. Maximum levels occur in mid-summer when the full effect of

the runoff from the drainage basin is felt. Fluctuations of several feet, but of short duration, are caused by wind effects. Northerly winds raise lake levels in the plant area, while winds from the south and east tend to lower the lake levels (Reference 3).

2.4.2 FLOODS

2.4.2.1 Flood History

No records of flooding in the plant area of the site exist, either from the two adjacent streams draining the coastal watershed, or from Lake Erie. The terrain is relatively flat and gently sloping toward the lake. The soil is relatively permeable and contains sand layers in the upper reaches of the drainage area. The ground surface in much of the catchment area is forested with a heavy mulch ground cover. Due to the flat terrain, permeable upper soil layers and the small catchment areas <Section 2.4.1.2>, it is unlikely that this location has ever been subjected to flooding or will experience severe flooding from surface runoff in the future.

2.4.2.2 Flood Design Considerations

The probability of any flooding in the area of the site is exceptionally low. The site area is passively protected from probable maximum flood hazards originating from Lake Erie and the adjacent streams. Studies discussed in <Section 2.4.3> have shown that even if a PMF is experienced, the streams will be contained within their respective drainage basins with no site impact except for the overtopping of the crossing at the plant main access road, which would temporarily prevent road access. Topography along the plant-side bank of the major stream will preclude flooding of the site by the PMF, allowing the plant to continue uninterrupted operation. Diversion Stream PMF is prevented from reaching the site by the presence of an earthen berm located between the stream’s engineered channel and the Remnant Minor Stream channel, allowing the plant to continue uninterrupted operation.

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Flooding from Lake Erie is extremely improbable. Final grade elevations in the immediate plant area vary from 617 to 620 feet (USGS). This is about 45 feet above the maximum monthly mean lake level of 575.4 feet (USGS) (Reference 61). Surge flooding is described in <Section 2.4.5>. Runup occurring coincidentally with the probable maximum setup would extend to about Elevation 607.9 feet on the bluff at the lake shore (Reference 65). This runup would still be about 12 feet below the 620 foot (USGS) plant grade elevation.

The controlling flood hazard is the Local Intense Precipitation (LIP) event. LIP water levels impacting the site are a result of the site’s drainage capacity during significant precipitation events as discussed in <Section 2.4.2.3>. Protection of safety-related facilities, systems, and equipment is discussed in <Section 2.4.2.3>, <Section 2.4.10> and <Section 2.4.14>.

External flood hazards and protection for PNPP are developed using the guidance of <Regulatory Guide 1.59> and <Regulatory Guide 1.102>. Flood hazards are determined using the guidance of (Reference 81), which is referenced by Appendix A of <Regulatory Guide 1.59>. For all off-site flood hazards, PNPP complies with Regulatory Position C.1 of <Regulatory Guide 1.59>. For these hazards, PNPP is fully passively protected by topography and exterior barriers are defined in <Regulatory Guide 1.102>. For the LIP hazard, PNPP employs an approach more closely aligned with Regulatory Position C.2 of <Regulatory Guide 1.59> with protection provided via a combination of permanent and temporary incorporated barriers as defined in <Regulatory Guide 1.102>. PNPP does not comply with Regulatory Position C.2.d of <Regulatory Guide 1.59> in that structures required to reach and maintain cold shutdown are not fully passively protected. These structures, like all safety-related structures and nonsafety-related structures that communicate with safety-related structures, are passively protected up to and including the effects of the Standard Project Storm (SPS) as derived and evaluated in (Reference 72) and (Reference 82), respectively. For hazards exceeding the SPS up to and including the Probable Maximum Precipitation event of the LIP domain which is shown

in <Figure 2.4-82>, protection features are deployed via operator action as discussed in <Section 2.4.10> and <Section 2.4.14>.

2.4.2.3 Effects of Local Intense Precipitation

The Local Intense Precipitation domain comprises the site property bordered to the north by Lake Erie, to the east by the Diversion Stream and to the west and south by the Major Stream. The LIP domain is shown by <Figure 2.4-82>. All safety related buildings are located within the LIP domain. The LIP event is based on a front-loaded Probable Maximum Precipitation event as described in <Section 2.4.3.1>.

Surface drainage is provided by a combination of overland drainage features and the site's storm drain system which is shown on <Figure 2.4-3>. Site topography provides overland runoff to several notable drainage features including the Major Stream, Remnant Minor Stream via the ESW Swale, the Barge Slip, Northwest Impoundment, and Lake Erie. Additionally, three separate subsurface storm drainage systems (East, West and South System) assist in surface drainage. Site topography and the storm drain system are designed to reduce the effects of the LIP flooding event. The controlling flood levels impacting the site are a result of the limitations of the site's surface drainage capabilities during the LIP event (Reference 66). The site surface drainage ability is largely influenced by site topography and ground cover type; location and dimensions of above grade objects, structures and buildings; roof drainage and storage; surface drainage features; and subsurface storm drain systems.

Due to drainage limitations, floodwater depths have the potential to impact safety-related structures and non-safety related structures that communicate with safety related structures. The flood protection requirements needed to protect safety-related facilities, systems, and equipment from this event are identified in <Section 2.4.10>.

Safety and non-safety building roofs are drained by a combination of roof pitch, roof drains, parapet openings and scuppers. The drainage

on the roofs is controlled to ensure that water depths developed during the LIP rainfall do not exceed the structural capacity of the roof (Reference 67). In case of complete blockage of roof downspouts on safety related buildings, building scuppers and parapet openings have been sized to ensure roofs will not exceed the maximum allowable loading limits.

Analysis of the LIP event is based on a front-loaded precipitation event distributed into hourly intensities as described in <Section 2.4.3.1>. The peak rainfall intensity of 13.19 inches per hour will occur during the first hour and total precipitation of 34.72 inches occurs over 48 hours (Reference 68). Analysis of the LIP event assumes a concurrent discharge of the ESW Swale as well as a failure of the Diversion Stream berm. The LIP analysis also assumes maximized boundary conditions for the Major Stream, Diversion Stream and Lake Erie. These conservative assumptions and inputs ensure the LIP analytical results are bounding of any potential event combination. Therefore, no combined events need to be postulated for PNPP.

Analysis of the LIP event is performed using a two-dimensional transient (unsteady state) volume conservation model (Reference 66). Delineation of the computational domain is based on knowledge of the Remnant Minor Stream, Major Stream, Lake Erie and Diversion Stream watersheds and topographic dividing features. The entire Remnant Minor Stream watershed is included in the LIP domain as it functions as a drainage swale for a large portion of the east side of the plant site. The LIP domain, depicted in <Figure 2.4-82>, is discretized into uniform grid cells for runoff analysis.

The LIP flood model simulates precipitation runoff effects over complex topography through a numerical approximation of the Shallow Water Equations and accounts for the complications in runoff. The model solves a set of partial differential equations (Saint-Venant's Equation) using the second-order Newton-Raphson tangent finite difference method. Appropriate two-dimensional runoff coefficients are specified based on landcover type. Buildings, structures and security

features are appropriately represented in the LIP domain model. The LIP flooding model also incorporates the plant roof and storm drain systems providing discharge analyses for these systems. The conduits modeled in the storm drain system are analyzed using the Manning Equation for gravity flow conditions and transitions to the Darcy-Weisbach Equation when pressurized flow conditions exist. Allowances are included for debris on catch basin openings and within the storm drain piping. LIP modeling is captured in (Reference 66).

The effects of the LIP event on building roofs is evaluated to ensure the structural capacity of buildings is not exceeded (Reference 67). This analysis considers the cascading effects of roof drainage from higher roof elevations to the locations where scuppers, roof drains or parapet openings discharge on lower roof elevations. For safety related buildings, this is determined through the capacity of building scuppers and parapet openings. For those roofs of non-safety buildings that function as an environmental interface for a safety related building, evaluation is provided (Reference 67). A non-safety building is considered an environmental interface to a safety related building if external floodwaters would enter the safety related building if the existence of all or part of the non-safety building was removed. The roof pitch, drains, and scuppers on non-safety roofs are designed to ensure that the effective loading of high-water levels does not exceed the structural capacity of the structure. Similar to the safety related buildings, the non-safety roof analysis includes consideration of the cascading effects of roof drainage from higher roof elevations at the locations where scuppers, roof drains or parapet openings discharge on lower roof elevations.

2.4.2.3.1 Non-LIP Precipitation Events

Additional analysis of the LIP domain is provided for less significant precipitation events including the Standard Project Storm (SPS) and Probable Maximum Winter Precipitation (PMWP) events. The precipitation data for these events are provided in <Section 2.4.3.1>. The runoff analyses for these events utilize the same underlying methodology

described above for the LIP event. The effects of the SPS is determined in (Reference 82). Modeling techniques for the plant storm drain system for LIP effects are retained in the SPS modeling.

The effects of the PMWP with concurrent snowmelt is determined in (Reference 78). This analysis assumes portions of the computational domain are maintained generally clear of snowpack due to normal operational activities. In these areas, a seasonally adjusted one-hour precipitation event of one-year recurrence interval is applied as an antecedent snowfall condition. The remainder of the computational domain is assumed to have a probable maximum snowpack present. The cool-season analysis models a portion of the storm drain system; (Reference 78) omits catch basin inlets which are located under the probable maximum snowpack. Only those catch basins located in areas routinely cleared of snowfall are credited with removing overland runoff. Other modeling techniques for the plant storm drain system for LIP effects are retained in the PMWP modeling.

2.4.3 PROBABLE MAXIMUM FLOOD (PMF) ON STREAMS AND RIVERS

In accordance with Appendix A of <Regulatory Guide 1.59> and the applicable sections of (Reference 55), the procedures and data of the U.S. Army Corps of Engineers (Reference 4) and the U.S. Bureau of Reclamation (Reference 5) were used to calculate effective Probable

Maximum Floods for the two streams mentioned in the preceding sections. The calculated PMF for the Major Stream was found to be 32,760 cfs (Reference 70), and for the Diversion Stream 4,330 cfs (Reference 71), at their outfalls into Lake Erie. <Figure 2.4-4> and <Figure 2.4-5> show the hydrographs generated for the PMF of both streams. <Figure 2.4-4A> and <Figure 2.4-5A> show the inundation profiles of the Major Stream and Diversion Stream, respectively.

2.4.3.1 Probable Maximum Precipitation

The Probable Maximum Precipitation (PMP) for areas from 10 to 1,000 square miles east of the 105th Meridian has been derived by the Corps of Engineers and is presented in Hydrometeorological Report 33 (Reference 4). For the Perry site, the PMP for the 6-hour, 12-hour, 24-hour and 48-hour storms are 26.91, 29.21, 32.43, and 34.73 inches, respectively (Reference 68).

Hydrometeorological Report 33 (Reference 4) does not provide methods to determine precipitation data for time intervals less than six hours. Guidance for obtaining hourly rainfall interval intensities is provided in procedures outlined by the U.S. Bureau of Reclamation (Reference 5). A front-loaded event sequence is used for LIP domain analysis. A mid-loaded event sequence is used for Rivers and Streams analyses as reflected in the following tables (Reference 68). Total precipitation values are the same; only the temporal distribution differs between LIP and Rivers and Streams analyses.

Design Basis Probable Maximum Precipitation - Local Intense Precipitation									
Time	1 HR	2 HR	3 HR	4 HR	5 HR	6 HR	12 HR	24 HR	48 HR
Incremental Rainfall (inches)	13.19	4.03	2.96	2.42	2.16	2.15	2.30	3.22	2.30
Cumulative Rainfall (inches)	13.19	17.22	20.18	22.60	24.76	26.91	29.21	32.43	34.73

Design Basis Probable Maximum Precipitation - Rivers and Streams									
Time	1 HR	2 HR	3 HR	4 HR	5 HR	6 HR	12 HR	24 HR	48 HR
Incremental Rainfall (inches)	2.15	2.42	2.96	13.19	4.03	2.16	2.30	3.22	2.30
Cumulative Rainfall (inches)	2.15	4.57	7.53	20.72	24.75	26.91	29.21	32.43	34.73

Cool-season cumulative precipitation and snowmelt values are determined in (Reference 69) to have peak intensity of 3.63 inches/hour and a 48-hour total of 26.58 inches. Probable Maximum Winter Precipitation data is obtained from (Reference 4) for the months of December, January and February, consistent with the cool-season months described in <Section 2.3.1.2.5>. Cool-season event combinations are obtained from (Reference 55). For PNPP a conservative snowmelt contribution is determined in (Reference 69) by assuming an inexhaustible snowpack. Snowmelt rates are determined using the guidance of (Reference 83). By comparison, the all-season event is bounding of the cool-season event. The all-season event is therefore used as the basis for streams and rivers analyses. Cool-season analysis for the LIP domain is discussed in <Section 2.4.2.3.1>. The cool-season event sequence is provided below.

Probable Maximum Winter Precipitation and Snowmelt									
Time	1 HR	2 HR	3 HR	4 HR	5 HR	6 HR	12 HR	24 HR	48 HR
Incremental Rainfall + Snowmelt (inches)	3.63	1.28	1.01	0.87	0.81	0.79	0.61	0.43	0.39
Cumulative Rainfall + Snowmelt (inches)	3.63	4.91	5.92	6.79	7.59	8.39	12.02	17.22	26.58

Standard Project Storm precipitation values are determined in (Reference 72) to have peak intensity of 3.747 inches/hour and a 24-hour total of 13.00 inches. Derivation of the Standard Project Storm is performed using the guidance of (Reference 6). By comparison, the

all-season event is bounding of the Standard Project Storm event. The all-season event is therefore used as the basis for streams and rivers analyses. Standard Project Storm analysis for the LIP domain is discussed in <Section 2.4.2.3.1>. The Standard Project Storm event sequence is provided below.

Standard Project Storm									
Time	6 HR	12 HR	13 HR	14 HR	15 HR	16 HR	17 HR	18 HR	24 HR
Incremental Rainfall (inches)	0.560	1.600	0.986	1.183	1.479	3.747	1.380	1.085	0.980
Cumulative Rainfall (inches)	0.560	2.160	3.146	4.329	5.808	9.555	10.935	12.020	13.000

2.4.3.2 Precipitation Losses

To increase the conservatism of the results, 100 percent antecedent saturation of the basins and no losses were assumed for any of the precipitation hazards described herein.

2.4.3.3 Runoff and Stream Course Models

Flood elevations for the Major and Diversion Streams are determined for the PMF peak runoff using the standard step method (steady state modeling) as described in (Reference 48) and in accordance with (Reference 55), (Reference 70), and (Reference 71). PMF peak runoffs are determined by application of triangular synthetic unit hydrographs, as described in (Reference 5) using the all-season precipitation event provided in <Section 2.4.3.1>. For conservatism the PMP was assumed to fall on fully saturated terrain, with 100 percent runoff. The size of the streams' drainage basins is described in <Section 2.4.1.2>. Coefficients used in the stream course models are described in <Section 2.4.3.5>. The PMF flow hydrographs for the Major Stream and the Diversion Stream are presented in <Figure 2.4-4> and <Figure 2.4-5>, respectively.

No stream course modeling is provided for the effects of the Standard Project Storm or cool-season events as discussed in <Section 2.4.3.1> since the all-season event is considered bounding with fully passive protection provided for Major and Diversion streams’ flood hazards.

The Remnant Minor Stream is not evaluated as a traditional stream or river. Instead, the Remnant Minor Stream is incorporated into the two-dimensional LIP domain model since the Remnant Minor Stream serves as a topographic drainage swale and does not function as a traditional stream. This allows the channel to respond dynamically (unsteady state analysis) as part of the LIP event.

2.4.3.4 Probable Maximum Flood Flow

The probable maximum flood flow hydrograph for the Major and Diversion Streams at their outfalls into Lake Erie are presented in <Figure 2.4-4> and <Figure 2.4-5>, respectively. These hydrographs correspond to the incidence of the PMP. Other factors such as surges or upstream dam

failures will not affect the streams' PMF. The PMF flow for both streams were calculated with the rain data given in <Section 2.4.3.1> and the procedures outlined in Pages 73 through 82 of (Reference 5).

2.4.3.5 Water Level Determinations

<Figure 2.4-6> and <Figure 2.4-8> show the bed elevation and PMF surface profiles for the Major and Diversion Streams, respectively. Surface profiles were determined according to the standard step method utilizing Manning's formula. The Manning coefficient of friction was assumed to be equal to 0.100 for the portions of the existing stream bed and overbanks, and equal to 0.024 for the trapezoidal sections used in the engineered streamcourse. The profiles and cross sections are shown on <Figure 2.4-8a>, <Figure 2.4-10>, and <Figure 2.4-11>. The steady state analysis computes water surface profiles from one cross section to the next by solving the Energy equation with an iterative procedure called the Standard Step Method. The Standard Step Method begins computations with the downstream boundary condition for subcritical flow. The unknown water surface elevation at the upstream cross section is first assumed by applying the downstream water depth. The corresponding conveyance and velocity head are determined for the applied water surface elevation. The friction slope between the two cross sections is then computed and used to solve the energy head loss between the two cross sections. The Energy equation is then solved to determine the water surface elevation at the upstream cross section. The solved water surface elevation is compared with the initial assumed elevation and iterations are repeated until numerical values converge. PMF surface profiles are provided in <Figure 2.4-6> for the Major Stream (Reference 70) and <Figure 2.4-8> for the Diversion Stream (Reference 71).

Based on the large flow areas at each structure, the Major Stream water surface profiles during the PMF were calculated under the assumption that the plant access road, rail line bridge, and the sediment control dams placed across the streams remained intact/open during the event. The Diversion Stream outfall culvert and Major Stream's Contractor

Access Road culvert were modeled as blocked during PMF conditions. Furthermore, a discharge coefficient of 3.00 was used for calculating the depth of the PMF passing over the broad crested weirs formed by inline structures.

The maximum PMF water surface elevation (WSE) along the Major Stream occurs upstream of the plant access road and was determined to be approximately 633.1 ft NGVD29 Perry Local Datum (PLD). The WSE at the Plant Access Road was determined to be 633 ft PLD (Reference 70). The maximum PMF WSE along the Diversion Stream is approximately 627 ft PLD at the upstream end of the modeled stream course (Reference 71). For both the Major Stream and Diversion Stream, topography plant-side of the stream course ensures the PMF inundation profile does not reach plant structures. This includes the natural ridge along the Major Stream and the earthen berm along the plant side bank of the Diversion Stream. The minimum freeboard for either stream exceeds one foot (Reference 70, Reference 71). The PMF flow elevation exceeds the bottom of the railroad bridge; however, analysis shows the bridge remains intact during PMF conditions (Reference 73).

As no recorded data exists, no correlation is possible between the results obtained and floods on record. For added conservatism, the flow for each reach of both streams was considered to be equal to the corresponding PMF flow at the outfall to Lake Erie.

2.4.3.6 Coincident Wind Wave Activity

Wind wave activity is of no concern with the flood conditions in these small streams as previously discussed.

2.4.4 POTENTIAL DAM FAILURES, SEISMICALLY INDUCED

Presently there are no impoundments upstream of the plant, and since the drainage basins of the two streams passing through the site are small and the terrain is quite flat, it is unlikely there will be any

impoundments in the future. Therefore, seismically-induced dam failure is not included as a design condition.

2.4.4.1 Dam Failure Permutations

This section is not applicable to PNPP.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

This section is not applicable to PNPP.

2.4.4.3 Water Level at Plant Site

This section is not applicable to PNPP.

center of the lake reach 102 mph. The temporal variation in setup height over the remaining storm period is plotted in the hydrograph shown in <Figure 2.4-28>. The spatial profile of the lake surface at maximum deflection is plotted in <Figure 2.4-29>.

It should be noted that these values do not reflect corrections for pressure differences or precipitation. With an antecedent water level of 575.4 feet (USGS), a precipitation value of 0.5 feet, a pressure correction of 0.3 feet, and a wind setup of 4.30 feet, the total maximum stillwater surface level at the plant site was computed to be 580.5 feet (USGS) (Reference 61).

Wind velocities for the PMS were generated at three hour intervals for ten stations around the periphery of Lake Erie using the techniques discussed in <Section 2.4.5.1>. <Figure 2.4-26> shows the locations of the ten stations. <Figure 2.4-30> and <Figure 2.4-31> show variations of wind speed and direction for the two stations (20,1) and (40,1) adjacent to the site and the two stations (20,21) and (40,21) north of the site, respectively. <Table 2.4-1> indicates the site setup winds at these stations.

The wind speed for the stations adjacent to the site averages above 70 mph for a period of 18 hours. During the same period the wind speeds for the stations across the lake from the site average 80 mph or above with a maximum of 110 mph being reached at the center of the lake.

2.4.5.3 Wave Action

The wind field used to determine the probable maximum setup was also used to find the concurrent wave action. As discussed in <Section 2.4.5.1>, this wind field was defined at ten stations located around the lake to give complete meteorological coverage of Lake Erie. A study of the critical fetch direction showed that the fetch lengths were approximately the same for winds directed toward the site from

The setup of the lake level causes the pumps to operate at a higher than normal flow rate since the static head is reduced. With these wave induced variations in the pump chamber water level, no problem exists in meeting the flow requirement described in <Section 9.2.1>.

2.4.5.5 Protective Structures

2.4.5.5.1 Shoreline Recession

2.4.5.5.1.1 Summary

Since plant grade is approximately 45 feet above the normal lake level and there are no safety-related structures within 380 feet of the lake shoreline (toe of bluff), damage to the shoreline bluff by an individual storm would not affect the operation or the safety of the PNPP. Data presented herein shows that the range of bluff recession in the vicinity of the emergency service water pumphouse has been less than 2 feet per year since 1937.

Erosion of the shoreline bluff is caused by groundwater seepage and direct runoff on the upper portions of the bluff, and by wave attack and runup at the lower portions. To monitor the combined effect of shoreline recession and bluff erosion, a semiannual (Spring and Fall) survey is being made at six profile locations established at regular intervals along the shoreline. This survey will continue on a semiannual basis from 1984 through 1989, at which time it will be continued on an annual basis (spring) for the life of the plant (Reference 74). <Figure 2.4-32> shows the exact locations of the survey lines. The sections that follow describe the bluff recession process.

2.4.5.5.1.2 Introduction

This section presents the findings of shoreline erosion studies and investigations for a 6,000 foot reach of the Lake Erie shoreline north

locally as much as 1,500 feet offshore. The lake bottom north of the site boundary is irregular but generally slopes at about 2.5 feet per 100 feet for the first 300 feet offshore, decreasing to about 5 feet in 1,000 feet for the next 4,000 feet or more.

2.4.5.5.1.4 Nature of Bluff Erosion

The principal factors affecting shoreline recession are variations in Lake Erie levels, wind (storm waves), water seepage and frost action. The composition and degree of compaction of the bluff materials are limiting factors in the rate of recession. Widespread slumping of the upper half of the bluff is the most prevalent bluff feature in the vicinity of the site. Slumping within the lacustrine and upper till deposits is caused mainly by groundwater seepage forces and frost action. Undercutting and removal of slump material by wave action complete the cycle of bluff recession.

2.4.5.5.1.5 Wind, Littoral Drift and Beaches

Local storms are the main cause of significant wave action. The shoreline at the site is subjected to waves from the southwest through the north to the northwest. Winds from the southwesterly direction set up the prevailing littoral current from the west to the east.

Generally, the west to east direction of littoral drift results in accretion of sand at the west side and the depletion of material to the east side of structures projecting from the shore. The nearest off-site structure is located near the northwest corner of the site boundary on the former Neff Perkins property. This structure, a sheet piled water conveyance channel, projects into Lake Erie about 135 feet. With the exception of the small beach northwest of the site, beaches along the shore in the study area are narrow, short in length and frequently transitory or submerged.

2.4.5.5.1.6 Lake Levels

Shoreline erosion is noticeably influenced by lake levels and the related storm waves. High levels allow the waves to directly contact the toe of bluff while waves from low lake levels are dissipated by wider beaches. Lake levels vary with climatic conditions which affect evaporation and inflows. Minimum lake levels usually occur in February and maximum levels occur in mid-summer. The maximum seasonal fluctuation is nearly three feet, but long term cycles of fluctuation, resulting from wet and dry periods are as much as eight feet. Predictions of future lake levels are difficult to make. However, future long term cycles of fluctuation are not expected to differ significantly from those presented in <Figure 2.4-33>.

<Figure 2.4-34> presents monthly recorded mean lake levels, from 1904 to 1979, which show short and long term trends. Long term annual fluctuations are shown in <Figure 2.4-33>. The 1973 mean level of Lake Erie was about five feet above the Lake Erie Low Water Datum (LWD) established at 570.5 feet (USGS) above mean tide at New York City; that mean level was at the peak of a high lake level cycle (Reference 61). Reducing lake levels to a common datum for the various years studied was not possible because dates (day and month) were not always given on the property survey information.

2.4.5.5.1.7 Effects of Ice

Almost every year, ice forms along the shore of Lake Erie. In winters with sustained periods of sub-zero temperatures, the entire lake freezes over; during the winter months, wave action upon the beach and the shoreline bluff is minimal or non-existent due to the buildup of ice along the shoreline. However, freezing and thawing of the groundwater seepage produces a detrimental effect on the bluff face that contributes to the rate of bluff erosion.

The total top of bluff change indicated by <Figure 2.4-35> for a 2,000 foot central reach of shoreline northwest of the plant ranges from 20 to 85 feet (<1 to 2 ft/yr) from 1937 to 1972 (35 years). For the same period, total top of bluff recession of reaches to the west and east of the central section ranged from 1 to 3 ft/yr and 4 to 5 ft/yr, respectively. Fluctuations of Lake Erie <Figure 2.4-34> indicate long term mean lake levels from 1934 were on an up cycle, peaking in 1952 and again in 1973 (Reference 61). A correlation of larger changes in bluff erosion during periods of high lake levels could not be evaluated with certainty due to lack of data, but high lake levels undoubtedly have a significant effect. <Table 2.4-2> shows annual recession rates relative to 1972 for each of the twelve erosion monitoring lines <Figure 2.4-32> and is summarized below:

Range (Ft/Yr) of Top of Bluff Recession Relative to 1972

Section	1937	1957	1964
West (J-I-H-G)	1-3	2-6	3-7
Central (F-L-K-A-B)	<1-2	1-5	1-7
East (C-D-E)	4-5	5	4-5

It should be remembered that the bluff erosion is a two-step process. One step is wave action on the lower till which forms the toe of the bluff. As previously discussed, the location of the toe of bluff is the factor that will determine the need for bluff protection. The other step is the random occurrence of localized slumping induced by groundwater seepage and frost action. Although a single slump (at Profile Line C between 10/72 and 4/73) indicated a significant change of 33 feet in the top of bluff location, future movement at the slumped location most likely will not occur again until the toe of bluff has receded to a point where most of the slumped material has been removed. The more recent comparisons (1957 and 1964) show the effects of the random slumping process on the relative top of bluff recession rates.

in the spring and fall of each year. The first complete profiles were taken in November, 1973, and are presented in <Figure 2.4-38>. Readings of the net changes in the bluff, at two year intervals, taken to date are also shown in <Figure 2.4-38>.

<Table 2.4-2> summarizes the recession rates for the profiles shown in <Figure 2.4-38> using profile information through September, 1978. In general, between 1972 and 1978, the top of bluff erosion rate varied (from <1 to 2 ft/yr) within the central shoreline reach. The eastern end of the central reach receded at about 7 ft/yr during this relatively short (six-year) period. This same area receded only 2 ft/yr between 1876 and 1978. The eastern and western tops of bluff receded from 2 to 6 ft/yr and 5 to 9 ft/yr, respectively, between 1972 and 1978. Again, the long term (1876 to 1978) top of bluff recession rates were 2 to 3 ft/yr and 1 to 4 ft/yr for the eastern and western reaches, respectively. In addition, the 1975 aerial survey was examined to assure that excessive erosion of the bluff did not take place between the ground survey profile lines.

<Table 2.4-2> shows that the shorter the time interval investigated, the greater is the recession rate for that time interval. This is due to significant, localized slumps being averaged over a relatively short time period. However, these short term, localized slumps are part of the overall recession cycle; in addition, they are attributable to the sustained high lake levels which peaked in 1975 and, therefore, allowed wave attack to undermine the toe of bluff during the high lake level period. The entire recession process is time and water level dependent; i.e., it occurs in a stop-start sequence, and should be averaged over a long period of time to obtain meaningful results.

2.4.5.5.1.10 Man Made Effects of Erosion

With the exception of the potential future bluff protection described later in this section and the interim protection discussed in <Section 2.4.5.5.9>, no permanent structures exist on the bluff.

Grading and clearing of the site were performed in such a manner that surface runoff was controlled and erosion minimized. The closest building structure (service water pumphouse) to the bluff is approximately 280 feet away. This structure is founded on shale rock well below the overlying glacial deposits. Controlled blasting (instrumentally monitored) was conducted during construction of PNPP. Two interim revetments, described in <Section 2.4.5.5.9>, have been built along the shoreline of the plant property. Protection is provided along the shoreline from the Remnant Minor Stream discharge to the Northwest Storm Impoundment Spillway. Protection is also provided at the outfall of the Diversion Stream. These two features are not continuous.

Changes in littoral drift accretion and depletion along the shoreline forming the north site boundary is not expected to change greatly as a result of possible future industrial development to the east and west of the site boundary. Both the State and Federal agencies play an active role in restricting construction of new structures projecting into Lake Erie that might affect littoral drift patterns or increase the rate of shoreline recession of neighboring shoreline property. Additionally, periodic inspections of unprotected areas ensure excessive erosion does not occur (Reference 74).

2.4.5.5.2 Maximum Allowable Shoreline Recession

<Section 2.5.5> describes the point to which bluff recession could progress without threatening the function of safety class structures. Bluff protection will be installed if the retreat closely approaches this limit.

2.4.5.5.3 Protective Measures

2.4.5.5.3.1 Protective Measures Description

If the shoreline retreat becomes threatening to the safety-related structures, the shoreline will be protected by a suitable permanent

construction that will protect the face and toe of the bluff. Final design and permit applications for the shoreline permanent protection construction will be initiated when the lake shoreline (toe of bluff)

has receded to a point 250 feet away from the closest safety class structure (emergency service water pumphouse). However, if the State of Ohio or the Federal Government develops an area-wide plan for shoreline protection that includes the plant, CEI will fully cooperate in implementing the plan on its property. The protective construction (Reference 29) (Reference 39) considers grading of the shoreline bluff and construction of a permanent protective revetment. Interim protective measures are described in <Section 2.4.5.5.9>.

a. Bluff Grading

Prior to installation of the shoreline protective revetment, the bluff will be graded to the dimensions shown in <Figure 2.4-39>. The bluff grading will be as follows:

1. Prior to the placement of the protective revetment, the bluff will be graded to a 2:1 slope between Elevation 555 feet (USGS) and Elevation 605 feet (USGS).

2. After the 2:1 grading is finished, a 10-foot wide berm will be cut at Elevation 605 feet (USGS).

3. After the berm is finished, further grading of the bluff to a 3:1 slope is required between Elevation 605 feet (USGS) and the top of the bluff.

b. Protective Revetment

The maximum wave runup, corresponding to the 9.7-foot high maximum wave breaker height (Reference 31) breaking on the rough 2:1 slope of the protective revetment, is found in (Reference 22) to be 16.4 ft. The wave runup would therefore reach Elevation 596.9 feet (USGS) when the lake is at Elevation 580.5 feet (USGS) i.e., maximum stillwater level PMS (Reference 61). However, it should be pointed out

that the freeboard between the top of the protective revetment, located at Elevation 605.0 feet (USGS), and the maximum wave breaker height runup is 8.1 feet. Therefore, it is not expected that the wave runup produced by the PMS will ever overtop the crest of the revetment. The shoreline protective revetment will protect a section of the lake shoreline approximately 2,000 feet long, comprised of the distance between the entrance to the barge unloading area and the outlet of the Remnant Minor Stream. Extension to the outfall of the Diversion Stream will also be evaluated.

The protective revetment as shown in <Figure 2.4-39> will offer sufficient protection to the emergency service water pumphouse. The shoreline protective revetment will be a rubble mound structure composed of the following layers of material:

1. The first layer is a permeable plastic fabric sheet, which is placed on the lake bed and over the area cut to the 2:1 slope.

2. On top of the permeable plastic fabric a 2-foot thick gravel filter blanket will be placed, as specified and shown in <Figure 2.4-39>.

3. The filter will be covered with two layers of 1,000 to 2,000 pound quarry stones fitted into the gravel material.

4. The two layers of large quarry stones will be surmounted by two layers of heavy armor stones having a weight of between 6 and 10 tons. The heavy armor stones will not only be randomly placed over the sloped surface, but will become an integral part of the protective toe of the rubble mound protection. The toe protection will extend 47 feet into the lake and will be founded on weathered bedrock at approximate Elevation 555.0 feet.

environmental impact report. This is required for any structure that will be built in Lake Erie below Elevation 572.8 feet IGLD (574.7 feet USGS).

b. Shore erosion permit

This permit will be issued by the Ohio Department of Natural Resources and requires detailed drawings of the proposed shoreline protection structures.

2.4.5.5.4 (Deleted)

2.4.5.5.5 (Deleted)

2.4.5.5.6 Barge Slip

A nonsafety-related barge slip was constructed northwest of the plant along the southern shoreline of Lake Erie. It is located in the general area incised by the Minor Stream where it originally entered the Lake (prior to plant construction).

An approach channel (averaging about seven feet in depth) was dredged from the lake to the barge slip. Lake dredging was performed to initially open the channel and then as required to maintain the opening prior to barge deliveries. Dredged material was disposed of on the plant site. The barge slip was constructed of steel sheet piling with tie-backs. The lake end of the barge slip was constructed about 30 feet south of the 1972 shoreline, with sheet pile wings being constructed toward the shoreline.

To protect the side slopes and shoreline at the lake end of the barge slip, a rubble mound protective revetment structure was installed along the shoreline, extending 50 and 80 feet, respectively, east and west of

2.4.5.5.7 Northwest Sediment Control Dam

A nonsafety-related sediment control earthen dam and associated concrete spillway were constructed at the northwest corner of the plant site. The toe of the spillway is approximately 90 feet south of the 1975 shoreline. The channel bottom between the spillway toe and the shoreline is protected by 2-foot-6-inch thick dumped riprap placed over a 1-foot-3-inch thick gravel filter.

2.4.5.5.8 Remnant Minor Stream and Diversion Stream Outlets

The Minor Stream, which originally entered the lake at the present barge slip location, was diverted east of the plant site prior to plant construction. The channel outfall to the lake was constructed using 96 inch diameter corrugated metal pipe installed over a layer of 500 to 8,000 pound dumped stone riprap. The pipe was terminated at the sheet piling protection installed along the lake shoreline in front of the plant site. In 2015, the Diversion Stream was installed to further route streamflow east of the plant, increasing the conveyance capacity of the streamcourse (Reference 75). The Diversion Stream intercepts the Minor Stream and routes streamflow through a trapezoidal engineered channel, discharging to Lake Erie. Following installation of the Diversion Stream, the Remnant Minor Stream (consisting of the construction-era engineered channel) functions only as a swale path for the Local Intense Precipitation domain. The outfall of the Diversion Stream is provided with a 48-inch pipe discharging to an armor stone revetment along the shoreline, discussed in <Section 2.4.5.5.9>. The outfall is sized for the streamflow resulting from a 100-year recurrence interval storm; flowrates in excess of this will overtop the outfall and flow directly to Lake Erie.

2.4.5.5.9 Interim Shore Protection

In the early spring of 1975, significant shore erosion was observed at three localized areas of the plant site in the vicinity of site construction operations. These accelerated erosion rates were directly

attributed to the sustained high lake levels which peaked in 1975 and which allowed wave attack to undermine the toe of the bluff during the high lake level period. In 1983, a rock protected sheet pile breakwall was erected. This sheet pile breakwall protects the section of the lake shoreline approximately 2,200 feet long, composed of the distance between the entrance to the barge unloading area and the outlet of the Remnant Minor Stream as shown in <Figure 2.4-39A>. In 1992, the temporary sunken barges were removed and the interim revetment was extended 1300 feet west to the northwest storm impoundment spillway as shown in <Figure 2.4-39A>. The design of the extension was essentially the same as the original 2200 foot revetment.

The top of the bluff should stabilize at approximately 300 feet from the emergency service water pumphouse. Therefore, this breakwall will provide significant protection to the shoreline and to the emergency service water pumphouse.

In 2015, as part of the Diversion Stream installation, an armor stone revetment was installed at the outfall of the Diversion Stream. The armor stone revetment protects approximately 100 feet of shoreline and exceeds the requirements of Technical Report No. 4 (Reference 22) (Reference 76). The revetment consists of a toe stone layer of four to five-ton stone and an armor stone layer of two to four-ton stone. The revetment begins below the lake surface and extends up the bluff to areas impacted by wave runup; the armor stone extends up to the base of the Diversion Stream culvert, approximate Elevation 589.6 ft PLD (Reference 77). The Diversion Stream revetment is depicted in <Figure 2.4-39B>.

2.4.6 PROBABLE MAXIMUM TSUNAMI FLOODING

Since the site is located on Lake Erie, an inland lake, tsunami occurrence is not applicable. The Lake Survey Center (NOAA) has found no evidence of earthquake related seiches.

2.4.7 ICE EFFECTS

2.4.7.1 Regional Ice Formation History

Lake Erie, the shallowest of the Great Lakes, is also the most thermally variable. It reacts rapidly to seasonal temperature changes and can build an appreciable ice cover in a comparatively short time (Reference 33).

As winter progresses, ice on Lake Erie spreads from west to east. In mid-December, ice begins to form in the extreme western portion of Lake Erie. It spreads eastward, partially aided by a prevailing west wind, until the beginning of March when the eastern end of the lake is

completely ice covered. At this time, the probability of ice cover in the site area exceeds 76 percent (Reference 32). The ice disappears from the western and central portions of Lake Erie by the end of March and by mid-April only a small area near Buffalo, New York, is still ice covered (Reference 33). Available information indicates that the critical period for ice development in the site area is from mid-February through mid-March with ice block formation prevalent during the spring ice breakup.

The intake structures are protected against the impact of ice islands by 10 vertical reinforced concrete caissons, each 6 or 7 feet in diameter, placed in a 70-foot diameter circle around each of the submerged intake structures. The spacing between the caissons is such that the intake water approach velocity will not be increased.

The three foot diameter discharge nozzle is encased in a 17-foot diameter reinforced concrete caisson for protection against the ice loads produced by the impact of a floating ice island crushing against the structure. <Figure 3.8-67> and <Figure 3.8-68> show details of caissons for the intake and discharge structures.

2.4.7.4 Ice Flooding

Ice flooding cannot occur because of the high bluffs between the buildings and the lake. Also, safety-related onshore buildings are set back from the top of the 45-foot high bluff to preclude ice forces being a problem.

Ice-induced flooding from the adjacent streams is not a concern. The base flow of the streams is 5 cfs or less per <Section 2.4.1.2> which precludes formation of appreciable ice blockage in the stream channels. The probable maximum winter precipitation (PMWP) rainfall is bounded by the all-season rainfall as presented in <Section 2.4.3.1>. The effects of the PMWP on the Local Intense Precipitation (LIP) domain is discussed in <Section 2.4.2.3.1>.

2.4.8 COOLING WATER TUNNELS AND OFFSHORE STRUCTURES

2.4.8.1 Cooling Water Tunnels

Service and cooling water for PNPP will be obtained from Lake Erie at approximately 2,600 feet offshore and carried to the plant site through a 10-foot diameter intake tunnel located in the underlying bedrock. The water is then returned to the lake through a similar discharge tunnel of shorter length. Short tunnels of the same diameter are used

Information Only

near the shore facilities to tap the cooling water tunnels for the emergency service water system. Tunnel locations are shown on <Figure 3.8-65> and <Figure 3.8-66>. A discussion of the tunnel function is given in <Section 2.4.11.6> and <Section 9.2.1>.

A three-foot diameter discharge was chosen as the best design for environmental protection. Details of the analysis are given in (Reference 36).

The design of the diffuser discharge structure is shown in <Figure 3.8-67>.

2.4.9 CHANNEL DIVERSIONS

This section is not applicable to PNPP since no cooling water channels exist from which flow could be diverted.

2.4.10 FLOODING PROTECTION REQUIREMENTS

Four prospective sources of flooding exist at the Perry site: Lake Erie to the north of the plant, flooding by the Major Stream which borders the site to the west and south, flooding from the Diversion Stream which borders the site to the east, and flooding within the local intense precipitation domain due to direct precipitation events.

All flooding sources are mitigated with unique flooding protection to ensure that floodwaters do not adversely impact safety related structures, systems and components. Protection is provided in the form of Exterior Barriers and Incorporated Barriers as defined in <Regulatory Guide 1.102>.

2.4.10.1 <u>Lake Erie Flood Hazards</u>

<Section 2.4.5> and <Section 2.4.7> discuss the possibility of flooding caused by the probable maximum surge including wave runup and ice conditions in the lake. The plant grade is sufficiently above lake hazards that plant land structures are protected from floodwaters and related forces. The peak water surface elevation (WSE), including wave runup is 607.9 ft PLD as provided in <Section 2.4.2.2>. This is well below the nominal ground-level floor elevation of plant structures of 620.5 ft PLD. As such, flooding from Lake Erie is not a concern.

On the intake side of the cooling water systems, all safety-related pumps and equipment will be located above Elevation 586′-6″ (USGS) in the emergency service water pumphouse. This elevation allows approximately three feet of freeboard over the simultaneous occurrence of the probable maximum setup, the maximum monthly mean lake level of record, and the associated oscillation of the pump chamber water level due to wave action over either the main or alternate submerged offshore intake structure.

No emergency procedures are required for Lake Erie flood hazards.

2.4.10.2 Major Stream Flood Hazards

The Major Stream probable maximum flood (PMF) inundation area is shown in <Figure 2.4-4a>. As shown in this figure, the streamflow resulting from a probable maximum precipitation (PMP) event is confined to the Major Stream's drainage basin and does not encroach on plant safety-related structures or nonsafety-related structures that communicate with safety-related structures. The Major Stream is an engineered feature external to the plant area designed to protect the site from inundation and functions as an Exterior Barrier as described in <Regulatory Guide 1.102>. Plant Structures, Systems and Components are passively protected from the Major Stream flood hazards for all events up to and including the PMP by the channel capacity and topographic features segregating the Major Stream's drainage basin from the local intense precipitation (LIP) domain.

The PMF discussed herein is based on the precipitation event provided in <Section 2.4.3.1> which bounds the standard project storm (SPS) and probable maximum winter precipitation (PMWP) events provided in <Section 2.4.3.1>.

No emergency procedures are required for Major Stream flood hazards.

2.4.10.3 Diversion Stream Flood Hazards

The Diversion Stream probable maximum flood (PMF) inundation area is shown in <Figure 2.4-5a>. As shown in this figure, the streamflow resulting from a probable maximum precipitation (PMP) event is confined to the Diversion Stream's drainage basin and does not encroach on plant safety-related structures or nonsafety-related that communicate with safety-related structures. The Diversion Stream is provided with an earthen embankment ("berm") which functions as an extension of the channel's west bank. The top of the Diversion Stream berm is above the peak water surface elevation (WSE) during the Diversion Stream PMF. The Diversion Stream and berm are engineered features external to the plant area designed to protect the site from inundation and functions as an Exterior Barrier as described in <Regulatory Guide 1.102>. Plant Structures, Systems and Components are passively protected from the Diversion Stream flood hazards for all events up to and including the PMP by the channel capacity and berm which segregates the Diversion Stream's drainage basin from the local intense precipitation (LIP) domain.

A failure of the Diversion Stream berm is postulated within the analysis for the LIP event (Reference 66). Guidance for determining the breach profile and size, as well as resulting breach flow rate, is determined using the guidance of the U.S. Army Corps of Engineers (Reference 84) (Reference 85). As such, any failure of the Diversion Stream berm is inherently bounded by the protection discussed for the LIP event.

The PMF discussed herein is based on the precipitation event provided in <Section 2.4.3.1> which bounds the standard project storm (SPS) and probable maximum winter precipitation (PMWP) events provided in <Section 2.4.3.1>.

No emergency procedures are required for Diversion Stream flood hazards.

2.4.10.4 Local Intense Precipitation Domain Flood Hazards

As discussed in <Section 2.4.2.3>, effects from the Local Intense Precipitation (LIP) result in the controlling flood event for the site. The LIP event results in water levels which exceed door thresholds and wall penetrations. For the LIP hazard, flood protection is provided in the form of incorporated barriers as defined in <Regulatory Guide 1.102>. Protection is provided for safety-related buildings and nonsafety-related buildings which communicate with safety-related buildings. These incorporated barriers provide protection for safety-related structures, systems, and components.

For the LIP hazard, both permanent and temporary (removable) incorporated barriers are utilized. Permanent incorporated barriers consist of safety-related building exteriors, steel closure plates, and concrete and aluminum flood walls. With the exception of the safety related building exteriors and closure plates at the Emergency Diesel Generator (EDG) overflow flame arrestors, the permanent incorporated barriers are nonsafety-related. Typical depictions of the concrete and aluminum flood walls are shown in <Figure 2.4-78> and <Figure 2.4-79>, respectively. Buildings protected by these features are depicted in <Figure 2.4-77>.

Temporary incorporated barriers are provided in the form of removable flood panels (flood stop logs) which are also nonsafety-related. These barriers are utilized at plant doorways which create openings in the otherwise continuous protection provided by the walls discussed above. These removable barriers, certified to the requirements of ANSI Standard ASNI/FM-2510, are generally stored in a central location and deployed in accordance with plant procedures. In some cases, where there is minimal impact on normal plant operation and no impact on personnel egress, these temporary incorporated barriers may be pre-deployed. As discussed in detail in <Section 2.4.14>, operator action is initiated by receipt of meteorological forecast, discussed in <Section 2.4.10.5>, of forthcoming significant precipitation. Typical depictions of the concrete and aluminum flood walls are show in

<Figure 2.4-78> and <Figure 2.4-79>, respectively. Typical examples of the removable incorporated barriers are depicted in <Figure 2.4-80> and <Figure 2.4-81>.

All incorporated barriers are designed for the entire range of flood forces (Reference 79). This includes the hydrodynamic forces resulting from the flooded condition (including fluid velocities) and the forces resulting from flood-born missile impact.

Deployment of flood protection features for the LIP hazard is incorporated into plant off-normal instructions.

Lesser precipitation hazards for the LIP domain, up to and including the standard project storm (SPS) and probable maximum winter precipitation event (with concurrent snowmelt), are mitigated using permanent passive protection which requires no operator action. These features include the walls discussed above, closure plates, ramps and raised thresholds. No emergency procedures for barrier deployment are required for SPS or PMWP effects.

2.4.10.5 Meteorological Forecast Warning

Using the guidance of (Reference 88), PNPP utilizes a two-tier meteorological alert to initiate deployment of the temporary barriers discussed in <Section 2.4.10.4> (Reference 89). The first-tier warning is set to 2.1 inches of precipitation in a 24-hour period. This event corresponds to a one-year recurrence interval precipitation event. The second-tier warning is set to 1.9 inches of precipitation in a one-hour period. This event corresponds to an approximately 18-year recurrence interval event. Compared to the Standard Project Storm (SPS) event for which fully passive protection is provided, which has a peak hourly intensity of 3.747 inches per hour as discussed in <Section 2.4.3.1>, sufficient overlap is provided to account for uncertainties associated with meteorological forecast limitations.

Meteorological forecasting is provided by an external service which is implemented by certified meteorologists. The forecasting employs an ensemble technique using diversified inputs. Input models are obtained from the National Blend of Models (NBM) and Weather Prediction Center (WPC). The NBM provides Quantitative Precipitation Forecasts (QPF) which the WPC provides Probabilistic Quantitative Precipitation Forecasts (PQPF). A 95th percentile confidence value is selected for the ensemble forecasting method per the guidance of (Reference 88). QPF forecasting provides a medium range forecast monitoring five days into the future. PQPF forecasting provides a short-range forecast monitoring 48 hours into the future.

The lower tier warning does not require any specific action by plant personnel. This alert instead serves to initiate heightened awareness thus allowing plant personnel to conservatively take any necessary preemptive actions. The higher alert serves as the initial condition for entry condition for off-normal instructions which includes deployment of temporary flood barriers.

80 mph for approximately 18 hours. For station (20,21), the wind direction varies from 340 to 310 degrees during the maximum wind period. For station (40,21), the wind direction varies from 340 to 320 in this period. The total distance that these winds travel over water is approximately 56 miles.

The computed setdown hydrograph for the site during the period of the PMS is shown in <Figure 2.4-63>. With the 4.03 foot maximum setdown, the minimum water surface elevation at the plant site would be 563.36 feet IGLD (565.26 feet USGS) (Reference 90). This is based on an antecedent water elevation of 567.4 feet (IGLD) (Reference 61), the minimum monthly level of record. The wave action coincident with this setdown was determined by procedures described in Technical Report No. 4 (Reference 22). Using an average 90 mph offshore, over water wind of unlimited duration with a 30 percent reduction to account for land effects in the nearshore zone, the Bretschneider method gives a 2.6 foot significant wave height and a 4.7 second period. Since the alternate intake (normal discharge structure) is closer to shore, the wave heights would be less than at the normal intake structure.

2.4.11.2.2.3 One-Dimensional Model for Setdown Surge

The analysis of the lake setdown surge at the plant site was made using the one-dimensional model described in Technical Report No. 4 (Reference 22), and briefly characterized in <Section 2.4.5.2.2>. The probable maximum setdown was computed with this method using a fetch defined by a line connecting grid points (36,3) (plant site) and (36,21) <Figure 2.4-26>.

<Figure 2.4-63> shows the hydrographs of computed setdown at the site for the PMS setdown wind data. Note that the curve does not reflect any adjustment for lake volume balance, pressure or precipitation. The maximum setdown shown on this curve is 4.03 feet (122.95 cm) (Reference 90).

Volume adjustment was made on the basis of Equation 2.4-1. <Figure 2.4-29> summarizes the results of the volume computations for both the setup and setdown cases at the time of their peak occurrence. The pressure adjustments were made by looking at the PMS pressure data and choosing the maximum pressure difference between adjacent stations across the lake. For both the PMS setup and setdown cases, the maximum pressure difference was found to be 10 mb ≅ 10 cm of water. No precipitation adjustment was included for the setdown case since it would only reduce the setdown and would not be indicative of the most conservative case.

2.4.11.3 Historical Low Water

Two gauges have provided data for the study of the variation in surface elevation of Lake Erie. The Cleveland gauge has a period of record for monthly and annual means dating back to 1860, with instantaneously recorded data since 1904. The gauge at Erie has continuous data from 1960 to the present. A third gauge at Ashtabula, installed in 1954, has been neglected for extended periods since its installation and much of the data recorded by this gauge is incomplete and unreliable due to malfunction of the gauge and its recording mechanism.

<Figure 2.4-2> reveals that the mean lake level has a cyclic variation. At Cleveland, the minimum monthly mean level of record was 567.4 feet IGLD (569.3 feet USGS) recorded in February 1936. The minimum recorded instantaneous level was 565.71 feet IGLD (567.61 feet USGS), which occurred on February 4, 1936. The maximum seiche setdown for each year was computed by subtracting the minimum instantaneous lake level from the mean level for the month in which it occurred. The peak setdown at Cleveland for the period of record was -3.04 feet, occurring December 8, 1927.

At Erie, the minimum monthly mean level was 568.27 feet IGLD (570.17 feet USGS) for December 1964 and the minimum recorded

instantaneous level was 566.00 feet IGLD (567.90 feet USGS), which occurred on March 10, 1964. The peak seiche setdown at Erie for the period of record is -2.90 feet, occurring March 10, 1964.

A frequency study of seiche setdown and minimum monthly mean stages was made for both the Cleveland and Erie gauges. The results obtained are shown in <Figure 2.4-64> and <Figure 2.4-65>, respectively. Log Pearson Type III and Linear Gumbel distributions are presented for each case.

Comparison of the seiche setdown curves in these figures shows that at a particular frequency the seiche at Erie exceeds that at Cleveland. This is felt to be attributable to dissimilar lengths of gauge records and topography in the vicinity of the gauging stations.

2.4.11.4 Future Controls

There are three hydrologic features which influence the site: Lake Erie and the two adjacent streams which border the site. The safety-related structures are located outside the drainage basin of both streams. No developments are foreseen for the streams as the base flow of each is 5 cfs or less. These streams are only used for drainage purposes.

Lake Erie drains into Lake Ontario via the Niagara River and is also connected to Lake Ontario by the Welland Canal. The Niagara River has two major control structures for power generation before reaching Niagara Falls. The upstream east channel has also been channelized for navigational purposes. However, no new developments involving control structures are envisioned.

The Welland Canal is approximately 26.5 miles long and has 8 locks capable of raising ocean-going vessels 326 feet from Lake Ontario to Lake Erie. A more complete description of the canal may be found in

<Section 2.4.11.6>. A section of the canal is being straightened between Miles 23.8 and 15 (Mile 23.8 is approximately 4 miles from the Lake Erie end). The new cross section will have a bottom width of 350 feet compared with 192 feet in the existing channel. The depth of the new channel will be 30 feet, three feet deeper than the present system it will replace.

For reasons outlined in the <Section 2.4.11.6>, it is considered most unlikely that any developments of the canal will influence the surface elevation of Lake Erie.

2.4.11.5 Plant Requirements

As described in <Section 2.4.11.2>, the probable minimum lake level at Perry is Elevation 565.26 feet (USGS). With the inverts of the intake ports at an average elevation of 552.65 feet, inflow of sufficient cooling water during this period is assured. The corresponding water level in the emergency service water pump chamber would be at Elevation 562.09 feet.

During this condition, water for the emergency service water system could also be supplied by the alternate intake system discussed in <Section 2.4.5.4> and <Section 9.2.5>. The crown and invert elevations of the diffuser nozzle of the discharge structure are 555.8 and 552.8 feet (USGS), respectively. The available 12.2-foot submergence over the diffuser nozzle would prevent air entrainment due to the inflow velocity (approximately 15.85 fps through the nozzle).

The minimum water level in the emergency service water pumphouse will occur during the hot standby condition when only the emergency service water systems are operating. At the probable minimum lake level with coincident wave action, the level in the emergency service water pump chamber for this condition will be Elevation 562.09 feet (USGS) (Reference 91). With the invert of the chamber at Elevation 537.0 feet, the 10-foot minimum

<Section 9.2.1>. The water will discharge outside the auxiliary building onto the ground. The slopes and elevations are set so that the flow will be away from the plant, across the road and down a graded swale between the cooling towers to the diverted stream <Figure 2.4-3>. The water will flow down to the stream and discharge into Lake Erie (Reference 86). The topography of the emergency service water alternate discharge area (Remnant Minor Stream) can be seen in <Figure 2.4-3>. Flow through all paths for both units will result in a total flow to the stream of approximately 50,000 gpm initially, which will be reduced to 25,000 gpm in a short period of time. These flow rates represent original plant design (two units). Since then, Unit 2 has been abandoned. As a result, the actual total flow exiting the ESW standpipes will be much less. In addition, the required ESW flow rates for Unit 1 have been analytically decreased to values less than those originally specified (reference <Figure 9.2-1(3)> for the required ESW system flow rates).

The contours of the site will be set to provide a depressed area east of the auxiliary buildings <Figure 2.4-3>. This depressed area will slope away from the plant buildings, over the road and through the swale to the Remnant Minor Stream. Elevations of the depressed areas will be such that no water exiting the plant under this unlikely event will result in flooding of the facility. This is confirmed via analysis captured in (Reference 86).

The main intake structures shown on <Figure 3.8-65> and <Figure 3.8-67> consist of two independent intake heads that are each connected to a six foot diameter shaft. The two vertical shafts convey the water into the 10-foot diameter intake tunnel shown in <Figure 3.8-65> and <Figure 3.8-66>. The intake heads are circular in plan and are covered with a velocity cap placed at a minimum of 13.3 feet below the low water datum (LWD), as shown in <Figure 3.8-67>. Inflow is through vertical openings around the periphery of the intake heads. The discharge

Further, assuming an accidental release of radioactive materials was to occur at the radwaste building and was to assume a travel path through the soils, the estimated transit time to the point of discharge into Lake Erie is 25 years if the groundwater was allowed to return to Elevation 620.0'. The maximum groundwater velocity would likely occur within the lacustrine deposits (upper 25 feet) as the underlying tills are relatively impervious. Coefficient of permeability K calculated from field testing within lacustrine materials ranged between 1.2×10^{-4} cm/sec and 4.2×10^{-7} cm/sec, as discussed in <Section 2.5.4>.

The hydraulic gradient was determined from differences in gravitational groundwater levels between the radwaste building and the future protected bluff slope, and the shortest distance between these points. Groundwater levels measured in proximity to the radwaste building (Boring 1-45) averaged about Elevation 620.0'. Near the edge of the bluff, the static groundwater level range occurs at around Elevation 616.0'.

The Perry site has been graded to approximate Elevation 620.0' PLD. The natural watercourse of the Minor Stream, located between the plant and the bluff, was relocated a trapezoidal engineered channel to the east. The abandoned channel has been filled with compacted soils to Elevation 620.0'. The maximum difference in groundwater levels expected between the plant and the bluff after completion of the site grading is conservatively estimated at 12 feet; however, the readjusted piezometric slope after grading will likely be much less. The minimum groundwater transit distance will occur when the 1979 bluff, which is approximately 1,010 feet from the radwaste building, has receded about 200 feet <Figure 2.4-39>. On the assumption that 200 feet of recession will actually occur during the life of the PNPP, a conservative distance of 800 feet has been used for establishing the groundwater gradient. For

In order to continuously monitor and detect significant amounts of radioactive concentrations discharging from the underdrain system, as a result of the postulated event, radiation monitors are located inside each of the two gravity discharge system manholes at the north end of the Unit 1 heater bay, at locations where the underdrain pump effluent travels through the gravity drain system. Details of the radiation monitor are discussed in <Section 11.5>.

The offsite effects of a hypothetical release, due to a seismic event, of radioactive liquids to the environs is described in <Section 15.7>.

2.4.13.5.4 Maintenance and Testing

Normal routine maintenance will be performed on the mechanical and electrical portions of the system. The manholes and gravity discharge pipes will be inspected annually to ensure that all parts of the system, including the porous piping, are in operating condition. Any blockage in the porous concrete periphery drain pipes will be cleared by mechanical or other suitable means. The following periodic tests will be performed to ensure continuous satisfactory performance of the system.

a. Continuity Test

This test will be performed semiannually for the first five years of operation and annually thereafter. The objective of the test is to verify that water will build up and draw down at the monitoring points, to establish that the underdrain system can reduce the hydrostatic pressure on building foundations to the desired level.

c. Long Term Performance of the Porous Concrete Underdrain System

The long term performance of the porous concrete pressure relief underdrain system is evaluated as follows:

1. Tornados

Tornados will not affect the operation of the underdrain system because it is underground. However, the manhole covers are exposed but are not considered missile proof because a missile entering the top of a manhole would not totally obstruct the flow of water in the manhole.

2. Earthquakes

The system is a Seismic Category I system. The seismic design and analysis for the gravity discharge pipe is discussed in <Section 2.4.13.5.1>. The seismic design and analysis for manholes is discussed in <Section 3.8.4>.

3. PMP (Probable Maximum Precipitation)

The effect of PMP on the underdrain system is negligible. Around the nuclear island buildings, the ground surface will be paved with asphalt, concrete, or backfilled with relatively impervious Class B fill of excavated lower till soils. The rate of infiltration through the Class B fill is calculated to be less than 3 gpm and the effect on the underdrain system is insignificant.

velocity and transport capability of groundwater to erode either Chagrin shale or lower till are negligible. The durability of the shale and its ability to resist subaqueous disintegration and decomposition have been documented by slaking tests and are discussed in <Section 2.5.4>. Lower till is nondispersive as demonstrated by the pinhole and soil conservation service laboratory dispersion test <Section 2.5.4>.

The association of lower till and Chagrin shale properties, coupled with their demonstrated ability to withstand subsurface weathering and erosion during the Recent Epoch, show that no basis exists for assuming a subsurface erosion potential. A conspicuous absence at the Perry site of montmorillonite-type clays, which are characteristically detected in overburden and bedrock undergoing subsurface erosion as well as extensive surface erosion, is considered significant positive evidence in further refutation of this potential.

d. Potential of the Underdrain System to Drain Water From Lake Erie or Along Plant Piping Systems

All piping except the underdrain system gravity flow discharge pipes running from the main plant buildings to the emergency service water pumphouse are designed and installed above Elevation 595.0’ (i.e., approximately 20 feet above the mean monthly high water level for Lake Erie of 575.4’ per <Section 2.4.1.1>). The gravity discharge lines enter the emergency service water pumphouse at invert Elevation 579.0’ which is approximately four feet above mean monthly high lake level. This ensures that no man-made flow path will exist between the pumphouses and the main plant buildings.

The lowest discharge points from any of the 13 deep plant manholes of the porous concrete underdrain system are at manholes No. 8 and 9 which have pipe invert elevations of 582.6’ and 583.5’, respectively <Figure 2.4-69>. Since these gravity discharge pipe inverts are above the maximum lake setup due to the PMS (580.5’), no backflow from the lake can occur through the Emergency Service Water Pumphouse into the porous concrete underdrain system.

The mean monthly high water level for Lake Erie is Elevation 575.4’ (Reference 61) which is seven feet above the groundwater level of 568.0’ maintained in the underdrain system. Because the plant is 800 feet from the lake, the possibility of seepage from the lake into the underdrain was considered.

Geologic mapping of the foundation area was conducted, and open fractures were grouted. None were seepage sources. In this way, all possible drainage paths from the lake into the underdrain systems were sealed.

Underground piping systems, which enter the main plant buildings, were set in bedding materials of granular material which have a higher permeability than the lacustrine soil through which the pipes run, and hence, provide a potential flow path for groundwater into the underdrain system. In order to prevent this flow, at two separate locations special bedding and backfill material were used around all external pipes where they enter the Class A fill at the plant buildings. The special bedding and backfill material is low permeability Class B fill.

The quantity that can flow through the backfill and bedding was included in the calculation of total groundwater flow into the underdrain system. The flow postulated through the pipe bedding material would not increase the calculated inflow to the underdrain system because that calculation assumes a saturated soil.

e. Infiltration Due to Rainfall, Surface Spills or Lawn Sprinkling

A layer of impervious Class B soil was placed over the Class A fill to reduce the amount of seepage possible from infiltration at the surface. Such seepage could result from either rainfall, surface spills, or lawn irrigation.

All underdrain system manholes have gasketed, watertight covers installed at the surface. The covers are normally locked or bolted in a closed position. The 24 outside manholes (two manholes constructed inside buildings) have heavy duty gray cast iron covers. These covers ensure minimal, if any, inleakage occurs into the underdrain system due to rainfall, surface spills or lawn sprinkling.

In the event of the rupture or leakage of an onsite reservoir (e.g., cooling tower basins and industrial waste lagoons), the discharge will generally drain overland away from the plant into neighboring streams and thus to Lake Erie. The area surrounding the plant is also provided with a storm drain system which will collect surface water approaching the plant and assist in drainage of the powerblock area. Because of the impermeable nature of the surface materials, there will not be sufficient time available to allow significant infiltration into the permanent underdrain system.

<Table 2.4-12> summarizes the major onsite storage facilities excluding the cooling tower basins.

Roofs of all buildings are drained to the storm drain system which discharges into the Major and Remnant Minor Streams that feed into Lake Erie. Rainfall on paved and unpaved areas around the main plant buildings generally routed by topography or the storm drain system away from the plant. Rainwater or lawn

a seismic related failure of non-Category I structures without compromising the full operating capability of the redundant permanent dewatering system.

2.4.13.5.6 Controlled Low Strength Material (CLSM)

Controlled Low Strength Material (CLSM) may be used as a replacement for Class B and Class C fill, and as a replacement for Class A fill when the Class A fill was used as bedding and backfill for buried piping and ductbanks only, and not as part of the Plant Underdrain system, or as a foundation for safety-related buildings or structures. Since the CLSM is equivalent to or better than Class B fill in bearing capacity and impermeability, this change has no effect on the results of USAR <Section 2.4.13.5.1> and <Section 2.4.13.5.5>.

2.4.14 TECHNICAL SPECIFICATION AND EMERGENCY OPERATION REQUIREMENTS

Safety-related facilities at PNPP are protected as described in the preceding sections. The only external flooding hazard which requires operator action to protect the Structures, Systems and Components (SSCs) required to achieve and maintain cold shutdown is the LIP event which represents the bounding external flood hazard for PNPP as determined in (Reference 66). Operator action is required only for precipitation hazards which exceed the Standard Project Storm (SPS). Operator action is limited to the deployment of removable incorporated barriers in the form of flooding stop logs. The requirements and guidance for this action is incorporated into plant off-normal instructions. The action is limited to the placing of the stop logs at pre-determined locations. This involves sliding the stop logs into pre-installed end channels and securing the panels with compression clamps. This action is completed without the need for hand tools.

Entry into the plant off-normal instructions is initiated by a meteorological forecast warning which is discussed in detail in <Section 2.4.10.5>. This action is required by PNPP’s Operations

Requirement Manual (ORM) in lieu of placing a limiting condition of operation into PNPP's Technical Specifications as discussed in <Regulatory Guide 1.102>. The alert provides approximately 48 hours of warning which grossly bound the estimated deployment duration of 12 hours. This ensures the barriers can be deployed well in advance of the onset of a flooded condition. If barriers are not deployed in the timeframe allotted in the ORM, a plant shutdown is initiated.

2.4.15 LIST OF PERSONS AND AGENCIES INTERVIEWED

The following is a listing, by sections, of individuals and agencies whose assistance was elicited during preparation of the Hydrologic Engineering discussions:

<Section 2.4.5.2>

Dr. L. Bajorunas, Director, Great Lakes Research Center, NOAA, Detroit, Michigan.

Mr. Elmer Kulp, Acting Chief, Vertical Control Section, Lake Survey Center, NOAA, Detroit, Michigan.

56. Pree, H. L., Jr., "Ashtabula and Conneaut Creek Basins, and Adjacent Lake Erie Tributaries," Ohio Department of Natural Resources, Division of Water, 1960.

57. DeWiest, Roger J. M., Geohydrology, John Wiley and Sons, Inc., New York, 1967.

58. Todd, D. K., Groundwater Hydrology, John Wiley and Sons, Inc., New York, 1959.

59. Woodward-Cylde Consultants letter of November 01, 1981, to Gilbert Associates, Inc. Ref: "See page Estimate."

60. Gilbert Associates Inc. Calculation 26:12, "Perry Emergency Service Water Pumphouse – Stability Analysis", Revision 0 (Retained as Perry Calculation 26:12.000, Revision 0).

61. PNPP Calculation 50:74.000, "Lake Erie Water Levels."

62. Not Used

63. Not Used

64. Not Used

65. PNPP Calculation DX-1.5.1, "Lake Erie Surge Study – Surge Analysis."

66. PNPP Calculation 50:64.000, "PNPP Site Modifications Local Intense Precipitation (Design Basis)."

67. PNPP Calculation 50:65.000, "Evaluation of Structural Roof Capacity for USAR Described PMP Event."

68. PNPP Calculation 50:71.000, "Design Basis Maximum Precipitation (PMP) Determination."

69. PNPP Calculation 50:79.000, "Probable Maximum Winter Precipitation (PMWP, Cool-Season PMP) and Snowmelt Contribution (Design Basis)."

70. PNPP Calculation 50:72.000, "Design Basis Major Stream Probable Maximum Flood (PMF)."

71. PNPP Calculation 50:73.000, "Design Basis Diversion Stream Probable Maximum Flood (PMF)."

72. PNPP Calculation 50:75.000, "Design Basis Standard Project Storm (SPS) Determination."

73. PNPP Calculation 21:02.000, "Misc. Yard Structures – Railroad Bridge."

74. PNPP Procedure EMARP-0005, "Monitoring Shoreline Recession and Bluff Erosion."

75. PNPP Engineering Change Package 13-0802, "Major/Minor Stream Modification."

76. PNPP Calculation 50:68.000, "Perry Nuclear Power Plant (PNPP) Diversion Stream Design Basis Shore Protection Analysis."

77. PNPP Drawing 744-0177-00012, "Stream Relocation Outfall Profile – Stream Outfall."

78. PNPP Calculation 50:80.000, "Effects of Cool-Season Probable Maximum Winter Precipitation (PMWP) Event."

79. PNPP Calculation 50:77.000, "Evaluation of Flood Barriers."

80. PNPP Calculation 21:21.000, "Hydrogen Water Chemistry Tank Foundations and Miscellaneous Equipment Foundations."

81. ANSI Standard ANS-N170-1976-ANS-2.8, "American National Standards for Determining Design Basis Flooding at Power Reactor Sites."

82. PNPP Calculation 50:76.000, "PNPP Design Basis Standard Project Storm (SPS) Rainfall Effects."

83. U.S. Army Corps of Engineers' "Snow Hydrology," June 1956.

84. U.S. Army Corps of Engineers' "Guidelines for Calculating and Routing a Dam-Break Flood," RD-5, Hydrologic Engineering Center, January 1977.

85. U.S. Army Corps of Engineers' "Guidelines for Corps Dams," RD-13, Hydrologic Engineering Center, June 1980.

86. PNPP Calculation 50:82.000, "ESW Swale Discharge Flooding Evaluation."

87. PNPP Calculation 50:83.000, "Mixed Bed Tank Discharge Evaluation."

88. NEI 15-05, "Warning Time for Local Intense Precipitation Events," Revision 6, April 2015, Nuclear Energy Institute.

89. PNPP Calculation 50:85.000, "Precipitation Hazard Alert Evaluation."

90. PNPP Calculation DX-1.5.0, “Lake Erie Surge Levels – Lake Level Studies – PMS.”

91. PNPP Calculation DX 1.1.2, “Emergency Service Water Pumphouse Water Levels.”

with Ohio State Highway 84 east from Painesville to Ashtabula. A lower ridge is contiguous to the north side of U.S. Highway 2 east of the Painesville interchange and U.S. Highway 20 further east. Presumably these ridges are laterally continuous to the west and east and more or less parallel to the present Lake Erie shoreline (Reference 129). Other than that which resulted from erosional processes, little change in the site morphology has taken place since the establishment of the present Lake Erie drainage outlet over Niagara Falls.

Steep bluffs along the southeast shoreline of Lake Erie are continuously subjected to wave action resulting in gradual shoreline recession. Two principal agents of bluff erosion occur: (a) undercutting and erosion by wave action and (b) slump and earthflow. At the site the materials in the shoreline bluff consist of lacustrine deposits underlain by highly compacted glacial till. Groundwater seepage from the face of the bluff is the primary contributing factor to instability of the lacustrine deposits. Wave action erodes the toe of the bluff (dense till) adding to instability of the upper section of the bluff, thereby accelerating the recession process. An approximate yearly rate of natural bluff recession of 5 to 15 feet was reported by the Ohio Division of Geological Survey (formerly Division of Shore Erosion) at Perry Township Park about a mile west of the Perry site (Reference 130). The Corps of Engineers reported a landward movement of 35 feet in the vicinity of the Perry site from 1876 to 1948 and 4 feet per year at Perry Township Park (Reference 131). Further discussion on bluff instability at the site is provided in <Section 2.4.5.5> and <Section 2.5.5>.

2.5.1.2.1.1 Topography

Only minimal topographic changes resulted from plant construction activities. These include minor regrading at and around plant structures and the channelizing of two small, adjacent streams. Post-construction changes include installation of an additional engineered trapezoidal channel (the Diversion Stream) and associated earthen berm and fill area as depicted in <Figure 2.4-7>. Other post-construction

activities have resulted in changes to plant grade elevation and re-contouring as depicted in <Figure 2.4-84>. None of these post-construction changes have introduced any features subject to geologic failure mechanisms (landslides, surface and subsurface subsidence, and collapse). Local relief and slope conditions remain essentially the same. The greatest of both is represented by the shoreline bluff. Excavated debris with variable relief estimated to be

100 feet at its zenith was stockpiled in the general vicinity of the Unit 2 cooling tower throughout much of the construction phase. This borrow pile provided the greatest local relief at the site. Excluding the presence of plant structures, permanent alterations to the preconstruction landscape are not readily apparent except for smooth contouring performed at the barge slip, former lakefront emergence for the minor stream. An elongated, discontinuous berm, approximately 100 feet wide at its base with 20 feet of maximum relief, is parallel and adjacent to Parmly Road. Although this berm is consistent with Lake Plain geomorphologic features, it together with the barge slip comprises the major site topographic alterations. <Figure 2.5-32> is a set of aerial photographs documenting construction stages. <Figure 2.5-33> shows final site topography.

2.5.1.2.1.2 Site Drainage

Final site drainage has developed over the life of the facility due to erection of plant structures and installation of the site storm drain system. However, overland flow continues to occur to Lake Erie and the adjacent streams consistent with pre-construction topography. Eastern, southern and western site drainage occurs via the site storm drainage system to the northwest sediment control dam and to the Major and Remnant Minor Stream engineered channels. The Major and Remnant Minor Stream engineered channels also provide for sediment control in settling basins preceding their emergence along the Lake Erie shoreline. Details of the site drainage, diversion channels and sediment control dams are discussed in <Section 2.4.1> and <Section 2.4.2>.

2.5.1.2.1.3 Soil Deposits

Soils in the locality of the site are derived predominantly from glaciolacustrine deposits. Lacustrine deposits occur as very fine sandy, clayey silt and silty clay. The lacustrine soil stratum above the till is as much as 30 feet thick. Lacustrine sediment permeability decreases with depth. The base of the till, which rests on shale

permeability. The latter was accomplished to assess the influences of silt contamination on the performance of the porous concrete drainage blanket.

The field studies indicated that the most severe contamination occurred at exposed and unprotected edges of the porous concrete blanket. Generally, these exposed edges existed in Unit 1 auxiliary building, Unit 1 heater bay pit, control complex, radwaste building, Unit 2 turbine power complex trench, Unit 2 condensate demineralizer pit, Unit 2 auxiliary building, and several underdrain manhole bases. In addition, the detailed investigations revealed a limited degree of contamination within localized zones beneath both reactor buildings.

The areas affected were corrected in all cases by one of two methods: complete removal and replacement with new porous concrete, or continuous flushing with water. Generally, areas determined to be heavily contaminated were removed. The areas found to be less contaminated were subjected to the flushing method.

Based upon the results of the testing and analyses, the following conclusions evolved. The infiltration of silt which occurred in localized areas of the then existing portions of the porous concrete blanket would have a negligible effect on the future performance of the underdrain/pressure relief system. Laboratory testing confirmed that significant pore pressures cannot build up in even highly contaminated porous concrete.

2.5.5 STABILITY OF SLOPES

The plant is constructed on an essentially level site and the final grades are similar to the preconstruction grades. All excavations for Seismic Category I plant structures have been backfilled and, hence, do not represent slopes which could fail and adversely affect the

safety of the plant. Other man-made slopes, such as the Diversion Stream, are spatially located away from the plant and therefore cannot fail in a way that adversely affects plant safety. The only natural slopes which could affect the safety of the plant are a bluff along Lake Erie which is described in the following sections.

2.5.5.1 Slope Characteristics

A steep bluff which forms the shoreline of Lake Erie is located approximately 300 feet north of the Emergency Service Water Pumphouse. The lower portion of this slope is periodically subjected to erosion due to wave action. In addition, some slumping of the upper bluff materials due to groundwater seepage and frost action has been observed. The resulting estimated average recession rate is two feet per year, as described in <Section 2.4.5.5>. The bluff is about 45 feet in height and has an average slope inclination of about 2 horizontal to 1 vertical, as shown in <Figure 2.5-209>.

2.5.5.2 Design Criteria and Analyses

Stability analyses have been conducted to determine the amount of bluff recession which can occur before the Emergency Service Water Pumphouse would become endangered. The subsurface stratigraphy of the bluff was determined from observations of the exposed bluff slope and from nearby test borings. The stability analyses were conducted using the LEASE-I and LEASE-II computer programs, which utilize the simplified Bishop circular arc method (Reference 259) (Reference 260) and the Morgenstern-Price method (Reference 261) (Reference 262), respectively. For the seismic condition, a seismic coefficient of 0.15 was used for pseudostatic analyses. The groundwater level was taken to be Elevation 615’ near the Emergency Service Water Pumphouse, exiting the bluff slope at Elevation 590’.

The soil strength parameters used in the stability analysis were determined based upon CIU triaxial compression tests on the lacustrine and upper till soils which are summarized in <Table 2.5-37>. Three sets

2.5.5.4 Compacted Fill

There is no compacted fill associated with the Lake Erie bluff.

2.5.6 EMBANKMENTS AND DAMS

There is one earthen embankment credited for flood protection at PNPP, the Diversion Stream berm, which is installed between the Remnant Minor Stream Channel and the Diversion Stream. The berm is constructed using the compacted excavation spoils from the Diversion Stream channel; typical construction is shown on <Figure 2.4-8a>. The berm is nonsafety-related and not seismically qualified. The berm does not normally restrain water and only serves a flood protection function during extreme streamflow conditions resulting from probable maximum or near-probable maximum precipitation events. The berm does not provide a cooling water function. Berm failure is evaluated and included in (Reference 66) of <Section 2.4> for flooding considerations. Periodic inspection of the berm is performed to ensure long-term stability and reliability of the topographic feature including inspection for settlement and piping formation.

2.5.7 REFERENCES FOR SECTION 2.5

1. Mauk, Frederick J., 1978, Geophysical Investigations of the Anna, Ohio, Earthquake Zone, Annual Progress Report: prepared for the U.S. Nuclear Regulatory Commission by the Department of Geology and Mineralogy, University of Michigan, Ann Arbor, UMGM-NUREG-78/03.

2. Christensen, D. H., Pollack, H. N., Lay, T., Schwartz, S. Y., 1986, Geophysical Investigations of the Western Ontario-Indiana Region, Final Report, October 1981 - September 1986, prepared for the U.S. N.R.C., <NUREG/CR-3145>, Department of Geological Sciences, the University of Michigan.

3. Trifunac, M. D. and Brady, A. G., 1975, "On the Correlation of Seismic Intensity Scales with the Peaks of Recorded Strong Ground Motion", Bulletin of the Seismological Society of America, V. 65, No. 1, pp. 139-162.

4. Weston Geophysical Corp., 1986, Investigations of confirmatory seismological and geological issues, Northeastern Ohio earthquake of January 31, 1986, prepared for Cleveland Electric Illuminating Co., Weston Geophysical Corporation.

3.4 WATER LEVEL (FLOOD) DESIGN

3.4.1 FLOOD PROTECTION

3.4.1.1 Flood Protection Measures for Seismic Category I Structures

<Section 2.4> describes the design flood and the flood mitigation strategies employed at PNPP. For off-site hazards (Lake Erie, adjacent streams), PNPP is passively protected from the effects of the entire range of flood-causing mechanisms via exterior barriers (as defined in Regulatory Guide 1.102) and site topography. For the LIP hazard, PNPP is protected by a combination of passive and deployable incorporated barriers as defined in Regulatory Guide 1.102. The protection features are discussed in detail and depicted in <Section 2.4>. Protection is provided to Unit 1 and Unit 2 powerblock structures as well as the Emergency Service Water Pumphouse. Where necessary, other plant SSCs are protected to ensure the SSCs are not inundated by floodwaters. This ensures all SSCs necessary to achieve and maintain safe shutdown are adequately protected.

Operator action required for temporary incorporated barriers are captured in plant emergency procedures. The required actions are time-validated to ensure sufficient time is available following receipt of the meteorological alert discussed in <Section 2.4>. The operator action consists of placing flood stop logs into pre-installed channels and securing the stop logs with compression clamps. No hand tools are required.

The portions of land safety class structures located below finished grade are protected on their outside surfaces by a continuous waterproofing membrane. Waterstops are provided at construction joints. To ensure shear transfer from the foundation media to the

reinforced concrete building foundations, the shear strength of the waterproofing membrane is greater than that required for the applicable loading conditions.

In the unlikely event that the waterproofing of the structures is insufficient, additional flood protection for safety class components, equipment and systems located below grade is provided; this is accomplished by floors that slope to floor drains. Details of the floor drain systems are discussed in <Section 9.3.3>.

3.4.1.2 Permanent Dewatering System

A permanent underdrain system is constructed under the main plant area as discussed in <Section 2.4.13.5>. This system ensures that the groundwater elevation will not exceed Elevation 590′-0″. All the safety class structures in the main plant area are designed to withstand groundwater to Elevation 590′-0″.

A more detailed discussion of the flooding possibilities and consequences of a circulating water yard pipe break, or an expansion joint rupture within the turbine building via flow through a base mat fracture, is presented in <Section 2.4.13.5> and <Section 10.4.5>.

The design criteria for ensuring the prevention of damage to safety-related equipment and systems by internal flooding due to the failure of non-Category I components and piping are:

a. The plant layout uses separation of Seismic Category I and non-Seismic Category I components by locating them, to the maximum extent possible, in separate buildings.

b. The ECCS equipment is located in separate, water tight compartments.

c. Small leaks will be handled by the floor drain system.

A review of the layout of systems and components was performed to ensure that Items a and b, above, have accomplished the prevention of flooding damage. Details of that analysis are discussed in <Section 3.6.2>.

External floodwater elevations are not a factor since protection is provided as described in <Section 3.4.1.1>. Pertaining to the Underdrain System, external flooding from the LIP event does not affect the system. This is due to the short duration of the flooded condition (several hours) and relative impermeability of the land cover types which prevents the underdrain system from being inundated during the LIP event. Additionally, the underdrain system manholes are provided with watertight covers which prevents inleakage. The plant underdrain system discussed in <Section 2.4.13.5> will maintain a groundwater level below Elevation 568′-6″ and will ensure that groundwater level will not exceed Elevation 590′-0″.

The mean monthly high water Elevation of 575.4’ (USGS), which occurred in June 1973, was used in the design of the following submerged safety class structures:

a. Cooling water tunnels.

b. Offshore intake structures.

c. Offshore discharge structure.

3.4.2 ANALYTICAL AND TEST PROCEDURES

The Safety Class I structures will not be subjected to wind wave, hurricane, tsunami, seiche, or dynamic water force. Safety Class I structures are subjected to flood forces as a result of the LIP event. These forces are evaluated in (Reference 79) of <Section 2.4>. Flood-related forces are shown to be bounded by other design basis forces such as tornado wind loads. This evaluation includes the impact of flood-born missiles as discussed in <Section 3.5.1.4>. Analysis is performed using standard fluid dynamics and structural engineering formulae. Refer to <Section 2.4> for a description of the site hydrology. The submerged offshore structures noted in <Section 3.4.1.2> were designed for the effect of dynamic wave action. Consideration of pressure oscillations in the cooling water tunnel system resulting from waves, as described in <Section 2.4.5>, shows that the oscillations are insignificant.

To arrive at a range of wave heights to be used in determining the dynamic forces on the submerged intake structures, wind speeds generated by the probable maximum storm (PMS) <Section 2.4.5> were applied from different directions to give different fetches. The characteristics of waves approaching the structures are affected by wave generation, refraction and shoaling. These factors are in turn dependent on the water depth so that the seiche at the site, which is coincident with the waves, is important. <Section 2.4.5> indicates

that the waves generated by the PMS have a significant height of 17 feet (the corresponding H_{max} = 30 ft). The 31.2-foot depth of water (at maximum stillwater level of 580.5 feet) near the intake structures, limits the wave height to 24.3 feet. All larger waves will break (based on wave breaking criteria of 0.78 of the depth).

Alternately, when the lake is at the recorded low monthly mean level of 569.3’ (USGS) (Reference 61 of <Section 2.4>), resulting in a 20.0 foot average water depth at the intakes, waves higher than 15.6 feet will break. For these reasons, the applicable dynamic forces for the design of the structures are due to the incipient breaking wave height corresponding to water depths between 20.0 feet (15.6 foot wave height) and 31.2 feet (24.3 foot waves). Since the intakes are axisymmetric, the direction of wave approach is not important. Aspects of the wave force analysis are discussed in <Section 2.4.5>.

In addition to the wave loads, the offshore intake and discharge structures are designed for ice loads as discussed in <Section 3.8.4>. These loads are several times larger in magnitude than the loads produced by wave action. Therefore, the results of wave loads were not critical in the structural design of the offshore structures.

3.4.3 FLOOD FORCE APPLICATION

The land safety class structures are analyzed for flood force application in (Reference 79) of <Section 2.4>. Applicable flood forces are determined therein to be grossly bounded by loads from other natural phenomena such as tornado wind loads.

c. Equipment for Maintenance

All other equipment, such as hoists, required during maintenance will either be removed during operation, moved to a location where it is not a potential hazard to safety-related equipment or seismically restrained to prevent it from becoming a missile.

3.5.1.3 Turbine Missiles

Turbine missiles are discussed in (Reference 2) and (Reference 11). Additional information can be found in <Section 10.2.3>.

3.5.1.4 Missiles Generated by Natural Phenomena

There are two sources for externally generated missiles due to natural phenomena. Tornado missiles are discussed in <Section 3.5.1.4.1> and <Section 3.5.1.4.2>. Flood-born missiles are discussed in <Section 3.5.1.4.3>.

3.5.1.4.1 Tornado Missile Selection

<Table 3.5-4> lists potential tornado missiles with corresponding design parameters considered in the plant design. This table represents the spectrum of potential missiles which would be generated on or near the site by the design basis tornado described in <Section 3.3>.

Missile selection is based on the potential for the element to exist on or near the site and the potential for the element to be lifted and accelerated by the tornado wind. Each of the missiles listed is capable of being lifted off the ground using an aerodynamic lift force for an assumed 0.2 second duration and the force of the vertical wind. Once aloft, the vertical tornado wind continues to act. Design velocity for the missile is determined by considering the force of the tornado's tangential wind on the element for the period of time during which the element is maintained aloft. Maximum velocity is limited to

that which would cause the element to exit from the tornado. The wood plank, eight inch wood pole, and the two automobile missiles reach exit velocity, while the remaining missiles cannot be sustained aloft for a sufficient time to achieve exit velocity. Further discussion of tornado missiles is contained in (Reference 3).

As discussed in <Section 3.5.1.4.3>, the effects of flood-borne missiles are bounded by those of tornado missiles. Therefore, tornado generated missiles are considered as the limiting natural phenomena hazard in the design of all structures which are required for ensuring the integrity of the reactor coolant pressure boundary, ensuring the capability to shutdown the reactor and maintain it in a safe shutdown condition, and those whose failure could lead to radioactive releases which would exceed offsite radiation exposure limits [25% of <10 CFR 100> or <10 CFR 50.67> (future revisions to design basis analyses that compare consequences to 10 CFR 100 will be updated to <10 CFR 50.67>) guideline exposures], as discussed in <Regulatory Guide 1.117>.

3.5.1.4.2 Tornado Missile Protection Methods

System and component safety classification and seismic category are given in <Table 3.2-1>. Specific location within the building is provided by the layout drawings <Figure 1.2-2>, <Figure 1.2-3>, <Figure 1.2-4>, <Figure 1.2-5>, <Figure 1.2-6>, <Figure 1.2-7>, <Figure 1.2-8>, <Figure 1.2-9>, <Figure 1.2-10>, <Figure 1.2-11>, <Figure 1.2-12>, <Figure 1.2-13>, <Figure 1.2-14>, <Figure 1.2-15>, <Figure 1.2-16>, and <Figure 1.2-17>. Those systems or components listed in <Table 3.2-1> that are required to ensure the integrity of the reactor coolant pressure boundary, maintain safe shutdown conditions or prevent release of radiation which would exceed offsite radiation exposure limits, are provided with tornado missile protection by location within Seismic Category I structures, by unique missile barriers, by the shielding of an adjacent Seismic Category I structure, or, they have been analyzed as discussed in <Section 3.5.1.4.2.1>.

<Table 3.2-1> also identifies Seismic Category I structures. The exterior walls and roof of these structures are required, by definition, to withstand the effects of the design basis tornado including tornado missiles. These elements are two foot thick (minimum) reinforced concrete having a 28 day compressive strength of 3,000 psi. Design approach is discussed in <Section 3.5.3>. Systems or components located wholly within these structures are considered protected from external

value will be compared to the 3.11×10^{-4} value from <Section 2.3.1.2.2>. The more conservative of these two values will be compared to the occurrence rate for the PNPP area provided in Reference 11, and the most conservative value will be utilized in the PNPP analysis.

2. The Fujita scale (F-scale) wind speeds will be used in lieu of the TORMIS wind speeds (F'-scale) for the F0 through F5 intensities. In addition, a wind speed range from 313 to 360 mph will be used for the F6 intensity to correspond to the tornado wind speed described in <Section 3.3.2.1.2>.

3. The PNPP analysis addresses the TORMIS reduction in tornado wind speed near the ground due to surface friction by injecting the potential tornado missiles into the tornado wind field at elevations above the surface of the ground. The increased injection height will increase the wind speed acting on the missile. The height of injection will ensure that the missiles are subjected to wind speeds greater than or equal to 246 MPH (a V_0/V_{33} value of 0.82).

4. The number of missiles used in the PNPP TORMIS analysis is a conservative value for site specific sources, such as laydown, parking, and warehouse areas. These are postulated by general walkdown information at PNPP.

See <Section 3.5.2> for additional considerations of protection from external missiles.

3.5.1.4.3 Flood-borne Missiles

3.5.1.4.3.1 Flood-borne Missile Selection

The spectrum of potential flood-borne missiles is determined via screening of the tornado missiles provided in <Table 3.5-4>. Per Reference 15), the solid steel rod and pipe is excluded due to lack of

buoyancy. The compact and passenger automobiles are excluded due to the limited floodwater depths and velocities which lack the ability to transport the automobiles. Therefore, the applicable flood-borne missiles are the 125 lb wood plank, 209 lb wood pole and 1,880 lb utility pole. By inspection, the utility pole is the bounding flood-borne missile.

3.5.1.4.3.2 Flood-borne Missile Protection

External flooding protection includes the exterior walls of safety-related buildings, concrete flood wall, aluminum flood wall, deployable flood barriers and steel closure plates. The location of the permanent protection features is depicted in <Figure 2.4-77>. Typical depictions of the concrete and aluminum walls are provided in <Figure 2.4-78> and <Figure 2.4-79>, respectively. Deployable barriers which are provided at building exterior doors are depicted in <Figure 2.4-80> and <Figure 2.4-81>. These features are evaluated in (Reference 15) and shown to be adequately designed for the resultant missile loads concurrent with the loads resulting from the bounding floodwater depth and velocity. Safety-related building exteriors are shown in (Reference 15) to also be capable of withstanding the flood related loads. In the case of safety-related building exteriors, the loads are shown to be grossly bounded by tornado wind and missile impact loads.

Flood-borne missile protection is evaluated using the guidance of (Reference 16), (Reference 17) and (Reference 18), as provided in (Reference 15). The impact forces are converted to equivalent static loads as directed by these references. Additional allowance is provided for concurrent static and hydrostatic flooding loads.

3.5.1.5 Missiles Generated by Events Near the Site

Due to the considerable distance between the potential sources of missiles from accidental explosions in the vicinity of the site to the safety-related structures on the site, no credible events can be postulated to occur.

15. PNPP Calculation 50:77.000, "Evaluation of Flood Barriers."

16. FEMA P-259, "Engineering Principles and Practices for Retrofitting Flood-Prone Residential Structures," 3rd Edition, January 2012.

17. ASCE 7, "Minimum Design Loads for Buildings and Other Structures," 2010 Ed.

18. ANSI A58.1, "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures," 1972.

routed through the Unit 2 turbine power complex and steam tunnel. To increase the storage volume in the TPC, all open penetrations between the turbine power complex and auxiliary buildings below the 599' elevation have been sealed.

Results of the flooding analysis indicate that an uncontrolled guillotine service water line break for 45 minutes after a seismic event (at which time operations will have been able to identify and isolate the leak) will result in the maximum flood height of 595'- 8 ½". Unit 2 turbine power complex/steam tunnel can provide sufficient storage capacity (up to elevation 598'- 11") for the Operators to respond to the event without jeopardizing safety-related equipment.

The flooding event due to a MEB 3.1 crack may occur over several days and this event will be discovered by the Operator during routine rounds. Therefore, a MEB 3.1 crack or a guillotine break of the service water line in Unit 2 turbine power complex or steam tunnel does not create a flooding concern.

3.6.2.3.5.9.3 Unit 2 Transformer Alley

The above ground SW System piping in the Transformer Alley has been evaluated for the effects of a non-mechanistic crack Moderate Energy Line Break (MELB), a seismic event, tornado missiles, and a malevolent vehicle blast during normal operation.

The above ground service water system piping in the Transformer Alley has been evaluated for the effects of tornado missiles as a result of the impact of a 12" X 4" wood plank (considered to be the worst case tornado missile). The results of the analysis provided in (Reference 11) show that passive protection features provided by (Reference 12) are adequate to prevent floodwater ingress due to a tornado missile strike. Hence, flooding in the Transformer Alley due to a tornado missile induced break is not a concern.

6. Moody, F. J., "Fluid Reactor and Impingement Loads," Vol. 1, ASCE Specialty Conference on Structural Design of Nuclear Plant Facilities, pp 219-262, dated December 1973.

7. Gilbert Associates, Inc., "DYREC Program User's Manual," GAI Report No. 1866, July 1975; revised March 1976.

8. McDonnell-Douglas Automation Co., "ICES DYNAL User's Manual," dated September 1971.

9. Eiber, R. J., et. al., "Investigation of the Initiation and Extent of Ductile Pipe Rupture - Phase 1 Final Report - Task 17," Battelle Memorial Institute, BMI-1866, dated July 1969.

10. Biggs, J. M., "Introduction to Structural Dynamics," McGraw-Hill, dated 1964.

11. PNPP Calculation 50:86.000, "Service Water Line Break in Unit 2 Transformer Alley."

12. PNPP Engineering Change 19-0155.

3.8.1 CONCRETE CONTAINMENT

The containment structure for this plant is composed of the free standing steel containment vessel and the annulus concrete acting compositely. A shield building forms a housing for the steel containment and annulus concrete which strengthens and stiffens the containment vessel. Both the shield building and the annulus concrete are discussed in this section beginning with the shield building. The shield building has the following functions:

a. Forms a biological shield for radiation from the reactor.

b. Provides protection for the containment vessel from ground water contact and pressure.

c. Provides weather and exterior missile protection for the containment vessel, including the effects of external flood hazards.

d. Provides a relatively leak tight structure so that the annulus exhaust gas treatment system can be used to minimize the escape of radioactive particles to the environment, by maintaining the annulus air space at a slight negative pressure.

3.8.1.1 Description of the Shield Building

The shield building is a reinforced concrete structure consisting of a flat foundation mat, a cylindrical wall and a shallow dome. The general configuration of the shield building and its relation to the other structures of the reactor building complex is shown in <Figure 3.8-1>. The foundation mat, common to the shield building, annulus concrete, containment vessel, and interior structure is circular in plan with a diameter of 136 feet and a thickness of 12 feet 6 inches. The

3.8.4.1.2 Auxiliary Building

This building is a reinforced concrete structure approximately 97 feet high by 102 feet wide by 192 feet long with the top of mat at Elevation 568’-4”. It houses safety class systems and components, including pumps and motors for the residual heat removal system (RHRS), the high pressure core spray (HPCS), low pressure core spray (LPCS), and the reactor core isolation cooling (RCIC) system. See <Figure 1.2-3>, <Figure 1.2-4>, <Figure 1.2-5>, <Figure 1.2-6>, <Figure 1.2-7>, <Figure 1.2-8>, <Figure 1.2-9>, <Figure 1.2-10>, and <Figure 1.2-11> for general configuration and relationship of the auxiliary building to other structures.

The concrete floors are supported by interior columns, interior walls and by exterior walls. Exterior walls and the roof are a minimum of 2 feet thick for protection of safety class equipment from exterior missiles and from the effects of the environment.

The auxiliary building has a tunnel to house the main steam and feedwater lines which run from the reactor building complex to the steam tunnel. The auxiliary building tunnel is approximately 36 feet wide and 28 feet high.

The function of this tunnel is to withstand the pressures and temperatures that could be produced by a postulated break in a feedwater or main steam line. Seals are provided around the tunnel at its junction with the steam tunnel on one end and a similar tunnel coming from the shield building on the other end. The seals do not form a structural connection to these other tunnels and, therefore, the auxiliary building remains structurally separate from other buildings above the foundation.

The radwaste building houses equipment used in the storage and processing of liquid and solid radioactive wastes.

The exterior walls and roof of the safety class portion consist of 2 to 3 foot thick reinforced concrete to provide shielding for the environment against radiation from the radioactive wastes. Interior shield walls are provided around tanks where necessary to shield personnel during normal operation of the plant. Tanks are mounted on the foundation of the building and on the floors at Elevations 602’-0” and 623’-6”. The concrete floor slabs are supported by exterior and interior walls and by interior columns. The building has no structural attachment to other structures above the top of foundation. The layout and configuration of the radwaste building are shown in <Figure 1.2-3>, <Figure 1.2-4>, <Figure 1.2-5>, <Figure 1.2-6>, <Figure 1.2-7>, <Figure 1.2-8>, <Figure 1.2-9>, and <Figure 1.2-10>.

Both the safety class and non-safety class portions of the Radwaste Building are provided with external flood protection features as described in <Section 2.4>. These features are also designed to withstand the effects of flood-borne missiles as discussed in <Section 3.5>.

3.8.4.1.7 Diesel Generator Building

This structure is a reinforced concrete building approximately 165 feet long, 78 feet wide and 26 feet high, with top of the foundation mat at Elevation 620’-6”. The structural system consists of walls and roof sized for protection against missiles and other natural phenomena including external flooding. Removable concrete missile shields are provided in front of each man access door and labyrinth shields for each man access door. The design of these shields is in accordance with the methods discussed in <Section 3.5>. The safety class equipment is mounted on the foundation slab at Elevation 620’-6”. A reinforced concrete air intake structure, 165 feet long, 32 feet wide and 20 feet high, is mounted to the roof of the building. The walls

and top of this housing are also designed for missile protection. In addition, concrete barriers are provided at the exhaust louvers to prevent external missiles from entering the diesel generator building. For additional details, see <Figure 1.2-3>, <Figure 1.2-4>, <Figure 1.2-5>, <Figure 1.2-6>, <Figure 1.2-7>, <Figure 1.2-8>, <Figure 1.2-9>, and <Figure 1.2-10>.

The diesel generator building houses generators, day tanks and other equipment necessary to supply standby electric power to operate safety systems in the event of a power failure of the plant generating equipment and offsite power. This standby electrical system is described in <Section 8.3>.

3.8.4.1.8 Offgas Building

This building is a reinforced concrete structure 99 feet 9 inches long, 52 feet wide and 80 feet high, that houses equipment used in the filtering and absorption of radioactive noncondensible gases from the main and auxiliary condensers. See <Figure 1.2-3>, <Figure 1.2-4>, <Figure 1.2-5>, <Figure 1.2-6>, <Figure 1.2-7>, <Figure 1.2-8>, <Figure 1.2-9>, and <Figure 1.2-10> for general configuration and relation to other structures.

The building is a four story structural system that is completely enclosed by a 2 foot thick roof slab and exterior walls of 2 feet minimum thickness, which are designed to resist the exterior missiles as defined in <Section 3.5> and external flood hazards as discussed in <Section 2.4>. The top of the concrete mat foundation is at Elevation 584’-0”.

3.8.4.1.9 Emergency Service Water Pumphouse

The emergency service water pumphouse is a reinforced concrete structure, rectangular in plan, located between the nonsafety class service water pumphouse and the nonsafety class discharge tunnel entrance structure. The five foot thick foundation mat is placed on one foot of porous concrete which is founded on chagrin shale at approximately Elevation 531’-0”. The structure has a reinforced concrete floor at Elevation 586’-6”, which supports the pumps and screens. The structure provides protection from external natural phenomena including external flood hazards and associated external missiles as discussed in <Section 2.4> and <Section 3.5>. Above this

floor an overhead crane is supported on steel girder rails which bear on reinforced concrete wall pilasters. Above that is a 2 foot thick reinforced concrete roof supported by steel

f. Range of Transient, Steady-State and Environmental Conditions

Environmental conditions for proper operation of the RPS components are discussed in <Section 3.11>. The RPS power supply range of steady-state and transient conditions are provided in <Chapter 8>.

g. Malfunctions, Accidents and Other Unusual Events Which Could Cause Damage to Safety Systems

Unusual events are defined as malfunctions, accidents and others which could cause damage to safety systems. <Chapter 15> and <Appendix 15A> describe the following credible accidents and events; floods, storms, tornadoes, earthquakes, fires, LOCA, pipe break outside containment, feedwater line break, and missiles. Each of these events is discussed below for the RPS.

All components essential to the operation of the RPS are designed, fabricated and mounted into appropriate seismically qualified structures. However, even though the sensors initiating reactor scram which monitor turbine stop valve position and turbine control valve fast closure are designed and purchased to Quality Class 1, Seismic Class I, they are physically mounted on equipment which is not Seismic Class I/Quality Class 1, and are located in the turbine generator building which is not Seismic Class I. For this reason, other diverse variables (reactor pressure and neutron flux trips) can be relied upon for reactor scram if components in the turbine generator building fail.

1. Floods

The buildings containing RPS components have been provided with flood protection features for the entire range of flood-causing mechanisms up to and including the Probable Maximum Flood (PMF) as discussed in <Section 2.4>. This ensures that no buildings containing RPS components will be adversely affected from external flood hazards.

d. Margin

The margin between operational limits and the limiting conditions of operation of ESF systems are accounted for in Technical Specifications.

e. Levels

Levels requiring protective action are established in Technical Specifications.

f. Range of Transient, Steady-State and Environmental Conditions

Environmental conditions for proper operation of the ESF components are discussed in <Section 3.11>.

g. Malfunctions, Accidents and Other Unusual Events Which Could Cause Damage to Safety System

<Chapter 15> describes the following credible accidents and events: floods, storms, tornadoes, earthquakes, fires, LOCA, pipe break outside containment. Each of these events is discussed below for the ESF systems.

1. Floods

The buildings containing ESF systems components have been provided with flood protection features for the entire range of flood-causing mechanisms up to and including the Probable Maximum Flood (PMF) as discussed in <Section 2.4>. This ensures that no buildings containing ESF systems components will be adversely affected from external flood hazards. For a discussion of internal flooding protection, refer to <Section 3.4.1> and <Section 3.6>.

e. Levels

Levels requiring protective action are established in Technical Specifications.

f. Range of Transient, Steady-State and Environmental Conditions

Refer to <Section 3.11> for environmental conditions. Refer to <Section 8.2.1> and <Section 8.3.1> for the maximum and minimum range of energy supply to the safe shutdown systems instrumentation and controls. All safety-related instrumentation and controls are specified and purchased to withstand the effects of these energy supply ranges.

g. Malfunctions, Accidents and Other Unusual Events Which Could Cause Damage to Safety System

<Chapter 15> describes the following credible accidents and events: floods, storms, tornadoes, earthquakes, fires, LOCA, pipe break outside containment, and feedwater line break. Each of these events is discussed below for the safe shutdown systems.

1. Floods

The buildings containing safe shutdown system components have been provided with flood protection features for the entire range of flood-causing mechanisms up to and including the Probable Maximum Flood (PMF) as discussed in <Section 2.4>. This ensures that no buildings containing safe shutdown systems components will be adversely affected from external flood hazards. For a discussion of internal flooding protection, refer to <Section 3.4.1> and <Section 3.6>.

Environmental conditions for proper operation of the systems described in <Section 7.6> are discussed in <Section 3.10> and <Section 3.11>.

g. Malfunctions, Accidents and Other Unusual Events Which Could Cause Damage to Safety Systems

<Chapter 15> and <Appendix 15A> describe the following credible accidents and events; floods, storms, tornadoes, earthquakes, fires, LOCA, pipe break outside containment, and missiles.

1. Floods

The buildings containing safety-related components have been provided with flood protection features for the entire range of flood-causing mechanisms up to and including the Probable Maximum Flood (PMF) as discussed in <Section 2.4>. This ensures that no buildings containing safety related components will be adversely affected from external flood hazards. For a discussion of internal flooding protection refer to <Section 3.4.1> and <Section 3.6>.

2. Storms and Tornadoes

The buildings containing safety-related components have been designed to withstand all credible meteorological events and tornadoes as described in <Section 3.3>.

3. Earthquakes

The structures containing safety-related system components have been seismically qualified as described in <Section 3.7> and <Section 3.8>, and will remain functional during and following a safe shutdown earthquake (SSE).

full core of fuel assemblies; it is designed to withstand all credible static and dynamic loadings, thereby preventing damage to the structure of the racks and the contained fuel, and minimizing distortion of the racks arrangement <Table 3.9-2>.

b. The racks are designed to protect the fuel assemblies from excessive physical damage which may cause the release of radioactive materials in excess of <10 CFR 20> and <10 CFR 50.67> requirements under normal or abnormal conditions caused by impacting from other fuel assemblies.

c. The racks are constructed in accordance with the Quality Assurance requirements of <10 CFR 50, Appendix B>.

d. The spent fuel storage racks are categorized as Safety Class 2 and Seismic Category I.

e. The spent fuel storage facility is designed in accordance with General Design Criteria 2, 3 and 4, and <Regulatory Guide 1.13>, <Regulatory Guide 1.29>, <Regulatory Guide 1.102> with exceptions as provided in <Section 2.4>, and <Regulatory Guide 1.117>. The design precludes any deleterious effects on spent fuel rack integrity due to natural phenomena such as earthquakes, tornadoes, hurricanes, missiles, and floods. Compliance with <Regulatory Guide 1.13> is discussed in <Section 9.1.2.3.3> and <Section 1.8>.

9.1.2.1.2 Structural - PAR Racks

Structural related safety design bases for Programmed and Remote Systems, Inc. (PAR) racks are as follows:

a. The densified spent fuel storage racks are designed to withstand all credible static and dynamic loadings to prevent damage to the

rack structure, thereby preserving the structural integrity of the contained fuel and minimizing distortion of its array in storage.

b. The densified racks are designed to protect the spent fuel assemblies from excessive physical damage which might cause the release of radioactive materials in excess of <10 CFR 20> requirements under normal and abnormal conditions.

c. The densified racks are constructed in accordance with the Quality Assurance requirements of <10 CFR 50, Appendix B>.

d. The densified racks are categorized as Safety Class 2 and Seismic Category I.

e. The densified racks provide storage spaces for a total of 4,020 spent fuel assemblies and 30 spaces for multi-purpose storage of containers for failed fuel, channels or control rods. The rack array in the fuel preparation and storage pool provides storage spaces for 1,620 spent fuel assemblies and 30 spaces for multi-purpose storage. The rack array in the spent fuel pool provides storage spaces for 2,400 spent fuel assemblies.

f. The densified rack design precludes the possibility of placing fuel elements anywhere within the array other than in the storage spaces provided.

g. The spent fuel storage facility is designed in accordance with General Design Criterion 2, 3 and 4, and <Regulatory Guide 1.13>, <Regulatory Guide 1.29>, <Regulatory Guide 1.102> with exceptions as provided in <Section 2.4>, and <Regulatory Guide 1.117>. This design precludes any deleterious effects on spent fuel rack integrity due to natural phenomena such as earthquakes, tornadoes, hurricanes, missiles, and floods.

9.3.8.2 System Description

The hydrogen water chemistry system is shown schematically by <Figure 9.3-35>. The hydrogen water chemistry system consists of a storage subsystem, a supply subsystem, and an injection subsystem.

Liquid Hydrogen (H_2) and Oxygen (O_2) are cryogenically stored. The liquid hydrogen is pumped into supplemental gaseous storage tanks. The gaseous and cryogenic storage tanks are designed, fabricated, tested, and stamped in accordance with ASME Section VIII, Division 1. Hydrogen and oxygen piping complies with standard industrial code requirements as specified by EPRI NP-5283-SR-A, "Guidelines for Permanent BWR Hydrogen Water Chemistry Installations-1987 Revision". The cryogenic piping is designed to ANSI B31.3 and all other system piping is designed to ANSI B31.1.

The storage tanks are designed to withstand tornado missiles and design basis seismic loading. The associated foundations are designed to ensure that tanks remain in place for seismic, design basis tornado, site-specific flood per (Reference 80) of <Section 2.4>, and ice and snow conditions. Drainage between the separate hydrogen and oxygen storage areas ensures that a liquid spill from either tank storage area will not flow toward, pond or accumulate within 75 ft. of each other.

The liquid hydrogen and oxygen are cryogenically stored in separate storage facilities remote from the nearest safety-related structure and air intakes into safety-related structures. The location of the storage facility complies with the separation distance outlined in EPRI NP-5283-SR-A, "Guidelines for Permanent BWR Hydrogen Water Chemistry Installations-1987 Revision" as well as OSHA Standards 29 CFR 1910.103 and 29 CFR 1910.104. This separation distance ensures that the thermal flux from a potential hydrogen gas fireball or the blast overpressure from a potential hydrogen blast will not cause failure of any

The maximum groundwater level is lowered to a point 10 feet below the bottom of the tanks in this area due to the main plant underdrain system and Class A bedding; this will avert the threat of possibly lifting the storage tanks due to hydrodynamic forces from a buildup of water around the tanks <Figure 9.5-21>, and <Figure 9.5-22>. The storage tanks are designed so that all openings are above the ground water levels to prevent the entrance of water. Tank openings (overflow flame arrestors, vent line flame arrestors, dipstick connection, fill connections and water removal connections) are either located above the design basis flood level (DBFL) or are provided with passive protection to prevent floodwater intrusion. Protection features include closure plates and caps judged to be substantially watertight. The only anticipated source of water into the tanks will result from moisture being carried with air that enters the tank through the vent. The maximum rate of this accumulation would occur during a prolonged run of the standby diesel generator when air is drawn into the tank to displace fuel used. Under the worst possible conditions on a hot humid day, approximately 42 cubic feet per hour of air will enter the tank and approximately 0.30 gallons of water per day will be deposited. This accumulation of water will be detected by routine sampling and will be pumped out as required.

9.5.4.4 Inspection and Testing Requirements

Proper operation of the transfer pumps and the level alarm signals will be checked at scheduled intervals to assure their availability. This includes checks of the following:

a. Primary transfer pumps start and stop automatically at the desired levels.

b. Standby transfer pumps start and stop automatically at the desired levels.

c. Alarm signals for high and low day tank levels function at the designated levels.

d. Low level signals for the storage tanks function at the designated levels.

silencer would be automatically bypassed through the missile barrier discharge. The system design conforms with the requirements of GDCs 1, 2, 4, 5, and 17 <Section 3.1>. Guidance presented in <Regulatory Guide 1.26>, <Regulatory Guide 1.29>, <Regulatory Guide 1.68>, <Regulatory Guide 1.102> with exceptions as discussed in <Section 2.4>, and <Regulatory Guide 1.117> has been considered in the design of the system. The degree of conformance with these regulatory guides is discussed in <Section 1.8>. Conformance with Branch Technical Positions ASB 3-1 and MEB 3-1, as relates to breaks in high and moderate energy piping systems outside containment, is discussed in <Section 3.6.1> and <Section 3.6.2>. The guidelines presented in Branch Technical Position ICSB-17 (PSB) have also been considered in the design of this system as discussed in <Chapter 8>.

The standby diesel generator combustion air intake and exhaust system is classified as Safety Class 3, Seismic Category I, except for the crankcase vent lines and exhaust silencers which are nonsafety-related. The system is designed in accordance with the requirements of ASME Code Section III, Class 3, NFPA-37 and DEMA Standards.

9.5.8.2 System Description

Each standby diesel generator combustion air intake and exhaust system consists of two air intake filters, two air intake silencers, expansion joints, two exhaust silencers, and associated piping connecting the equipment.

Combustion air at a rate of 14,078 scfm (each filter) is drawn through 50 percent capacity, oil bath type, air intake filters located in louvered cubicles on the diesel generator building roof. These filters clean the ambient air for admittance to the diesel generator. The air

The intake and exhaust systems contain no flow control devices (louvers, dampers). The standby diesel generator combustion air intake and exhaust systems are shown in <Figure 9.5-12>, and the layout arrangement is shown in <Figure 1.2-6> and <Figure 1.2-13>.

9.5.8.3 Safety Evaluation

Each standby diesel generator is provided with a completely separate and independent combustion air intake and exhaust system. These systems are not redundant since there are three divisions of the emergency power system <Section 8.3>. The system is protected from external flood hazards by a combination of elevation, passive incorporated barriers and temporary barriers deployed per operator action as discussed in <Section 2.4>.

Arrangement and location of combustion air intake and exhaust, as shown in <Figure 1.2-6> and <Figure 1.2-13>, are such that the dilution or contamination of intake air by exhaust products will not preclude the operation of the standby and HPCS diesel generators. Recirculation of standby diesel engine combustion products to the air intakes is prevented by locating the exhaust stacks at a higher elevation away from the intakes. Since hot exhaust gases rise and disperse, significant recirculation into the intakes cannot occur. Combustion gases exhaust from the standby diesel engine at a rate of 30,500 scfm. These gases exhaust through a spark arresting type exhaust silencer. It would be necessary for more than 13.2 percent of the exhaust gas (4,026.4 scfm) to be recirculated into the air intakes to deteriorate operation of the standby diesel generator. This same percentage recirculation, 13.2 percent, would apply to the HPCS diesel generator air intake before degradation of performance would occur.

The exhaust plane of the silencer for the standby diesel generator exhaust system is 44 feet horizontal distance from the air inlet piping and 5’-7” above the high point of the inlet louvers. The exhaust plane of the HPCS diesel exhaust silencer is 29 feet horizontal distance from the air inlet and 6 feet above the high point of the inlet louvers. In

auxiliary building. At this location a grass swale is provided to carry the flow from the auxiliary building area, between the cooling towers, to the Remnant Minor Stream Channel on the east side of the plant. This water then flows in the Remnant Minor Stream Channel over the sediment control dam and ultimately enters Lake Erie at the shoreline. If this path were used by the effluents following the postulated accident, dilution of the radioactive liquids would occur in the Remnant Minor Stream Channel and in Lake Erie with the non-contaminated lake water. In calculating the resultant individual exposures from this pathway, it was conservatively assumed that no dilution occurred in the grass swale.

g. No credit is taken for any settling or plating out of the radioisotopes.

h. The dose conversion factors for the isotopes considered are taken from (Reference 7).

i. For the purposes of calculating the average fraction of <10 CFR 20, Appendix B> effluent concentration, the total release of the radioisotopes into the lake is averaged over a one year period. (Radiological assessments performed prior to October 4, 1993 that were used for the plant design bases as discussed in this USAR were evaluated against the <10 CFR 20> regulations prior to October 4, 1993. Radiological assessments for plant design bases modifications that are performed after October 4, 1993 will be evaluated using the revised <10 CFR 20> dated October 4, 1993.)

j. The resultant ingestion exposure is calculated for an individual drinking potentially contaminated water for a period of one year at a rate of 2,000 cc/day. The isotopic concentrations in this water are conservatively assumed to be the concentrations calculated at the nearest drinking water intake.