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2.0 Site Characteristics

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2.1 Geography and Demography

McGuire Nuclear Station is located geographically near the center of the highly industrialized region of the Carolinas. The land is predominantly rural non-farm with a small amount of land being used to support beef cattle and farming. Recreation in the area is confined mostly to the lake and shores of the Lake Norman and Mountain Island Reservoir.

[HISTORICAL INFORMATION NOT REQUIRED TO BE REVISED]

2.1.1 Site Location

The McGuire Nuclear Station site is located as shown in Figure 2-1 at latitude 35 degrees - 25 minutes - 59 seconds north and longitude 80 degrees - 56 minutes - 55 seconds west. The Universal Transverse Mercator Grid Coordinates are shown in Figure 2-2. The site is in northwestern Mecklenburg County, North Carolina, 17 miles north-northwest of Charlotte, North Carolina. McGuire site is bounded to the west by the Catawba River channel and to the north by the 32,510 acre Lake Norman as shown in Figure 2-3. Lake Norman is impounded by Duke Power Company's Cowans Ford Dam hydroelectric station, which is located immediately west of the site and on the Catawba River channel.

Lake Norman extends 34 miles up the Catawba River to Duke's Lookout Shoals Hydroelectric Station. The tailwater of Cowans Ford Dam is the upper limit of Duke's Mountain Island Lake. Mountain Island Dam is located 15 miles downstream from the site.

Duke's Marshall Steam Station is located on the western shore of Lake Norman, approximately 16 miles upstream from the site.

2.1.2 Site Description

The site area varies in elevation from 650 to 800 feet msl. The powerhouse yard is elevation 760, the administration building and yard are elevation 747 and the switchyard, across highway N.C. 73 is elevation 738. Lake Norman full pond elevation 760 is retained in the site area by the east abutment dike, which has a crest elevation at the site of 780 msl.

The access railroad and access road for the plant enter the site from the south along N.C. Highway 73.

All of the property within the 2500 foot radius Exclusion Area and approximately 30,000 acres around Lake Norman is owned in fee by Duke. Figure 2-4 shows a plot plan on the McGuire site.

2.1.2.1 Exclusion Area Control

The Exclusion Area boundary is formed by a 2500 foot radius centered on the Reactor Building's centerlines as shown in Figure 2-5.

The only commercial enterprise within the Exclusion Area is the McGuire Nuclear Station and its associated activities. Other activities in the exclusion area are limited to highway traffic on North Carolina 73, on the southern edge of the Exclusion Area and recreation on Lake Norman to the north.

In the event of an emergency, traffic in the local area will be controlled through agreements with Charlotte Mecklenburg Police, and the North Carolina Highway Patrol.

Emergency plans and procedures are fully described in Section 13.3.

A security fence restricts admittance to the immediate plant area through a guarded gate.

Duke, through ownership of property and through agreements with and cooperation of the Charlotte Mecklenburg Police and North Carolina Highway Patrol, and Lake Norman Marine Commission, can exercise adequate control in the Exclusion Area.

2.1.2.2 Boundaries for Establishing Effluent Release Limits

Normally, the only gaseous radioactive waste released from this station will be the activity contained in Auxillary Building ventilation exhaust air and in occasional Reactor Building purges, both of which are continuously monitored and controlled. Releases based on the quantities of various radionuclides and site meteorology are limited by dose rate at or beyond the site boundary. These release limits allow operational flexibility while maintaining doses to the public which are small percentages of the limits of 10CFR50.

Information regarding radioactive gaseous and liquid effluents, which will allow identification of structures and release points as well as definition of unrestricted areas within the site boundary that are accessible to members of the public, shall be as shown in Figure 2-5. The definition of unrestricted area used in implementing the Radiological Effluent Technical Specifications has been expanded over that in 10 CFR Part 20.3(a) (17). The unrestricted area boundary may coincide with the Exclusion (fenced) Area boundary, as defined in 10 CFR Part 100.3(a), but the unrestricted area does not include areas over water bodies. The concept of unrestricted areas, established at or beyond the site boundary, is utilized in the Limiting Conditions for Operation to keep levels of radioactive materials in liquid and gaseous effluents as low as is reasonably achievable, pursuant to 10 CFR Part 50.36a.

The boundary used to establish liquid effluent release points is the discharge structure. Distances from release points to the Exclusion Area Boundary are shown in Figure 2-5. Details of the release points are given in Sections 11.2.7 and 11.3.7.

2.1.3 Population and Population Distribution

Current population data information can be found in the “Evacuation Time Estimates for the McGuire Nuclear Site Plume Exposure Pathway Emergency Planning Zone” document. The station is required to develop and maintain this document in accordance with 10CFR50.47

[HISTORICAL INFORMATION NOT REQUIRED TO BE REVISED]

Population centers (population of approximately 25,000 or more) within a 100 mile radius of the site are shown in Figure 2-6 and Table 2-26. The population of counties within a 50 mile radius of the McGuire Nuclear Station is shown in Figure 2-7 and Table 2-27.

All population and population distribution data are based on the 1970 census, except as noted. The 1970 population within the five mile radius is based on an actual house count made June 17-19, 1970 and 3.17 persons per household. To disaggregate the 1970 county census division populations into each radial sector, road densities, population accumulations, land usage and general area information are considered.

A summary of the population and population distribution (1970-2020) in each sector within 50 miles of the site is given in Table 2-1.

Future population levels for the next four decades are based on population projections made by the U.S. Environmental Protection Agency (Reference 1). The method of distribution of the projected populations is the same as for 1970 population.

2.1.3.1 Population Within 10 Miles

Figure 2-8 through Figure 2-13 present detailed information on the present and projected resident population and its distribution within a 10 mile radius of the plant site. The population distribution is shown in 22-1/2 degree sectors for 1, 2, 3, 4, 5 and 10 mile radii. See Table 2-1 for 1990 census populations.

2.1.3.2 Population Between 10 and 50 Miles

Figure 2-14 and Figure 2-19 present detailed information describing the resident population and its distribution and projection for four decades. See Table 2-1 for actual 1990 census populations.

2.1.3.3 Low Population Zone

The Low Population Zone for McGuire Nuclear Station is an area extending from the centerline of reactors at a radius of 5.5 miles. The 5.5 mile radius was selected on the basis of the guidelines set forth in 10CFR 100. Population densities and road systems within this area are such that appropriate protective measures can be taken to protect the population in the event such action should become necessary. Specific actions to be taken are detailed in Section 13.3.

The population within the Low Population Zone is detailed on Figure 2-20 through Figure 2-25.

Calculated dose levels that could possibly exist in portions of the Low Population Zone, as a result of accident conditions, are detailed in Chapter 15.

2.1.3.4 Transient Population

Lake Norman is the major contributor to the part-time population within a 50 mile radius of the site. The recreational opportunities offered by the lake, such as boating and fishing, increase the population of the area during the summer months. A peak day recreational population of approximately 40,000 can be expected in the Lake Norman area.

Figure 2-26 and Figure 2-27 show the current (1970) recreation area peak attendance for zero to five mile and five to ten mile radii respectively.

Daily population changes within the Low Population Zone occur with residents leaving the area in the morning, commuting to population centers for employment and returning in the evening. There are no large industries within the Low Population Zone to draw residents during working hours.

The Duke Power Company Training and Technology Center is located just outside the Exclusion Area Boundary. This facility has a projected work force and student enrollment of 352.

2.1.3.5 Population Center

Charlotte, North Carolina, with a 1970 population of 241,178, is the population center, as defined in 10CFR 100, nearest to McGuire. The closest point from McGuire to the population center boundary is 11 miles. The location of other population centers are shown in Figure 2-6 and Table 2-26.

2.1.3.6 Public Facilities and Institutions

Public facilities, including schools, hospitals, prisons, and parks within 10 miles of the site, are detailed on Figure 2-28. Populations of the facilities are given on Table 2-2.

2.1.4 Uses Of Adjacent Land And Waters

The land area harvested or cultivated in the counties within 50 miles of the site is detailed in Figure 2-29. The acreage of production and yield in bushels per acre of the principal food products are listed in Table 2-3.

Figure 2-30 and Table 2-28 show the number of dairy animals that are pastured in the counties within 50 miles of the site. The nearest location where commercial dairying is performed is 2.5 miles East of the site. Figure 2-31 and Table 2-29 show the location of dairy farms and number of dairy cows within five miles of the site.

Sources of drinking water are detailed on Figure 2-50 and Table 2-16.

No commercial fisheries were permitted on Lake Norman until April, 1973, when permission was granted by the North Carolina Wildlife Resources Commission for commercial catfishing in the lake. Duke has been advised by the Wildlife Commission that the required information on commercial fish catches is not yet available. However, information regarding the commercial catch can probably be obtained by the regulatory staff by requesting it from Mr. Clyde P. Patton, Executive Director, North Carolina Wildlife Resources Commission, 325 N. Salisbury Street, Albemarle Building, Raleigh, North Carolina 27611.

Information regarding fish species composition and relative abundance in Lake Norman is provided in Table 2-4 and in Appendix 4A of the McGuire Environmental Report (Operating License Stage).

The important sport fishes in Lake Norman are listed by the North Carolina Wildlife Resources Commission in terms of percent — catch as follows: "41 percent sunfish, 33 percent crappie, 13 percent largemouth bass, 5 percent catfish, 4 percent white bass, 3 percent carp." (Reference 2).

The existing land use within two miles of the site is shown on Figure 2-32. Land owned by Duke within the Low Population Zone (5.5 mile radius) is shown on Figure 2-33.

The Duke Power Company Training and Technology Center is located northeast of McGuire Nuclear Station on a peninsula just outside the Exclusion Area Boundary. The station will be responsible for providing security support services. Also, McGuire and the Center will share demineralized water reserves produced at the plant and piped to the center. No large quantities of caustic or flammable material will be stored on site. A chemical waste tank and low level radioactive waste tank, each with a 1000 gallon capacity, will be buried on site with contents removed periodically by a licensed contractor.

2.1.5 References

1. *"Population by County, Historic (1940-1970) and Projected (1980-2020) Region IV" published by the Environmental Protection Agency, Atlanta, Georgia, July, 1972.*
2. *A Catalog of the Inland Fishing Waters in North Carolina, 1968.*

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2.2 Nearby Industrial, Transportation and Military Facilities

Current industry data information can be found in the "Evacuation Time Estimates for the McGuire Nuclear Site Plume Exposure Pathway Emergency Planning Zone" document. The station is required to develop and maintain this document in accordance with 10CFR50.47.

[HISTORICAL INFORMATION NOT REQUIRED TO BE REVISED]

Military and transportation facilities are nearly non-existent and only a few industrial facilities are located in the vicinity of McGuire. The few facilities that do exist have no effect on the McGuire Nuclear Station nor will McGuire Nuclear Station have any effect on the existing facilities.

2.2.1 Locations and Routes

Figure 2-35 shows the locations of State and Federal highways, railroads, airports, pipe lines, and tank farms within 20 miles, manufacturing plants within 10 miles and quarry operations within 5 miles of McGuire Nuclear Station. Table 2-5 gives details for Figure 2-35. There are no chemical plants within 10 miles of the site. There are no military installations within 50 miles of the site with the exception of the North Carolina National Guard at Douglas Municipal Airport.

Transcontinental Pipeline, Inc., has four major pipelines in the vicinity of McGuire Nuclear Station. Two of these pipes are located approximately two miles to the north of the site and two are located approximately one mile to the south. The locations and details of these pipes which transport methane gas are shown on Figure 2-36.

Douglas International Airport, located approximately 14.5 miles south of the site, is the largest of the 11 airports within 20 miles of the site. There are 3 runways: 5/23 having a length of 7500 feet, 18L/36R having a length of 8845 feet, and 18R/36L having a length of 10,000 feet. The long runway can accommodate wheel loadings for the L-1011 at a weight of 350,000 pounds and the 13-747 at a weight of 750,000 pounds and future aircraft derivatives.

Annual aircraft operations are forecast at 340,000 in 1985. Table 2-6 lists the total operations by category. Table 2-7 denotes the percent of operations by aircraft type. The Air National Guard operates six C-130 aircraft that weight approximately 135,000 pounds each and fly approximately 20 flights per week.

Total operations at Douglas International Airport for July 1, 1972 to June 30, 1973 period were 182,000 of which 125,000 were conducted under instrument flight rules. Commercial and military flights into and out of Douglas are controlled by the Airport tower on regulated flight paths. Private, light aircraft is controlled within seven miles of the airport.

There are no airports within 10 miles of McGuire Nuclear Station that handle or expect to handle aircraft heavier than 10,000 pounds.

The other civilian airports within 20 miles of the site handle private light aircrafts which are controlled by FAA Regulations only within two miles of the airport. The larger civilian airports are described below.

Gastonia Airport, located 20 miles south-southwest of the site, has a 3500 foot lighted hard surface runway serving approximately 84 permanently based aircraft. Annual air operations number approximately 75,000 and the maximum weight of the aircraft is approximately 7,000 pounds. No commercial flights other than light aviation air charter are available. A rotating beacon refueling pad and flight school are located at this airport. Major maintenance, charter and air taxi service are also available.

Lincoln Airport, located 19 miles west of the site, has a 5,200 foot paved runway. There are 25 permanently stationed aircraft whose maximum weight is 6,000 pounds and annual operations are approximately 24,000.

Long Island is a restricted airport with a 2,800 foot unpaved runway located 16.5 miles north-northwest of McGuire. There are two aircraft stationed permanently and the air operations total approximately 520 annually.

Lake Norman Airport, located 12.5 miles north of the site, has a 2,400 foot paved and lighted runway, major facilities, and a flight school. There are 30 to 35 private planes permanently based at this facility. The largest of the aircraft weights approximately 7,000 pounds.

Miller Airfield, located 17 miles northeast of the site, has a 2,100 foot unpaved runway. Of the 15 privately owned and permanently stationed aircraft, the largest weighs approximately 10,000 pounds. No maintenance services are provided at this facility, but charger services and a flight school are available.

Spencer Airfield is a private airport with a 2,200 foot unpaved runway. Four aircraft are permanently stationed here and the largest of these is under 3,000 pounds. Approximately 520 air operations occur annually.

Bradford Airfield, located approximately 8.5 miles east-southeast of the site, has a 3,600 foot unpaved runway. There are eight permanently stationed aircraft, all weighing under 3,000 pounds. The annual air operations number approximately 2,600.

Concord regional airport, located approximately 14 miles East of the site, has a 5,500 foot paved runway. Annual operations are approximately 60,000 and maximum aircraft weight capacity is 72,000 pounds.

2.2.2 Descriptions

There are no known products, hazardous to the plant, manufactured or stored within five miles of McGuire Nuclear Station. The major north-south transportation corridors in the vicinity of the site are U.S. 321, located approximately 15 miles west of the site, N.C. 16, located approximately three miles west of the site, and I-77 located approximately five miles east of the site. The major east-west transportation corridors are I-40, located approximately 25 miles north of the site, and I-85, located approximately 12 miles south of the site. N.C. 150, located approximately 11 miles northwest of the site, and N.C. 73, located approximately 0.4 miles south of the site, are primarily used by local residents, commuters, and for recreational access to Lake Norman. These highways are shown on Figure 2-35.

Daily average traffic loads for I-77, N.C. 16, N.C. 150, and N.C. 73 for 1970 through 1974 and 1993 are presented in Table 2-8.

There are no manufacturers or suppliers of hazardous materials within 10 miles of the site. Gasoline and oil tank trucks, with capacities approaching 10,000 gallons for local delivery to service stations, marinas, and homes, and tank trucks with maximum capacities of 500,000 cubic feet of liquid oxygen, shipped every two weeks from Charlotte, North Carolina, to Union Carbide, Reeves Bros., and Florida Steel (see Figure 2-35 for locations) can, at the driver's discretion, be expected to travel via N.C. 73 or I-77.

The United States Department of Transportation (USDOT), North Carolina Department of Transportation (NCDOT), Interstate Commerce Commission (ICC), North Carolina Highway Patrol, United States Bureau of Mines, North Carolina Department of Natural and Economic Resources, Civil Defense agencies, and local shipment companies were contacted by Duke to obtain specific information concerning routes, frequency, types, and amounts of hazardous *materials transported near the McGuire Station. It was found that this or similar information was not maintained by these agencies or companies. The shipment of hazardous materials is, however, regulated by the USDOT with regard to materials loading and unloading (49CFR 173.30 and 49CFR 177, Part B), container requirements (49CFR 173.24 and 49CFR 477.812), quantity limitations (49CFR 173.26), the reporting of accidents (49CFR 177, Part D), and the specification of containers (49CFR 178).*

Based on the regulations noted above and the proximity of alternate major high speed highways bypassing the site, Duke believes that the probability of McGuire Nuclear Station being affected by shipment of hazardous materials on highway N.C. 73 is insignificant.

Transcontinental Pipeline, Inc., has four major pipelines in the vicinity of McGuire Nuclear Station which transport methane. Figure 2-36 shows details for Transcontinental pipelines in the vicinity of the site and also gives pipeline and valve characteristics of the pipes. All isolation valves shown on Figure 2-36 have automatic-rate-of-drop line shutdown devices with line breaker alarms that respond to breaks in magnitude of 25-30 psi per minute drop (Reference 2). If a 25-30 psi per minute drop is experienced, the line breaker shuts the valve instantly (Reference 2). These isolation valves can also be closed manually (Reference 2.

Hedrick Industries operates a quarry, the Lake Norman Plant, which is located approximately 3.8 miles west of McGuire. No explosives are stored at the quarry site. Approximately 4,000 to 5,000 lbs. of ANFO (Ammonium Nitrate and Fuel Oil) are delivered on the day that a shot is to occur and any excess is returned to the manufacturer.

Hedrick also has a permit to operate a quarry near Lucia, approximately 4.3 miles Southwest of McGuire. This plant is not in operation at this time but is expected to be operational in the future.

Drinking water is now supplied by Charlotte Mecklenburg Utility Department.

The activities of the North Carolina Air National Guard, based on Douglas Airport, are described in Section 2.2.1.

2.2.3 Evaluations

Since there are no shipping activities on Lake Norman and the largest vessels on the lake are yachts in approximately 40 foot range, intake structure protection is not required. The icing encountered on a lake in this geographic region is insignificant with regard to intake structure operation. Accidental upstream spills of oil or corrosives would have no effect on either of the two concrete intake structures.

The potential consequences to the McGuire facility from a gas pipeline rupture was considered. Two pipelines, one 36 inch diameter and one 42 inch diameter, located one mile south of the plant, are shown on Figure 2-36. A rupture of the 42 inch diameter pipe was postulated for the accident analysis. A typical gas analysis indicated 96.5 percent methane, 2.0 percent ethane, and other trace gases. The energy content of the gas was 1022 BTU per cubic foot at 60°F and one atmosphere pressure. The density of the gas was .58 that of air (at 60°F). Normal operating pressure of the pipe is about 600 psi with a maximum pressure of 800 psi. Isolation valves are located 15 miles apart (see Figure 2-36). The flammability limits of the gas are five percent to 15 percent by volume (Reference 6). The hypothesized accident was a rupture at the nearest approach to the plant with a leakage rate the same as the maximum flow through the pipe. Plume rise is that predicted by G. A. Briggs, (Reference 3), and dispersion calculations were made according to D. B. Turner (Reference 4). The effects of gaseous explosions were calculated according to the recommendations of lotti, et al.

The potential effects of the gas at the plant were considered first. Under any atmospheric stability and wind condition, the ground level concentrations at the plant are considerably below the flammability threshold.

The consequences of an unconfined in-air explosion was considered next. The worst case conditions are postulated to be Pasquill F stability with a two m/sec wind directed from the rupture to the plant. (Pasquill G was not considered realistic for such an elevated plume.) Conservatively, no initial mixing is assumed, although the actual release of such a high pressure gas would result in considerable dilution. A centerline concentration of the upper flammability limit of 15 percent is estimated to occur 530 meters downwind of the release and the lower limit of five percent 1050 meters from the release. An explosion of all the gas within these flammability limits assumed at a worst 10 percent concentration, would result in a shock wave of 1.3 psi overpressure at the plant.

The effect of a surface blast at the point of rupture was finally considered. The mass of gas participating in such an explosion is impossible to specify. No large structures to confine the gas exist near the postulated rupture; so, most likely, a fraction of the gas would detonate and the remaining fraction of the escaping gas would burn impressively but with no associated pressure wave. An explosion of all the gas between the isolation valves at a uniform concentration of 10 percent by volume represents an extremely conservative worst case. An overpressure of 1.8 psi at the plant would result from such a blast.

The pipeline operators do not contemplate the use of these lines for propane transport (Reference 7). A change to propane transport would require approval by the Federal Power Commission at which time the pipeline company would have to publish a public notice. If a change to propane were considered a hazard to safe plant operations, intervention procedures could then be initiated.

The postulated events of either the detonation of the total explosives to be stored at the existing or proposed quarry sites, or the detonation of a legally loaded truck carrying TNT (Reference 8) at the closet approach point to the McGuire plant on Highway 73, would have no significant impact on McGuire Nuclear Station. There are no significant quantities of dangerous chemicals stored near McGuire.

The only potential fire hazard in the plant vicinity is a brush fire. The plant fire protection system is adequate to prevent any possible damage from a fire to this origin.

Only small quantities of chlorine may be temporarily stored on site since chlorine will not be used for condenser cleaning at McGuire. No individual container on the site will contain more than 150 pounds of chlorine. It is unlikely that leaks from chlorine containers would result in dangerous concentrations in the Control Room; however, the Control Room can be isolated from the outside environment, if necessary, and portable breathing equipment, suitable for protection against chlorine, is available.

The McGuire Plant is not located in any FAA established flight paths. Light private aircraft operates within FAA rulings with respect to altitude and flight paths. The established flight path closest to the site is approximately two miles east of the site.

No high natural-draft cooling towers or other tall structures, such as stacks, are used at the site; therefore, no evaluation has been made for collapse.

2.2.4 References

1. *Letter from Mr. R. C. Birmingham, Airport Manager, Douglas Municipal Airport, March 20, 1975.*

2. *Telephone conversation with Mr. Louis Graham, Senior Engineer, Pipeline Department, Transcontinental Pipeline, Inc., July 23, 1974.*
3. *Briggs, G. A., Plume Rise, U.S. Division of Technical Information, 1969.*
4. *Turner, D. B., Workbook of Atmospheric Dispersion Estimates, U.S. Division of Technical Information, 1968.*
5. *Iotti, R. C., et al, "Hazards to Nuclear Plants from on (or near) Site Gaseous Explosions", Water-Reactor Safety (conf.), USAEC conf. 730304.*
6. *Handbook of Chemistry and Physics, Chemical Rubber co., Cleveland, p. D-82, 1970, 1971.*
7. *Letter from Gene E. Proper, Superintendent - Pipelines, Transcontinental Gas Pipe Line Corporation, Dated 4-1-75.*
8. *Regulatory Guide 1.91, Revision 1, Feb. 1978, Evaluation of Explosions Postulated to Occur on Transportation Routes near Nuclear Power Plants.*

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2.3 Meteorology

Meteorology is evaluated for use in structural design and in consideration of environmental safeguards for gaseous releases. The following paragraphs summarize the atmospheric characteristics pertinent to these design bases.

2.3.1 Regional Meteorology

2.3.1.1 Data Sources

Climatic Atlas of the United States, United States Department of Commerce, Environmental Science Services Administration, Environmental Data Service, June, 1968.

Climate of the States, North Carolina, Climatography of the United States, No. 60-31, United States Department of Commerce, Weather Bureau, February, 1960.

Climate of the States, South Carolina, Climatography of the United States, No. 60-31, United States Department of Commerce, Weather Bureau, December, 1959.

Tornado Occurrences in the United States, United States Department of Commerce, Weather Bureau, Technical Paper No. 20, 1960. Thom, H.C.S., "Tornado Probabilities", *Monthly Weather Review*, October - December, 1963.

Tropical Cyclones of the North Atlantic Ocean, United States Department of Commerce, Weather Bureau, Technical Paper No. 55, 1965.

Mean Number of Thunderstorm Days in the United States, United States Department of Commerce, Weather Bureau, Technical Paper No. 19, September, 1952.

Holzworth, G. C., *Mixing Heights, Wind Speeds and Potential for Urban Air Pollution Throughout the Contiguous United States*, Environmental Protection Agency, January, 1972.

2.3.1.2 General Climate

Synoptic features during winter effect rather frequent alternation between mild and cool periods with occasional outbreaks of cold air. Such intrusions of cold air, however, are modified in the crossing and descent of the Appalachian Mountains. Summers, noted for their greater persistence in flow pattern, experience fairly constant trajectories from the south and southwest with advection of maritime tropical air. Wintertime precipitation occurs primarily in connection with migratory low pressure systems. Recurrence and areal distribution, therefore, are reasonably uniform. Summer rains on the contrary are associated more with showers and thundershowers of the air mass variety, occasioned by intense and uneven heating of the earth's surface.

Severe weather, although infrequent, is most likely from March-October. During this season wind, water and hail damage can result from the thunderstorm, tornado and tropical storm (or hurricane).

2.3.1.3 Severe Weather

Winter conditions as a rule are not conducive to the development of major snow storms. Long-term records for the area show highest 24 hour snowfall near 18 inches. (Winston-Salem, N.C., December, 1930). The ice storm, a much more frequent occurrence, does effect considerable damage over limited areas and can be expected several times a year. Typical accumulations range between one-quarter to one-half inch.

Spring, summer and autumn storms, phenomena of widespread consequence, are the major bearers of severe weather. For the area of North Carolina, South Carolina and their coastal waters, an average of one tropical storm per year and one hurricane every other year has been computed based on a period of record of 63 years (1901-1963). Within this period, seven years were void of any activity while nine years produced a combined total of three storms per year. Highest winds over the area are 110 miles per hour (fastest mile, Cape Hatteras, N.C., September, 1944) along the coast and 80 miles per hour (fastest mile for inland maxima, Wilmington, N.C., October, 1954). Maximum 24 hour rainfalls, again higher for coastal stations, have been recorded near 15 inches along the coast (Cape Hatteras, N.C., June, 1949) to near 9 inches inland (Wilmington, N.C., September, 1938). Figure 2-37 relates tornado frequency to two degree squares for the period 1916-1955. For the site area a total of 50 tornados are shown per two degree square (square area about 125 miles by 125 miles). To put in terms of probability for a point (nuclear station), such a translation predicts a recurrence interval of 4405 years. Thunderstorms with greater frequencies during the summer occur 45-50 days per year (from Charlotte, N.C., period of record 73 years). Associated hail can be expected about one day per year in coastal areas and one or more days per year over inland areas from the period of record 1955-1967 (Reference 3).

The tornado parameters and tornado frequency values used in the probabilistic tornado risk analysis (TORMIS) described in Section 3.5.2.8.1 are found in Reference 5.

Meteorological conditions assumed for design bases are addressed in Section 3.3.2.1 for tornado loadings and in Section 3.8.1.4 for general wind and snow loadings. Criteria for design tornados include a rotational speed of 300 mph, a translational speed of 60 mph and a vacuum pressure differential of 3 psi in 3 seconds. Design speed for general wind loading is 95 mph (fastest mile). Snow loading for design purposes is 20 pounds per square foot.

Air pollution over the Carolinas is of greatest potential during the fall. An average of ten episode - days per year has been computed for a period of five years (from upper air observations at area Weather Service Stations, i.e., Athens, Georgia; Greensboro, N.C.; Cape Hatteras, N.C. and Charleston, S.C.).

2.3.2 Local Meteorology

2.3.2.1 Data Sources

Climatic Atlas of the United States, United States Department of Commerce, Environmental Science Services Administration, Environmental Data Service, June, 1968.

Climate of the States, North Carolina, Climatography of the United States, No. 60-31, United States Department of Commerce, Weather Bureau, February, 1960.

Local Climatological Data, Annual Summary with Comparative Data, North Carolina, United States Department of Commerce, National Oceanic and Atmospheric Administration, Environmental Data Service, 1971.

2.3.2.2 Normal and Extreme Values of Meteorological Parameters

Table 2-9 depicts normal and extreme values for the following parameters: temperature, rain, sleet and snow, fog, relative humidity, dew point and wind direction and speed.

Thunderstorm occurrence by season is: 11 for spring (March-May), 29 for summer (June-August), 5 for fall (September-November) and 1 for winter (December-February). (Reference 4)

2.3.2.3 Potential Influence of the Plant and Its Facilities on Local Meteorology

Consideration has been given to possible environmental effects associated with heat dissipation from the cooling pond (Lake Norman, vicinity of McGuire Nuclear Station). A review of the literature and operating experience to date would suggest that effects of fogging and icing are minimal for the properly designed cooling pond. Neither increased density of natural fog nor the occurrence of "steam fog" is expected to penetrate farther than the downwind periphery of the cooling pond area. Highest frequencies of both natural fog and steam fog occur during winter, giving rise to some potential for contact icing along downwind shorelines. No significant icing, however, has been observed at existing Duke Power plants, where cooling water is circulated via the cooling pond.

2.3.2.4 Topographic Description

Figure 2-38 and Figure 2-39 portray in plan view and vertical cross section the surrounding topography and relief to five miles.

Minor channeling can be expected over the river valley but as evidenced in Figure 2-39 appreciable constrictions do not influence such trajectories.

2.3.3 Onsite Meteorological Measurements Program

2.3.3.1 May 1, 1970 - May 31, 1974

Onsite meteorological measurements were made for the preliminary study period of October, 1970 - October, 1971 for wind direction and speed, horizontal wind direction fluctuation, temperature and vertical temperature gradient. Actual data was recorded for the period May 1, 1970 - May 31, 1974 with measurements from instruments described below.

Relative positions of instruments with respect to station yard, noted in Figure 2-41, are SE at 500 feet and WNW at 3400 feet, respectively, for measurements of wind and temperature. Relative elevations of both surface levels and instrument levels are depicted in Figure 2-42.

Wind measurements were made with the Packard Bell Model W/S 101B Series Wind Direction - Speed system with starting thresholds of 0.7 and 0.6 miles per hour for direction and speed respectively. Temperature and delta temperature measurements were made with the Leeds and Northrup 8100 Series 100 Ohm Resistance Temperature Devise with Package Bell Model 327 Thermal Radiation Shields. Wind direction and speed were recorded in an instrument shelter on Esterline Angus Model A 601 C Strip Chart Recorders with a system accuracy of ± 5.4 degrees for direction and ± 0.45 miles per hour for speed. Temperature and delta temperature were recorded on the Leeds and Northrup Speedomax W Recorder with a system accuracy of ± 1°F for temperature (at 30 foot level) and ± 0.5°F for delta temperature (130 foot level referenced to the 30 foot level).

2.3.3.2 January 29, 1976 - December 31, 1980

Wind and temperature measurements were resumed at a permanent meteorological facility on January 29, 1976 (See Figure 2-41). Instrument heights remained unchanged. A new temperature system, however, was installed with an accuracy of ±0.85 degrees F for temperature and ± 0.18 degrees F for delta temperature.

With regard to instrument exposure at the permanent facility, consideration has been given to the influence of local topographic features on air flow characteristics. Building and dam elevations are shown in Section 2.4.10. Heights above station grade are nominally 20, 100 and

140 feet, respectively, for the dam, turbine and reactor buildings. Linear distances from the permanent facility to these features are depicted in Figure 2-41 as approximately 350 feet to the dam, 700 feet to the reactor buildings and 500 feet to the turbine buildings. Guidance from NRC regulatory Standard Review Plan, Section 2.3.3 was also incorporated with respect to the distance criterion of five height lengths. No serious perturbation then is expected in background characteristics.

The following calibration-maintenance schedule is extracted from the Duke Power Company manual, *Schedule and Procedures for Calibration and Maintenance of Meteorological Instruments*, as pertained to the care of these instruments.

“Schedule for Calibration and Maintenance of Meteorological Instruments”

Weekly

The following field checks are performed each week before old charts are replaced and pens re-inked:

1. Wind Direction
    a. Recorder time accuracy
    b. Recorder zero
    c. Translator zero
    d. Translator full scale
    e. Continuity (dirty potentiometer)
    f. Gross linearity
    g. True direction
2. Wind Speed
    a. Recorder time accuracy
    b. Recorder zero
    c. Translator zero
    d. Hand-held annemometer (certified by Kahl Scientific Instrument Corp.)
3. Temperature and Delta Temperature Recorder time accuracy

Quarterly

The following laboratory checks are performed each quarter:

1. Wind Direction
    a. Refined linearity
    b. Transmitter starting torque
2. Wind Speed
    a. Electronic simulation to translator (over total range of speeds)
    b. Transmitter starting torque
    c. Transmitter shaft end play

Semiannually

The following field checks are performed twice each year:

1. Temperature and Delta Temperature
   a. Electronic simulation to transmitter (over total range of temperature)
   b. Comparison with precision mercurial thermometers

All data are reduced manually and keypunched for storage on magnetic tape. Procedures for data reduction are as follows:

Procedures for Reduction of Meteorological Data

General

Letter transmitted with charts is checked for any errors noted in the weekly calibration procedure. Correction curves for errors noted are to be obtained from the Meteorologist. Upon reduction of corrected data, correction curves are initialed by the reader and returned to the Meteorologist.

Wind

Wind direction and speed are averaged over 30 minute intervals preceding each hour and logged on the hour. Wind range is measured during 30 minute intervals preceding each hour and logged on the hour. Wind direction and speed are averaged with a transparent straight edge making a visual integration by equal area apportionment. Wind range is measured by counting direction intervals between extreme directions, eliminating monentary peaking.

Temperature

Temperature and delta temperature are averaged over one hour intervals, 30 minutes before and after each hour and logged on the hour. Temperature corresponds to absolute temperature trace. Delta temperature is delta reading between the lowest and highest sensor (100 foot separation). Both temperature and delta temperature are averaged by equal area technique employed in reduction of wind data.

2.3.3.3 January 1, 1981 - May 23, 1983

Meteorological measurements were taken with a new monitoring system for the period January 1, 1981 to May 23, 1983. This system, manufactured by Meteorological Research Institute (MRI), included all previous meteorological channels and also dew point temperature monitoring at 805 ft. elevation. Data collection during this time period was from a digital tape recording system and lost data was supplemented from manual chart reductions. Instrument calibration frequency for this period was in accordance with Technical Specification Section 4.3.3.4 and Table 4.3-5.

2.3.3.4 May 28, 1983 – September 2, 1998

Beginning May 28, 1983, operational measurements consist of near real-time digital outputs in addition to the previously described analog system. An entirely new set of instrumentation has been installed including the measurement of rainfall (at ground level). The mid level delta-temperature measurement point is discontinued.

Instrument specifications for operational measurements are:

1. Wind Direction
   a. Manufacturer Teledyne Geotech

   b. Time-averaged digital accuracy ±2 degrees of azimuth
   c. Time-averaged analog accuracy ±6 degrees of azimuth
   d. Starting threshold 0.3 m/sec at 10 degrees initial deflection
   e. Damping ratio 0.4 at 10 degrees initial deflection
   f. Distance constant 1.1m

2. Wind Speed
   a. Manufacturer Teledyne Geotech
   b. Time-averaged digital accuracy ±0.27 m/sec for speeds <26 m/sec
   c. Time-averaged analog accuracy ±0.40 m/sec for speeds <26 m/sec
   d. Starting threshold 0.3 m/sec
   e. Distance constant 1.5 m
3. Temperature
   a. Manufacturer Teledyne Geotech
   b. Time-averaged digital accuracy ±0.1 degrees C
   c. Time-averaged analog accuracy ±0.2 degrees C
4. Delta Temperature
   a. Manufacturer Teledyne Geotech
   b. Time-averaged digital accuracy ±0.09 degrees C
   c. Time-averaged analog accuracy ±0.09 degrees C
5. Dew Point*
   a. Manufacturer EG&G
   b. Time-averaged digital accuracy ±0.3 degrees C
   c. Time-averaged analog accuracy ±0.3 degrees C
6. Precipitation
   a. Manufacturer Teledyne Geotech
   b. Digital accuracy ±6% of total accumulation at 15 cm/hr
   c. Analog accuracy ±6% of total accumulation at 15 cm/hr
   d. Resolution 0.25 mm

Maintenance, calibration and repair procedures, as well as logs, are available at the McGuire site. The following field checks are performed weekly by plant personnel:

1. Wind Direction
   a. Recorder Time Accuracy
   b. Recorder Zero
   c. Translator Zero
   d. Translator Full Scale

2. Wind Speed
   a. Recorder Time Accuracy
   b. Recorder Zero
   c. Translator Zero
   d. Translator Full Scale
3. Air/Delta Temperature
   a. Recorder Time Accuracy
   b. Aspirator Air Flow

Sigma theta (fluctuation of lower level wind direction) data collection began November 7, 1984.

*The dew point temperature monitoring system was discontinued on November 3, 1987 due to no regulatory requirements for the collection of dew point temperature data at the McGuire site.

Both upper and lower level wind direction systems were upgraded from potentiometric sensors to resolver sensors on November 4, 1987. This greatly improved the performance and reliability of the wind direction system.

The present meteorological monitoring system complies with the recommendations of NRC Regulatory Guide 1.23.

Near real-time digital outputs of meteorological measurements are summarized for end-to-end 15 minute periods for use in a near real-time puff-advection model to calculate offsite dose during potential radiological emergencies. The Operator Aid Computer (OAC) System computes the 15 minute quantities (except sigma theta) from a sampling interval of 60 seconds. It calculates 15 minute average values for high and low level wind direction and speed; 15 minute averages are also calculated for delta temperature and ambient temperature. Total water equivalence is computed for precipitation. A 15 minute standard deviation of low level wind direction (sigma theta) is calculated by a field unit with a sampling interval of one second. All 15 minute values are stored with a 24 hour recall. Permanent archiving of data from the digital system is made by combining the 15 minute quantities into one hour values.

A data logger (with modem) was installed at the meteorological tower site on October 7, 1997 to monitor all meteorological data channels. This data logger allows meteorological data access via station telephone circuit and is independent of the primary OAC data collection system.

2.3.3.5 September 2, 1998 to Present

On September 2, 1998, a new meteorological tower and new meteorological instrumentation was made operational. This new tower and meteorological system are located approximately 1500 feet North North-East of station center. This new tower is a guyed, 60 meters tall tower having two instrument monitoring levels which use an instrument boom elevator for locating the sensors. The two instrument levels are 60 meters and 10 meters above ground grade. The tower base elevation is 768.08 feet msl.

Each monitoring level consists of a wind speed sensor, wind direction sensor and an air temperature sensor. A temperature differential value is determined from the 60M and 10M air temperature readings. Also, a precipitation gauge is located approximately 100 feet NNE from the tower base at an elevation of 1 meter above ground grade.

All meteorological parameters are calibrated semi-annually. All tower sensors (wind speed, wind direction and air temperature) are replaced semi-annually with newly certified sensors. The precipitation is field calibrated.

The meteorological system is serviced weekly to ensure system operational accuracy. Typically, zero and span checks are performed for each meteorological parameter along with recorder time accuracy.

This new meteorological tower and meteorological instrumentation comply with the recommendations of NRC Regulatory Guide 1.23, Rev. 0, 1972.

2.3.4 Short Term (Accident) Diffusion Estimates

2.3.4.1 Objective

Data collected onsite from October 17, 1970 - October 16, 1971 provides the basis for diffusion estimates as they relate to inadvertent release of radioactive material. Table 2-10 displays the joint frequencies of wind direction and speed by atmospheric stability type as they were observed onsite. Figure 2-43 and Figure 2-44 represent distributions of hourly dispersion factors at the Exclusion Area boundary (2500 feet) and the Low Population Zone boundary (29,000 feet) respectively, for the period of record noted. Percentile levels are 6 to 96 inclusive for the Exclusion Area boundary distribution and 4 to 96 inclusive for the Low Population Zone boundary distribution. Frequencies result from cumulative summation of percentage values in Table 2-10 in decreasing order of relative concentration computed for selected wind speed class intervals. All calm wind occurrences are considered in the distribution. Data recovery for this period was 89.34 percent of total observations. Estimates of diffusion for longer time periods (up to 30 days) assume the following combination of atmospheric conditions (as per AEC Regulatory Guide 1.4, "Assumptions Used for Evaluating the Potential Radiological Consequences of a Loss of Coolant Accident for Pressurized Water Reactors"):

Time Period	Atmospheric Condition
0-8 hours	Pasquill Type G, wind speed 0.9 meter/sec, uniform wind direction
8-24 hours	Pasquill Type F, wind speed 1 meter/sec, variable wind direction within a 22.5 degree sector
1-4 days	1. 40 percent Pasquill Type D, wind speed 3 meters/sec 2. 60 percent Pasquill Type F, wind speed 2 meters/sec 3. Wind direction - variable within a 22.5 degree sector
4-30 days	1. 33.3 percent Pasquill Type C, wind speed 3 meters/sec 2. 33.3 percent Pasquill Type D, wind speed 3 meters/sec 3. 33.3 percent Pasquill Type F, wind speed 2 meters/sec 4. Wind direction - 33.3 percent frequency in a 22.5 degree sector

Short term diffusion estimates are included in Table 2-11 a composite summary for all time periods.

2.3.4.2 Calculations

Hourly dispersion factors are developed by computation from the widely accepted Pasquill-Gifford gaussian equation (Reference 1):

$$X/Q = \frac{1}{\bar{u}(\sigma y \sigma z \Pi + CA)}$$

where

X/Q	= normalized concentration at plume centerline (sec/m^3)
$\bar{u}$	= mean wind speed through the vertical extent of the plume (m/sec)
σy	= crosswind concentration distribution standard deviation (m)
σz	= vertical concentration distribution standard deviation (m)
C	= containment structure shape factor = 0.5
A	= cross-sectional area of containment structure normal to the wind - 1616m^2

Dispersion parameters are selected as indexed by vertical temperature gradient according to the following schedule:

Stability Class	**Vertical Temperature Gradient**
G	greater than +2.2°F in 100 feet
F	+0.9 to +2.2°F in 100 ft
E	-0.3 to +0.8°F in 100 ft
D	-0.8 to -0.4°F in 100 ft
B-C	-1.0 to -0.9°F in 100 ft
A	less than -1.0°F in 100 ft

Note: The small range of temperature gradient defining stability categories B and C precludes a differentiation of these stabilities from the field data. In all such cases, the plume spread parameters for C stability (less unstable than B) are used.

Plume spread-distance relationships assumed are those suggested by D. B. Turner. (Reference 2)

2.3.5 Long Term (Routine) Diffusion Estimates

Data currently in use can be found in the Offsite Dose Calculation Manual (ODCM).

[HISTORICAL INFORMATION NOT REQUIRED TO BE REVISED]

2.3.5.1 Objective

Onsite data from October 17, 1970 - October 16, 1971 also provide the basis for diffusion estimates appropriate in consideration of routine operation and release of radioactive material. Average dispersion factors are computed, covering the stated period of record, for angular intervals of five degrees at ten distances (to 50 miles), utilizing a computer program to store and accumulate successive hourly values. The areal distribution of annual average relative concentrations is portrayed in Table 2-13 (Sheets 1-10) and Figure 2-46, Figure 2-47, and Figure 2-48.

2.3.5.2 Calculations

Successive hourly values are calculated to crosswind distances of ± 20 degrees from observed wind directions. Points in the computational grid beyond ± 20 degrees for any one hour are assumed at zero relative concentration for that hour. A gaussian form is again assumed with computation from:

$$X/Q = \frac{1}{\bar{u}(\sigma_y \sigma_z \Pi + CA)} \exp[-1/2(\frac{y}{\sigma y})^2]$$

where y = crosswind distance from plume centerline (m)

A building wake factor is included since lateral spread rate is determined for each hour as a function of stability class. Dispersion parameters, selection criteria and plume spread relationships are also identical to those used for short term estimates. Calm hours are included in the averages by assuming persistence in the wind direction last observed.

2.3.6 Summary Of Diffusion Estimate

Table 2-11 depicts dispersion factors for each type of release at appropriate distances and percentile values (probability of occurrence).

STAR processing of Charlotte Airport data has been accomplished for the period of onsite data, (October 17, 1970 - October 16, 1971) in addition to a five year period, (January, 1969 - December, 1973). See Table 2-12. Comparison of wind direction and speed, and of stability type (Pasquill designation) forms the basis for judging the representativeness of data for the year October 17, 1970 - October 16, 1971, with regard to long-term conditions (e.g., five year period). Consideration of wind speed by stability type for the two periods shows a lower speed in general for the period October 17, 1970 October 16, 1971; the occurrence of calms and winds less than 4 knots are up four percentage points from 15% for the period January, 1969 - December, 1973. A slight shift in stability frequencies is noted for the period October 17, 1970 - October 16, 1971: "G" increases, "F" and "E" decrease, "D" increases and "C", "B", and "A" decrease.

Some change in wind direction frequencies, also minor, is noted for the period October 17, 1970 - October 16, 1971: easterly, westerly and southwesterly directions increase while southerly, northerly and northeasterly directions decrease. On balance, the period is taken as reasonably representative of long-term conditions in the vicinity of the site with some conservatism with respect to accident relative concentration estimates as indexed by the joint distribution of wind direction and speed by stability type.

An additional year of onsite data have been collected using a measurement system which conforms to the recommendations of Regulatory Guide 1.23. The location of instrumentation is shown in Figure 2-41 marked permanent meteorological facility. Other discussion relating to instrument accuracy and sensitivity at this facility is included in Section 2.3.3. Dispersion estimates have been developed from this data base and are presented in the following summary.

Table 2-14 displays the joint frequencies of wind direction and speed by atmospheric stability type as they were observed on site for the period February, 1976 - January, 1977. Figure 2-49 represents the distribution of hourly dispersion factors at the Exclusion Area Boundary (2500 feet). Frequencies result from cumulative summation of percentage values in Table 2-14 in decreasing order of relative concentration computed for selected wind speed class intervals. All calm wind occurrences are considered in the distribution. Data recovery for this period was 94% of total observations.

Annual average dispersion factors were also calculated for the period of record using the calculational model in Section 2.3.5. The resulting areal distribution of annual average relative concentration is portrayed in Table 2-15.

2.3.7 References

1. *Meteorology and Atomic Energy, 1968, United States Atomic Energy Commission, Division of Technical Information, July, 1968.*
2. *Workbook of Atmospheric Dispersion Estimates, D. Bruce Turner, United States Department of Health, Education and Welfare, 1969.*
3. *"Severe Local Storm Occurrence, 1955-1967", U.S. Weather Bureau Technical Memorandum WBTM-FCST #12, September, 1969.*
4. *Mean Number of Thunderstorm Days in the United States, U.S. Department of Commerce, Weather Bureau, Technical Paper #19, September, 1952.*
5. *MCC-1139.01-00-0298, "MNS Tornado Missile TORMIS Analysis".*

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2.4 Hydrology

[HISTORICAL INFORMATION NOT REQUIRED TO BE REVISED]

> ***Note:***
>
> *This section contains references to Former FSAR Appendices 2A through 2H. These appendices contain historical site data needed by the NRC reviewers prior to Construction. This historical information is now archived as hard copies stored in a fireproof cabinet under the control of the Regulatory Compliance Group for the lifetime of the plant.*

William B. McGuire Nuclear Station is located immediately east of Duke's Cowans Ford Dam and Hydroelectric Station and south of Lake Norman. Plant grade of safety-related structures is E1. 760.0. The safety-related facilities, systems, and equipment are protected against floods up to computed maximum water level, elevation 773.01 msl using 40 mph sustained wind, or E1. 774.75 when hurricane winds of 96 mph are considered. Determination of the maximum static flood level, elevation 767.9, included consideration of dam failures from seismic and hydrologic causes. The study of flood analyses for Lake Norman is given in Former Appendix 2F.

The nearest major user of groundwater for public use is in Cornelius, located approximately six miles northeast of the plant. The nearest industrial user of surface water for human consumption is located approximately 17.8 river miles downstream of the plant. The Charlotte Municipal Water Intake is located 11.2 river miles downstream of the plant. Ground and surface water sources are not affected from either normal or postulated accidental radioactive releases. As discussed in Section 11.6, periodic samples will be taken of various private and municipal water supplies to insure that they are not affected by the operation of the station.

2.4.1 Hydrologic Description

2.4.1.1 Site and Facility

The location and description of the site are given in Sections 2.1.1 and 2.1.2. The station yard at elevation 760 feet ms1 is protected upstream from the maximum flood level 767.9 feet ms1 and maximum wave effect of an additional 5.11 feet by an earthen embankment built to elevation 780 feet msl. The downstream side of the plant yard does not require additional protection since the maximum discharge through Cowans Ford, based on the flood studies of Former Appendix 2F, is 365,900 cfs and results in a maximum tailwater elevation of 698.5 msl. This is 61.5 feet below the McGuire yard elevation of 760.0 msl.

The intake structure, discharge structure and earth dike are designed to withstand the maximum flood level with a 5.11 foot wind generated wave superimposed. The only significant alterations of the site which will have a bearing on the site hydrology are the construction of the Standby Nuclear Service Water Pond Dam and Waste Water Collection Basin Dam. Hydrologic considerations for the Standby Nuclear Service Water Pond Dam are described in Former Appendix 2G. The Waste Water Collection Basin Dam, a homogeneous earth fill dam, is located approximately 2400 feet southwest of the center of the plant. The dam is designed for normal loadings and has a crest width of 20 feet at elevation 697.0 msl. The dam has a maximum height of 32 feet, a length of 190 feet, and a freeboard of 7 feet at static pond level. The upstream slope is protected by riprap above elevation 685.0 msl. The downstream slope is protected by grass with its toe riprapped. Both slopes are 2 horizontal to 1 vertical. The outlet works consist of a 66 inch pipe through the dam that discharges into a paved ditch downstream of the dam. Figure 2-4 shows the location of both dams.

All category 1 safety-related structures located at the plant are listed in Table 3-1. The structures from Table 3-1 which are potentially affected by the water levels due to Probable Maximum Precipitation (PMP) or reservoir floods are as follows:

1. *PMP on local plant yard or floods and wind wave activity on Lake Norman-*

   Auxiliary Building

   Reactor Building

   Refueling Water Storage Tank Foundations

2. *Standby Nuclear Service Water Pond floods and wind wave activity-*

   Standby Nuclear Service Water Pond Dam and Overflow Spillway Structure

2.4.1.2 Hydrosphere

The principal stream which drains the site is the Catawba River. The Catawba River begins at the Blue Ridge Divide near Old Fort, North Carolina, and flows in an easterly direction to a point near Millersville, North Carolina. It then flows in a southerly direction and joins the Wateree River near Camden, South Carolina. The Catawba has a length of approximately 240 miles and a drainage area of approximately 4,750 square miles.

Lake Norman and Cowans Ford Dam are part of Duke's Catawba River hydroelectric system containing 11 hydroelectric reservoirs and dams, and extending along approximately 221 miles of the Catawba River. Lake Norman forms the tailwater of Lookout Shoals Dam, located 34 miles upstream from Cowans Ford, and Mountain Island Lake forms the tailwater for Cowans Ford. Mountain Island Dam is located 15 miles downstream from Cowans Ford.

Shown in Plate VII, Former Appendix 2F, is a map reflecting the major hydrologic features of the region.

A USGS Gauging Station was located 30 miles upstream from the present location of Cowans Ford Dam near Catawba, North Carolina, until it was inundated by the water of Lake Norman in 1962. The average discharge past this point for a period of record of 30 years and a drainage area of 1,535 square miles was 2,337 cfs. The maximum flow recorded at this gauge was 177,000 cfs (783.29 ft. ms1) on August 14, 1940, and the minimum flow of 85 cfs occurring on September 15, 1957. On July 16, 1916, the river reached a known flood stage of 44.1 feet (790.59 ft. msl). It has been estimated that this storm produced a flow of 199,500 cfs at the Cowans Ford Dam site on July 17, 1916. The average flow at the Cowans Ford site is approximately 2,670 cfs.

Figure 2-50 indicates the raw water sources in the area (reservoir, stream and groundwater), the location of the raw water intakes and distance of these intakes from the site. Table 2-16 lists the owner, location and use rate of water from Lake Norman through the Wateree sub-basin. Groundwater users of the immediate area are tabulated in Former Appendix 2B.

Lake Norman, with a volume of 1,093,600 acre-feet, and an average discharge of 2,670 cfs has an average retention time of 206 days. The retention time of the cooling water effluent from McGuire Nuclear Station is less because the point of discharge is located in the southern end of the lake near the Cowans Ford hydro intake. The McGuire cooling water is discharged approximately 2.5 miles from the Cowans Ford intake and must circumvent a peninsula having a length of 2,700 feet before turning in the direction of the hydro intake.

Non-radioactive liquid effluent from the Conventional Waste Water Treatment System is discharged to the Catawba River downstream of Cowans Ford Dam.

2.4.2 Floods

2.4.2.1 Flood History

The Catawba River above Lake Norman has four reservoirs as shown on Plate VII of Former Appendix 2F. Table 2-17 shows the normal operating level and the full pond levels of each upstream reservoir. Since a large storage volume exists, the floods of record have been modified.

The maximum estimated flow at the site was 199,500 cfs (based on gauge height at USGS Gauge Station 2-1425, Catawba, North Carolina) on July 17, 1916. Table 2-18 gives estimated peak flow frequency at McGuire site in Catawba River.

2.4.2.2 Flood Design Considerations

Flood levels for the site are based on the following criteria:

1. *Probable Maximum Flood (PMF) Resulting from probable maximum precipitation in the drainage area.*
2. *A Standard Project Flood (SPF) passing through Lake Norman combined with seismic failure of one of the upstream dams. The Standard Project Flood is considered equal to one-half of the PMF.*

The station yard is protected by an earthen dike with crest elevation 780 feet msl and is not subject to flooding caused by the most severe assumed flood condition combined with the highest wave effort. Details of the flood studies are presented in Former Appendix 2F.

The effect of wind on wave height and runup on the McGuire site have been evaluated and found to be within the limits set forth in the design of the earthen dike. Further details are discussed in Sections 2.4.3.6 and 2.4.5.

2.4.3 Probable Maximum Flood (PMF) On Streams And Rivers

The PMF is calculated by taking the probable maximum precipitation as described in Section 2.4.3.1, and placing it over each of the reservoir drainage areas upstream of Lake Norman. Each flood was routed through the Catawba River system to determine the most critical position for producing the maximum reservoir elevation at the site.

A PMF was derived from past meteorological studies as detailed in Former Appendix 2F.

The PMF was routed through the Catawba River system by means of a flood routing program set up for an IBM 360-65 computer. The resulting hydrographs and reservoir elevations in Lake Norman are shown on Plate IV of Former Appendix 2F.

The hydrologic analyses and hydraulic design criteria related to the Stand by Nuclear Service Water Pond are presented in Former Appendix 2G.

There are no significant tributaries flowing into Lake Norman within 15 miles of the McGuire site. Between 18 miles upstream of the McGuire site and Lookout Shoals Dam, 34 miles upstream there are four significant creeks flowing into Lake Norman, all of which having a relatively small drainage area when compared with the surface area of Lake Norman. A local PMF over one of these areas or all of them would not cause the water level of Lake Norman to exceed the level determined by the PMP over the entire region in which these drainage basins already had been included.

2.4.3.1 Probable Maximum Precipitation (PMP)

As a guide to the determination of time and area rainfall distribution pattern over the Catawba River basin, the storm of July 13-17, 1916 was selected and the following adjustments made:

1. *Rainfall depth-duration values were distributed in accordance with that of the 1916 storm. The adjusted July, 1916 storm most closely approximates the theoretical Probable Maximum Precipitation depth-area-duration values for the Lake Norman drainage area.*
2. *Storm position was transposed over a limited distance within Catawba River basin to produce a maximum concentration of precipitation over a selected area.*
3. *Precipitation amounts were increased 40 percent.*

Table 2-19 details the hourly incremental rainfall excess for a 54 hour period for the Probable Maximum Precipitation.

A detailed hydrologic study of the McGuire site is presented in Former Appendix 2F.

Temporary flood barriers will be installed during postulated beyond design bases flood events in doors to protect external flood water from entering the safety related structures.

2.4.3.2 Precipitation Losses

The topography of the Catawba River basin is gentle to moderate sloping toward the river in a southeasterly direction. The soil designation is Ultisoil U5-3, made according to the National Cooperative Soil Survey Classification of 1967 (Reference 1). These soils are low in bases and have subsurface horizons of clay accumulation; usually moist, but during the warm season of the year some are dry.

Initial loss for conditions usually preceding major floods in humid regions normally range from about 0.2 to 0.5 inches and is relatively small in comparison with the flood runoff volume. A value of 0.5 inch was taken for initial loss in this study.

Infiltration rates vary throughout the storm period from a high rate at the beginning to a relatively low and uniform rate as the precipitation continues. It is common practice to assume infiltration rates from 0.05 to 0.10 inch per hour, depending upon antecedent field moisture conditions, slope, and soil type. A hydrologic study made by the Corps of Engineers of the Saluda River Basin above Chappels, South Carolina, indicate an infiltration rate of 0.12 inches per hour (Reference 2). The Saluda River Basin is 1,290 square miles and lies approximately 80 miles southwest of the Catawba River. The topography, soil group, and climate of both basins are very similar. Based upon this study, an infiltration rate of 0.10 inches per hour was used.

2.4.3.3 Runoff Model

Each of the reservoir drainage areas was divided into subareas depending on the number of larger tributary streams flowing into each reservoir as shown on Plate 1 of Former Appendix 2F. Unit hydrographs of these areas were developed and the probable maximum precipitation was applied to these hydrographs with appropriate losses. This method is further detailed in Section 5 of Former Appendix 2F.

2.4.3.4 Probable Maximum Flood Flow

A series of computer simulations were performed, using different storm center positioning and different upstream dam failure sequences. The probable maximum flood flow and corresponding flood elevation at Cowans Ford (Lake Norman) are given on Plates IV and VI of

Former Appendix 2F. Further details of this study are presented in Section 5 of Former Appendix 2F.

2.4.3.5 Water Level Determination

Routing of the PMF through the Catawba River system results in the hydrographs and reservoir elevation at Lake Norman as shown on Plate IV of Former Appendix 2F. As described in Section 4 of Former Appendix 2F, in routing the PMF, discharges from generation of power are assumed to continue. In storm No.14A, the discharges continued unless reservoir levels overtop bulkheads protecting powerhouses or switchyards, at which time the hydroelectric units are considered to be closed for the remainder of the storm period. Based on these considerations, Lake Norman surcharge for PMF will be to elevation 767.9 msl. This results in a freeboard of 2.1 feet on the intake structure and 7.1 feet on the dam embankment near the McGuire site. A tabulation of results of the different storms is given on Plate VI Former Appendix 2F.

2.4.3.6 Coincident Wind Wave Activity

In Former Appendix 2F, Section 7, a total wave height on the embankment was calculated at 6.0 feet. This height superimposed atop the PMF elevation 767.9 feet msl yields an elevation of 773.9 feet msl allowing 6.1 feet of freeboard on the McGuire embankment.

In addition to these computations for wave height and runup based on methods from Moliter 7 discussed in Section 7 of Former Appendix 2F, an alternate procedure for determining wave heights, runup, and the associated static and dynamic forces was performed. This analysis uses the procedures and techniques from the Corps of Engineers' "Shore Protection Manual" (Reference 8) Using this Reference, wave affects were analyzed for a stated overland wind speed from the most critical direction coincidental with the Probable Maximum Flood (PMF) elevation at 767.9 feet msl for Lake Norman.

The Cowans Ford Dam and the McGuire Intake Structure, which are situated at the most southern portion of Lake Norman, are both subject to the same wind waves as shown by Figure 2-51. Although neither of these structures are safety-related (see Section 2.4.1.1), and their failure would not endanger the plant, they were analyzed in order to determine their ability to withstand the design wave forces. The design waves for Lake Norman are generated by a 40 miles per hour sustained overland wind over the entire length of the effective fetch. The maximum fetch for Lake Norman is 5.30 miles, and the effective fetch is 2.04 miles as calculated on Figure 2-52. Table 2-20 and Table 2-21 give the resulting data for both the Cowans Ford Dam and the McGuire Intake Structure for the 40 mph design waves. Data for hurricane waves as discussed in Section 2.4.5.4 is also listed on these tables. The final results of the analysis are presented in Section 2.4.5.8. The results of these calculations confirm that both Cowans Ford Dam and the McGuire Intake Structure will be safe from any damaging effects due to wave forces.

2.4.4 Potential Dam Failures (Seismically Induced)

The effects of the seismic failure of upstream dams coincident with the SPF has been evaluated and the results are presented on Plate VI of Former Appendix 2F. As illustrated in the plate, the highest water surface elevation is 762.6 msl or 5.3 feet below the PMF level at Cowans Ford.

In the event of the highly improbable failure of Cowans Ford Dam, the normal source of condenser cooling water and nuclear service water will be lost. However, as described in Section 9.2.5, the 35 acre Standby Nuclear Service Water (SNSW) Pond provides cooling water *required for heat dissipation for an accident (LOCA) in one unit and for a safe shutdown of the second unit.*

2.4.4.1 Reservoir Description

Five reservoirs on the Catawba River from the headwaters to and including Cowans Ford Dam influence conditions at McGuire Nuclear Station. The reservoirs and a brief description of each in order of its location from the headwaters are presented in Section 2 of Former Appendix 2F. Plates VII through XVI of Former Appendix 2F give additional information on drainage areas above the reservoirs and a description of the structures. Ownership, date first operated, seismic design criteria and spillway design criteria are presented in Table 2-22.

2.4.4.2 Dam Failure Permutations

The location of all dams and reservoirs on the Catawba River whose failure would influence conditions at the McGuire Nuclear Station site are shown on Plate VII of Former Appendix 2F.

Multiple failures of two or more consecutive dams from a single seismic event are highly unlikely. However, such an event will not result in a higher pond elevation at the site than due to possible failure of only one of the dams.

The minimum distance between two upstream dams on the Catawba River is 10 miles (Plate VII, Former Appendix 2F). A multiple failure of two nearest dams will, therefore, result in two smaller flood waves passing through the site in quick succession. By the time the effect of failure of the upper dam reaches the site, peak flow due to the breach in the lower dam would have passed the site, unless the loss of the upper dam causes the loss of the lower dam. The duration of flood at the site would be longer in case of a simultaneous failure, but the peak flood elevation would not be higher than those given on Plate VI, Former Appendix 2F.

2.4.4.3 Unsteady Flow Analysis of Potential Dam Failures

All of the dams above Cowans Ford are composed of part earth embankment and part concrete gravity type structures. In the event of a seismic condition of sufficient magnitude as to cause structural failure to the dams, it was assumed that where the structure was composed of both earth embankment and concrete sections, the weaker portion would be the earth embankment and its failure could be anticipated. The discharge from an abrupt earth embankment breach was computed from an equation by Snyder: (Reference 3)

$$Q\,max = 0.29\, g^{0.5} K^{0.28} W_b D_b^{1.5}$$

where:

Q max *is the peak discharge at the dam in cfs*

g *is the acceleration of gravity.*

W_b *is the width of breach in feet.*

D_b *denotes the depth of water above the breach in feet.*

K $= \frac{W_d}{W_b} \times \frac{Y_o}{D_b}$

W_d *is the width of the dam in feet.*

Y_o *is the depth of water before the breach in feet.*

1. *Earth Embankments*

   As described in Section 6 of Former Appendix 2F, the initial breach in an earth embankment was assumed to be 300 feet in length. In addition, the erosion rate of the exposed embankment faces was taken as 1.4 feet per minute. All of the earth embankments have been constructed on bedrock surface or on a shallow overburden; it was, therefore, assumed that shortly after an embankment breached, the invert would scour to bedrock. Seismic failure of an earth embankment was assumed to take place in a section having the greatest height.

2. *All of the reservoir side slopes are gently sloping and it is believed there is no potential for a seismic shock causing a landslide into a reservoir pool resulting in a flood wave.*

2.4.4.4 Water Level at Plant Site

Storm No. 19 from Former Appendix 2F centered upstream of Cowans Ford Dam with seismic failure of Bridgewater Dam produced the high water elevation at the site of 762.6 feet msl. Maximum waterheight of 5.11 feet presented in Table 2-20 superimposed on the above elevation gives a high elevation of 767.71 feet msl.

2.4.5 Probable Maximum Surge And Seiche Flooding

2.4.5.1 Probable Maximum Winds and Associated Meteorological Parameters

The probable maximum winds in the vicinity of the McGuire Station will be generated by a Probable Maximum Hurricane (PMH) as defined in the unpublished National Oceanic and Atmospheric Administration (NOAA) report HUR7-97 (Reference 4). The critical parameters associated with the PMH are:

Central Pressure Index (CP1)	*26.91" Hg*
Asymptotic Pressure at Outer Periphery	*31.00" Hg*
Representative Small Radius to Region of Maximum Winds	*7 Nautical Miles*
Representative High Speed of Translation of Hurricane Center	*40 Knots*
Adjusted Maximum Overwater Wind Velocity	*124 MPH*
Adjusted Maximum Overwater Wind Velocity over the Effective Fetch	*84-96 MPH*

The critical path for the PMH parallels the path of the July, 1916, hurricane originating in lower latitudes, moving northwest and ashore at Myrtle Beach, South Carolina, turning west and passing directly through the plant area.

The maximum overwater wind velocity over the effective fetch is less than the actual hurricane wind velocity due to the orientation of Lake Norman with respect to the direction of approach of the PMH.

2.4.5.2 Surge and Seiche History

Lake Norman, immediately north of the plant, is a relatively new inland lake with no history of surge or seiche flooding.

2.4.5.3 Surge and Seiche Sources

Maximum historical winds and atmospheric pressure changes described in Section 2.3 do not approach those of the PMH. The surge caused by the adjusted maximum wind of 96 MPH blowing in the direction of the effective fetch to the earthdike is 0.90 feet. The equation from Reference 5 used to determine the surge is:

$$S = \frac{V^2F}{1400D}$$

where:

V = wind speed in MPH = 96 MPH

F = twice the effective fetch = 4.08 miles

D = average water depth = 30 feet

The seiche estimated to be caused by the maximum pressure gradient during a PMH is 1.93 feet. The assumptions made in this computation are that the eye of the PMH is centered over the plant, the effective seiche distance is nine miles, and the atmospheric pressure varies according to formula 1 of HUR 7-97. For both surge and seiche computations, the Lake Norman water surface elevation is assumed at 760.0 msl.

2.4.5.4 Wave Action

Deep water waves generated by the Probable Maximum Hurricane, PMH, winds were analyzed to determine any effects on the Cowans Ford Dam and the McGuire Intake Structure. This analysis also uses the procedures and techniques demonstrated in the "Shore Protection Manual" for the wind effects of the PMH winds of 96 miles per hour coincidental with the full pond water level elevation of 760 feet msl for Lake Norman.

This design criteria is an extremely unlikely occurrence, since the maximum wind speed recorded for the area is 57 mph (see Former Appendix 2F, Section 7.0). Also the ASCE Transaction paper 3269 recommends a wind speed of 95 mph for a 100-year recurrence interval. Table 2-20 and Table 2-21 give the resulting wave criteria obtained for both the Cowans Ford Dam and the McGuire Intake Structure for the continuous duration over the effective fetch (See Figure 2-52).

2.4.5.5 Resonance

Due to the natural irregularity of Lake Norman's shoreline, there is no resonance of water waves.

2.4.5.6 Runup

The runup associated with the breaking of significant and maximum waves caused by Probable Maximum Hurricane, PMH, is 9.41 feet and 11.92 feet, respectively, as shown in Table 2-20 and Table 2-21. The most severe combination of surge (assumed to be setup), seich, and wave runup with Lake Norman at full pond (elevation 760.0) results in water elevation of 774.75 for maximum waves, and 772.24 for significant waves. This results in no overtopping of the Cowans Ford Dam embankment at elevation 775.0 feet msl or of the elevation 780.0 feet msl protective dike upstream of the plant yard. Therefore, wave runup presents no problems to any safety-related facilities.

As shown by Section 3-3 on Figure 2-51, the operating deck for the McGuire Intake Structure at elevation 770.0 feet msl is also subject to wave overtopping. This is not a plant safety problem, however, as the primary function of the intake structure and its associated pumping equipment is to supply condenser cooling water to the turbines in the Turbine Building, a Non-Category I structure. Since the vertical runup elevation due to significant and maximum waves is 772.39 feet msl and 778.27 feet msl respectively, overtopping results in approximately 2.19 cfs per foot of water actually spilling onto the operating deck of the intake structure. The intake structure is approximately 60 feet wide with vertical screens, pumps, and various openings restricting the flow of the water across approximately 80% of the structure width. Thus the actual runup on the dike at elevation 780.0 feet msl behind the intake structure will be greatly reduced or eliminated. There is a drainage ditch immediately behind the intake structure parallel to the dike that will carry approximately 1.0 cfs. Additionally, the front slope of this dike immediately behind the intake structure will be protected with stone in order to eliminate any potential erosion from water splash. Taking all of these considerations into account, the dike is adequately protected against wind generated waves.

2.4.5.7 Protective Structures

The location and design of protective structures is discussed in Section 2.4.10.

2.4.5.8 Wave Forces

The ability of the Cowans Ford Dam protective riprap to resist the effects of wave forces is related to the quantity, layer thickness, and individual weight of the riprap stones placed on the dam. The riprap are randomly placed, hard, durable stones which are not subject to cracking and erosion effects. The weight of the riprap required to resist wave attack is given by the following equation from the "Shore Protection Manual". Reference 8

$$W = \frac{WrH^3}{K_D (Sr-1)^3 \cot 0}$$

where:

W	=	*weight in pounds of an individual unit*
Wr	=	*unit weight of the riprap, 165 lbs/ft^3*
H	=	*design wave height*
Sr	=	*specific gravity of the riprap, Wr/Ww*
Ww	=	*unit weight of water, 62.4 lbs/ft^3*
0	=	*angle of structure slope measured from the horizontal in degrees*
K_D	=	*stability coefficient that varies primarily with the shape of the riprap, roughness of the riprap surface, sharpness of edges, and degree of interlocking obtained in placement*

The average weight of individual riprap units required to withstand the design waves with 0-5% displacement damage is 1770 pounds for the 96 mph winds and 139 pounds for the 40 mph winds (See Table 2-20). As can be seen from Figure 2-53, the size of the riprap units that are used on that portion of the Cowans Ford Embankment above elevation 775.0 feet msl protecting the plant safety-related facilities is 1328 pounds to 2213 pounds (75% to 125% of Wavg.), and the average size of stone is approximately 2.5 feet in diameter. An 18 inch thick filter bed of *well-graded material, from fine sand to 3 inch size gravel, is used as protection beneath the riprap. The required thickness of the riprap layer is calculated by the following formula:(Reference 8)*

$$r = nk\left(\frac{W}{Wr}\right)^{1/3}$$

where:

r = total layer thickness in feet

n = number of layers of riprap units comprising the cover layer

W = individual weight of a riprap unit in pounds

Wr = unit weight of the riprap units

Using this equation, the minimum thickness required for the riprap layer is 5.0 feet, and it extends to the dam crest on the upstream slope. The riprap is inspected at regular intervals by qualified personnel as part of the Duke Power Company Dam Inspection Program.

The structural integrity of the Cowars Ford Dam and the McGuire Intake Structure is also analyzed for the postulated wave forces. The structures were considered to be subject to the static forces associated with non-breaking waves because the "Shore Protection Manual" (Reference 8), "in protected regions or where the fetch is limited, and when depth at the structure is greater than about 1.5 times the maximum expected wave height, non-breaking waves may occur". The static forces exerted against the structures for these waves are given in Table 2-20 and Table 2-21 and were used in the stability calculations.

A stability analysis was also performed on Cowans Ford Dam in order to determine the resistance against sliding when subjected to the given wave forces. As shown in Table 2-20, the factor of safety against sliding is 2.83 for the 96 mph winds and 2.26 for the 40 mph winds. Thus, the wind generated waves will present no detrimental stability effects to the Cowans Ford Dam.

The McGuire Intake Structure is also analyzed against stability failure as well as failure to the structural members when subject to the given wave forces:

1. *Stability Failure*

   To positively insure against stability failure from overturning or sliding toward the lake due to maximum earth pressure forces, the intake structure is anchored into the solid rock foundation with grouted steel dowels. Also, when the wave forces are applied against the open face of the intake structure, they act against the saturated earth pressures and thereby increase the factors of safety against overturning and sliding. The following table gives the various factors of safety for the intake structure, both before and after the wave forces are applied:

	Normal Pool		***PMF Pool***	
	Lake Level @ El 760 Ft MSL		***Lake Level @ 767.9 Ft MSL***	
	Without Wave Forces	***With Wave Forces***	***Without Wave Forces***	***With Wave Forces***
F.S. Sliding	*2.22*	*2.82*	*3.01*	*5.60*
F.S. Overturning	*2.95*	*2.98*	*3.25*	*3.49*

It can be seen from the above tabulation that the addition of the wave forces significantly increases the factors of safety against overturning and sliding in each case, and it is positively concluded that addition of the wind generated wave forces cannot cause a stability failure of the intake structure.

2. Structural Failure

The structural members of the intake structure were designed for at rest saturated earth pressures coincidental to the unwatering of any one of the intake structure cells. The forces and moments obtained from this design criteria used in the analysis of the intake structure were compared to the forces and moments associated with non-breaking waves acting against the members. The skimmer wall, with bottom at elevation 745.0 feet msl across the front of each bay of the intake structure, also acts as a diaphragm member between the piers. This element is considered the most critical member of the structure subject to the wave forces due to the relative size of piers, slab, etc. This member was calculated to have a safety factor of 1.77 against structural failure due to the worst possible wave forces in combination with the design conditions. Therefore, the Intake Structure is safe against structural failure due to forces and moments associated with the postulated wind generated waves of Lake Norman.

2.4.6 Probable Maximum Tsunami Flooding

Does not apply to inland sites.

2.4.7 Ice Flooding

Ice flooding is not applicable to the site area.

2.4.8 Cooling Water Canals And Reservoirs

2.4.8.1 Canals

A short intake channel, approximately 350 feet long, has been constructed to assure condenser cooling water pumping capacity at the intake. The bottom of the intake channel is at El. 715.00 msl which is also the bottom of the intake structure inlet. The width of the channel bottom approximates the 245 foot width of the intake structure inlet. The sides of the channel slope at 2 horizontal to 1 vertical to original ground. Wind waves are not expected to affect the channel since at full pond, El. 760 msl, the channel is almost completely submerged, and at lower lake levels the channel is protected by a natural freeboard of the original ground topography. The intake, channel embankments are further protected by a 4 foot layer of riprap extending for a distance of 100 feet in front of the intake structure.

The discharge water passes through a discharge canal approximately 3000 feet long and into Lake Norman. This canal is 75 feet wide at the bottom, El. 720 msl to El. 730 msl, and is excavated through both soil and rock. The sides of the canal vary in slope depending upon the excavated material. The location also gives it protection from wind and wave activity. The rocky soil and vegetation of the discharge canal embankments give sufficient protection from erosion due to wind and water activity.

The intake channel and discharge canal are shown on Figure 2-4 and typical cross sections are shown on Figure 2-54.

2.4.8.2 Reservoirs

The upstream reservoirs on the Catawba River along with their hydrologic features are discussed in Former Appendix 2F.

The Nuclear Service Water Pond is discussed in Section 9.2.2 and Former Appendix 2G.

2.4.9 Channel Diversions

There are five reservoirs on the Catawba River upstream of Cowans Ford Dam, all of which have operating hydroelectric power plants located on them. Since Duke owns and controls the levels of each reservoir above the site of McGuire Nuclear Station, any upstream diversion or rerouting of the source of cooling water is very unlikely to happen. No present means exist to divert or reroute other than minor amounts used for municipal water supply.

2.4.10 Flooding Protection Requirements

All safety-related structures on the plant yard at elevation 760 feet msl are protected from the flood levels of Lake Norman by an earth dam and dike extension of Cowans Ford Dam. This embankment crest elevation is 775 feet msl adjacent to the spillway at Cowans Ford Dam and rises to elevation 780 feet msl upstream of the plant yard, as shown in the plan on Figure 2-51. Coincident seismic and flood events, considered "reasonable possible combinations of natural phenomena" for Cowans Ford Dam and the adjacent embankments were analyzed to ensure that plant facilities would not be affected by combinations of events required by PSAR review staff. These design criteria and analysis of results are presented in Former Appendix 2H, Section 3.1 thru 4. The various combinations that have been evaluated are:

1. *The 100-year flood level (El. 760.0) coincident with one-half Safe Shutdown Earthquake (1/2 SSE) or Operating Basis Earthquake (OBE). The Safe Shutdown Earthquake (SSE) was termed Design Basis Earthquake (DBE) in PSAR.*
2. *The Standard Project Flood level (El. 762.6) and associated wind generated waves and runup coincident with the Operating Basis Earthquake (OBE).*
3. *The Probable Maximum Flood level (El. 767.9) with associated wind generated waves and runup.*

In the postulated event that failure of the Cowans Ford Dam should occur due to an improbable combination of maximum seismic loading and/or flood an analysis is performed to determine the potential effect on downstream safety-related plant facilities.

Figure 2-51 illustrates the comparative configurations of the earth dam and dike sections of Cowans Ford Dam and their relationship to the plant yard facilities. The original foundation grade elevations of the El. 780 Dike around the plant yard, sections 2-2 and 4-4 is very close to or above yard grade. In contrast, the El. 775 portion earth dam, Section 5-5 between the plant yard and the concrete spillway structure at Cowans Ford Dam has an original foundation grade which varies from about yard grade down to about El. 700 feet msl and is considered much more critical in any seismic loading due to its relative height. Therefore, the failure or breach of Cowans Ford Dam due to any unspecified seismic event is assumed to occur in the El. 775 portion earth dam embankment section.

Available technical literature indicates that breaches in earth dams, as a result of the Chilean earthquake of March 26, 1965, can vary in length from 45 to 100 meters (Reference 9). On this bases, it is assumed that an initial breach in the Cowans Ford Dam earth embankment at elevation 775 feet msl can be 300 feet in length. The base of the embankment breach is taken *at approximate natural ground level for Lake Norman during a Standard Project Flood (SPF) is 762.6 feet msl, which corresponds to a maximum initial depth of 62.6 feet behind the earth dam.*

The dam breach analysis is based on the techniques and principles of unsteady flow as presented by Chow (Reference 10) and Henderson (Reference 11). In order to consider the worst possible situation, the following assumptions are made:

1. *The downstream bed from the dam is dry. (Flow downstream of the dam would reduce the wave surge height and thus be less critical.)*
2. *The channel is frictionless, rectangular, and horizontal.*
3. *Vertical accelerations are negligible.*
4. *A 300 foot section of the dam is instantaneously and completely removed.*

According to Henderson, (Reference 11) the maximum surge height downbreach is 4y/9, where y is the initial depth of the lake. Therefore, the maximum wave height is

$$4y/9 = 4(62.6)/9 = 27.82\text{feet}$$

The additional discharge over the spillway at Cowans Ford, when the lake is at maximum SPF stage, has been neglected as lowering of the lake level would reduce the assumed surge height. Based on the assumed conditions, a maximum surge height at elevation 727,82 feet msl will result. Since this analysis has neglected any energy dissipation due to friction and changes in channel alignment and geometry, it predicts the most conservative upper limit for the maximum downstream water elevation resulting from a seismic breach.

The maximum stage of the Catawba River downstream of Cowans Ford Dam has also been determined following the postulated breach of the dam. The effects of the flood waters from Lake Norman are evaluated for the following three combinations of events: 1) a 300 foot breach induced by the OBE with SPF, 2) a 600 foot breach induced by the OBE with SPF, and 3) a 1000 foot breach induced by the SSE.

For each of the above conditions, the instantaneous flow at the moment of breach was calculated from the equation: (Reference 14)

$$Q\max = 0.29 g^{0.5} K^{0.28} W_b D_b^{1.5}$$

where:

Q max	*= peak discharge at the dam, cfs*
g	*= acceleration of gravity*
W_b	*= width of breach*
D_b	*= depth of water above the breach*
K	*= ratio of the width of the dam to the width of the breach*

This discharge continues for approximately 20 minutes, the time required for the negative wave generated by the breach to travel the reach of Lake Norman (approximately 5 miles) and return.

After the initial failure, the dam breach has been assumed to widen at the rate of 1.4 feet per minute. The maximum breach width is assumed to be 1330 feet. The breached section is assumed to act as a broad-crested weir, and the discharge is calculated from the equation: (Reference 15)

$$Q = 3.087LH^{3/2}$$

where:

Q = discharge, cfs

L = length of weir

H = head on weir

The critical control section downstream of Cowans Ford Dam is highway NC 73 because the trapezoidal cross-sectional openings at the Catawba River bridge (area equal to 38,000 ft^2) can pass only 210,000 cfs at Elevation 689.0. Therefore, the flood flow following the dam breach will overtop the highway embankment before a maximum stage is reached. Conservatively assuming that the highway embankment remains intact, the discharge rating curve for the highway embankment, shown in Figure 2-57, was developed from the weir equation above.

The maximum stage is reached when the inflow and outflow are equal. Neglecting any available storage in the area between Cowans Ford Dam and Highway NC 73 yields a conservative maximum water surface elevation. Tabulated below is a summary of the postulated seismic and flood failure modes and the resulting flood water elevations. Figure 2-58 shows the breach hydrographs for these conditions:

Seismic Event	Flood	Initial Breach Width	Peak Discharge	Catawba River Maximum Stage
OBE	SPF	300 Ft.	1,264,000 cfs	719 ft, MSL
OBE	SPF	600 Ft.	1,415,000 cfs	720 ft, MSL
SSE	---	1000 Ft.	1,641,000 cfs	721 ft, MSL

The plant yard and associated safety related facilities are located at Elevation 760.0, above the maximum theoretical surge wave and river stage, and are therefore protected from any damage as a result of the dam breach. The only safety-related facility which could be affected by the dam failure is the Standby Nuclear Service Water (SNSW) Pond Dam located approximately 2000 feet east of the river channel up a narrow valley. This facility has a foundation at approximately elevation 690.0 feet msl and crest elevation of 747.0 feet msl. Both the maximum surge wave and maximum downstream river stage are below the SNSW Pond Dam crest, and therefore, the dam will not be overtopped during the postulated dam breach. As shown in Figure 2-59, the topography of the area between Cowans Ford Dam and the SNSW Pond Dam will prevent detrimental erosion on the downstream face of the SNSW Pond Dam. The rising stage of the Catawba River will cause water to back up onto the downstream slope of the dam, but the maximum vertical rise rate is less than 4 fpm. This velocity is well below the maximum permissible velocity for the existing in-place riprap and grass. Fluctuation of the water surface will, therefore, not adversely affect the integrity of the SNSW Pond dam.

Wind wave activity coincident with dam failure is not considered significant due to the conservatism of the breach analysis and the short time period that the maximum stage exists before it recedes.

Another possible source of flooding results from the local Probable Maximum Precipitation (PMP), discussed in Section 2.4.3.1. The plant safety-related buildings are protected against flooding from the PMP with a system of roof drains, a surface collection system, and ditches *arranged around the plant in such a way as to direct runoff away from the plant to natural drainage channels. (See Site Drainage Map, Figure 2-55.)*

Roof drains, designed to discharge 5 inches per hour, are provided on all safety-related buildings. Continuous architectural parapets are provided on the roofs at elevations 804+6 and 825+0 of the Fuel Handling Building and on the roof of the Reactor Building. Parapets are provided on not more than three sides on all other roof levels of the Auxiliary Building. The roof plan is shown on Figure 2-60. The parapets are shown on Figure 3-106 and Figure 3-107.

The parapets on the Fuel Handling Building are provided with scuppers to permit any water that may pond on the roof to be discharged onto the plant yard. The scuppers, located on Figure 2-60, are 6 inches high by 9 inches long and are placed 3.5 inches above the finished roof surface.

When partially submerged, the discharge capacity of a scupper is calculated from the formula: (Reference 13)

$$Q = 3.33LH^{1.5}$$

where:

Q = discharge, cfs

L = length of scupper (0.75 feet)

H = head, ft.

When fully submerged, the scupper acts as an orifice, and the discharge capacity is computed from the formula:

$$Q = .63A\sqrt{2gh}$$

where:

Q = discharge, cfs

A = area of scupper, ft.^{2}

g = acceleration of gravity

h = head, ft.

Using the above relationships and assuming that all roof drains are clogged, the maximum depth of water that accumulates on the Fuel Handling Building roof during the local Probable Maximum Precipitation (PMP) is calculated to be 11.5 inches. The Fuel Handling Building roof has been checked for the load combination:

$$U = 1.5D + 1.0R$$

where:

U = ultimate load, psf

D = total dead load, psf

R = maximum hydrostatic load during the PMP, psf

This load combination does not exceed the design bases loading for the Fuel Handling Building roof.

The parapets on the Reactor Building are not provided with scuppers, and if the roof drains become clogged, water will pond to the top of the parapets before spilling over the side. The Reactor Building roof is designed to withstand the hydrostatic load resulting from the maximum accumulation of water behind the parapet during the PMP.

There is no parapet at the south end of the main level of the Auxiliary Building at Elevation 784+0. Thus a pathway is provided to discharge any water that may pond on the roof during the PMP. The roof loading due to the maximum accumulation of water does not exceed the design bases loading for any portion of the Auxiliary Building roof.

Scuppers in the parapets of the Fuel Handling Buildings and Auxiliary Building roof areas allow for secondary drainage for postulated beyond design basis flood events.

All pipesleeves, ventilators and curbs penetrating the roof of any safety-related building has been extended above the calculated level of excess ponding to eliminate flooding paths during a local PMP. All hatches and other safety penetrations are adequately waterproofed to ensure their integrity during the PMP.

During the PMP, any runoff on building roofs that is not discharged by roof drains to the storm drain system flows off the roofs and onto the plant yard.

The buried storm drainage system is designed for 4 inches per hour precipitation with any remaining precipitation stored in the plant yard or overflowing the plant yard perimeter by sheet flow. Considerable storage of precipitation results from the 1 foot differential between the plant yard high points and ridge lines at elevation 760.0 feet msl and the top of the catch basins at elevation 759.0 feet msl. This creates "pyramids" of storage around the plant yard which have a capacity of approximately 155,000 cubic feet. Runoff is directed away from the plant toward the catch basins with a minimum ground slope of 1.4%. Although the yard drainage system, itself, was not designed to discharge the Probable Maximum Precipitation (PMP), the system has been analyzed to ensure that the buildup of water due to PMP will not endanger any safety-related facilities.

The runoff from local PMP across the plant area was determined using the Rational Method formula: (Reference 12)

$$Q = CiA$$

where:

Q	=	*peak rate of runoff in cfs*
C	=	*weighted runoff coefficient expressing the ratio of rate of runoff to rate of rainfall*
i	=	*average intensity of rainfall in inches per hour for PMP during the time of concentration*
A	=	*drainage area in acres (plant yard within access road)*

In calculating the total quantity of rainfall for the plant area, the rainfall intensity, i = 14.7 in/hr., was selected as discussed in Former Appendix 2G. Table 2-23 shows a tabulation of rainfall intensities and critically arranged time increments for the PMP. the runoff model used in design assumes the most conservative runoff coefficient, C = 1.0.

After the quantity of runoff has been determined, two methods of analysis are performed to determine whether excess water backup would affect any safety related facilities. One method of analysis assumes perimeter runoff, and the storm drainage system is operational to one-half its total capacity. This would account for any debris or other material that could be partially *blocking the system. The second method of analysis is the highly unlikely event that the storm drainage system is completely inoperative or blocked, and the entire PMP runoff is discharged by sheet flow at perimeter of yard.*

In the first method of analysis, the depth of water in the plant yard is determined by using the Orifice Equation: (Reference 12)

$$Q = CA\sqrt{2gh}$$

where:

Q = discharge as determined from the Rational Method

C = discharge coefficient, 0.6

A = cross-sectional area of the flowing water taken at right angles to the direction of flow

g = gravitational constant

h = head under which the water is flowing

Based on these calculations, it is determined that water will pond to elevation 760.28 feet msl, which is below the doorway grades of safety-related structures (See Figure 2-56).

In the second method of analysis, it is assumed that the perimeter of the protected area acts as a weir for runoff to overflow the perimeter. When the quantity of flow from PMP equals the quantity of flow crossing the weir in a given time period, equilibrium has been reached, and the depth of ponding is determined. The runoff across the weir is determined by the formula: (Reference 13)

The equation below was updated in the 2012 revision.

$$Q = 2.68(LH^{3/2})$$

where:

Q = the rate of discharge over the weir

L = the length of the weir

H = depth of flow over the weir

Some of the plant structures present obstructions to water flowing over the entire weir, therefore, the length of the weir is not assumed to be the entire distance around the plant but is divided into segments determined by how the runoff would actually flow through the plant yard. Based on these calculations, water will pond to elevation 760.375 feet msl which is 1 1/2 inches below the doorway grades of safety-related structures (See Figure 2-56).

As a result of the two analyses, it is positively determined that no doorway or opening of a safety-related facility will be flooded by the runoff from PMP. The maximum water depth adjacent to the plant without taking credit for storm drainage is elevation 760.375 feet msl, which is 0.125 feet below the minimum elevation of any safety-related building opening. Any openings that are below El. 760.5 feet are provided with curbs, drains, or inclined ramps to prevent the inflow of water.

Temporary flood barriers will be installed during postulated beyond design bases flood events in doors to protect external flood water from entering the safety related structures.

The magnitude of the normal snow load and the severe snow and ice load is equal to 20 and 40 pounds per square foot, respectively (Reference 17). The roofs will carry these loads as discussed in Section 3.8.

The maximum recorded snowfall at Charlotte during a 44-year period of record is 12.0 inches in 24 hours and 19.3 inches in 30 days. Thus the normal snow design load of 20 psf provides a safety factor greater than 3.

No ice accumulation records are maintained by any weather service, however, typical ice accumulations in the region are from one-fourth to one-half inch. The roof load caused by this severe icing coincident with the extreme snowfall of record would not exceed the design bases of any safety-related roof.

2.4.11 Low Water Considerations

2.4.11.1 Low Flow in River and Streams

As required in the Federal Power Commission License, Project No. 2232, for the Catawba-Wateree Project, the following minimum flow is maintained by Duke downstream of Lookout Shoals Dam:

60 cfs - Minimum instantaneous flow
278 cfs - Minimum average daily flow

According to the FPC License, the maximum drawdown level for Lake Norman is 25 feet (to el. 735). However, drawdown is limited to improve the recreational use values and potential of Lake Norman to 15 feet (el. 745).

2.4.11.2 Low Water Resulting From Surges

Surges are not applicable to this site as mentioned in Section 2.4.5.2.

2.4.11.3 Historical Low Water

The minimum water elevations for Lake Norman since commercial operation are shown in Figure 2-61.

2.4.11.4 Future Control

The minimum flow downstream of Lookout Shoals Dam into Lake Norman is stated in Section 2.4.11.1. The average river flow at Cowans Ford Dam is 2760 cfs. No future projects affecting the minimum flow are proposed on the Catawba River upstream of Cowans Ford Dam. The minimum flow below Cowans Ford Dam, 80 cfs continuously and 311 cfs average daily, as required in the FPC license, remains unaffected by McGuire.

2.4.11.5 Plant Requirements

The required minimum condenser cooling water for McGuire Nuclear Station, Units 1 and 2 operating at full power is 2,860 cfs in the winter with a Delta T of 24°F across the condensers and a maximum of 4,540 cfs in the summer with a Delta T of 16°F across the condensers. Design basis for the Condenser Cooling Water System is detailed in Section 10.4.5.

The minimum service water requirement is 2,000 gpm/unit during station shutdown. The maximum service water requirement is 17,500 gpm/unit to a maximum of 30,000 gpm for the *station. Details of the low pressure service water system are given in Section 3.6 of the McGuire Environmental Report and in FSAR Section 9.2.1.*

The intake and discharge structures are designed to minimize the effect of turbulence and vortexing at the extreme operating ranges of Lake Norman to provide for less effluent mixing.

Under normal circumstances, the nuclear service water is withdrawn from the Low Level Intake Cooling Water System. An auxiliary source is provided as described in Section 9.2.2 with the final alternative being the Standby Nuclear Service Water Pond.

2.4.11.6 Dependability Requirements

Impending low lake level could occur only in the event of the failure of Cowans Ford Dam. Should this happen, the station operators would begin shutdown and switch to the SNSW Pond as the source of nuclear service water.

Section 9.2.5 details the operating equipment of the SNSW Pond. The SNSW pumps are inline pumps whose centerline elevation is at El. 719.4 feet msl. The centerline elevation of the intake pipelines located in the SNSW Pond is 700.0 feet msl. The maximum drawdown in the SNSW Pond is elevation 739.5 feet msl. However, the pond is sized to meet the ultimate heat sink requirements at the lowest elevation of 738.0 feet msl.

Ample condenser cooling water is available with Lake Norman at its minimum operating level of elevation 745.0.

One hundred thirteen miles of the Catawba River upstream of Cowans Ford Dam are controlled by a system of five hydroelectric lakes whose elevations are strictly controlled by Duke for the production of hydroelectric power.

However, the impact of possible drought conditions has been studied. As shown in Table 2-24, the minimum average inflow for various flow conditions and recurrence intervals is given. As shown in the table, the total of the evaporative losses due to the heat dissipation of the condenser cooling water at McGuire and Marshall Steam Stations, and the minimum average flow required downstream of Cowans Ford, is greater than the minimum average inflow; however, available storage from Lake Norman is more than adequate to maintain the required minimum flows.

The evaporative losses do not include natural evaporation from Lake Norman which is estimated to be 40 inches per year (Reference 6). The average rainfall over Lake Norman, 44 inches per year, more than restores the loss by natural evaporation.

2.4.12 Environmental Acceptance of Effluents

Any radioactive liquid effluents are released to the condenser circulating water discharge. There is no credible accidental release of radioactive liquids to groundwater since any spills in the station are collected in floor drains and sumps for treatment if required. Since rainfall rather than impounded surface water is the source of groundwater in the area, there is no risk of groundwater contamination.

Station discharge concentrations do not exceed limits applied to public drinking water. Also, some decay occurs after discharge and there is additional dilution downstream of Cowans Ford Dam. The expected release concentrations are shown in Section 11.2.8.

Chemical discharges are first retained in the Conventional Waste Water Treatment System for appropriate treatment and then are discharged to the Catawba River below Cowans Ford Dam.

The discharge concentrations do not exceed limits applied by appropriate governmental agencies.

A detail study of thermal effects of the condenser cooling water discharge is contained in McGuire ER-OLS, Section 4.

2.4.13 Groundwater

2.4.13.1 Description and Onsite Use

The station site lies within the groundwater region known as the Charlotte area, which is part of the Piedmont Groundwater Province. Groundwater in this area is derived entirely from local precipitation and is contained in the pores that occur in the weathered material above the relatively unweathered rock and in the fractures in the igneous and metamorphic rock. The water table varies from ground surface elevation in valleys to more than 100 feet below the surface on sharply rising hills. A full discussion of site groundwater hydrology is contained in Former Appendix 2B. Groundwater is not a water source for the station.

2.4.13.2 Sources

Groundwater is used as a domestic water supply for the few residences in the area immediately surrounding the site. This is the only use of groundwater in the foreseeable future for this area. The locations of existing wells in the area are shown on Former Appendix 2B, Figure 2B-1, and data on these wells are given in Former Appendix 2B, Table 2B-1. The occurrence, location, and movement of groundwater at the site is controlled primarily by the water level in Lake Norman, which borders the site on the north. Flow of groundwater is normal to groundwater contours shown on Former Appendix 2B, Figure 2B-2. Soil permeability values for the area range from about 200 to 300 feet per year (Former Appendix 2B, Table 2B-3). A typical section of the groundwater gradient is shown in Former Appendix 2B, Figure 2B-3. No reversal of groundwater flow is expected due to the topography. The only expected groundwater recharge areas within the influence of the station are adjacent to the Standby Nuclear Service Water Pond and the Waste Water Collection Basin as shown on Figure 2-4.

2.4.13.3 Accident Effects

Spills of potentially radioactive liquids within the station are not a source of groundwater contamination due to the floor drain systems in the Containment and the Auxiliary Buildings. Liquids cannot seep through the concrete as all sumps are stainless steel lined. Concentrations of cesium and strontium in wells resulting from groundwater movement from the lake to the well are less than any lake concentrations due to the additional decay afforded by the ion exchange action of the soil. Former Appendix 2B, Section 2.6 discusses the ion exchange potential of the soil at the site.

An analysis of the potential contamination of domestic wells by Cs and Sr was performed based on the following conservative assumptions:

1. *Average groundwater movement rate of 300 ft. per year (Former Appendix 2B, Section 2.3).*
2. *Minimum shoreline to well distance of 100 ft. (Former Appendix 2B, Figure 2B-1).*
3. *Effective Cs and Sr movement rates of 1/92 and 1/46, respectively, that of groundwater (Former Appendix 2B, Section 2.6).*
4. *Concentration of Cs and Sr in Lake Norman water as discussed in Section 11.2.8.*

The slower movement rates of Cs and Sr in the groundwater due to the ion exchange action of the soil result in sufficient decay time to effect a reduction in concentration of approximately 0.5 and 0.67, respectively, that of the initial Lake Norman concentrations.

Due to the proximity of the closest well to its source (100 ft), the possibility of a direct path for groundwater flow was not overlooked. In this case, the Cs and Sr concentration in the well would be just equal to the Lake Norman concentration.

These results are tabulated below and in either case the concentrations are substantially below 10CFR 20 limits.

Isotope	Concentration in Domestic Well Water (μCi/ml)		Fraction of 10 CFR 20 Limits	
	Ground Water Movement		Ground Water Movement	
	300 ft/yr	Instantaneous	300 ft/yr	Instantaneous
Cs 137	2.2×10^{-11}	4.4×10^{-11}	1.1×10^{-6}	2.2×10^{-6}
Sr 90	5.0×10^{-15}	7.4×10^{-15}	1.7×10^{-9}	2.5×10^{-9}

As previously stated there is no credible accidental release of radioactive liquids to the groundwater due to containment provided by Category 1 structures. However, an accidental release of liquid radioactive material based upon assumed failure of the floor drain tank and subsequent failure of a Category 1 floor is postulated to demonstrate the potential radiation contamination to groundwater. A postulated rupture of the floor drain tank would result in a spill of radioactive material, which would eventually flow into the groundwater sumps of the Category 1 buildings. Groundwater or contaminated liquid collected in sumps A or B would be pumped to the Turbine Building sumps. Normal sump discharges are pumped to the Conventional Waste Water Treatment System for treatment prior to release to the Catawba River. If the radiation monitors in the discharge lines to the Conventional Waste Water Treatment System indicate that radioactive materials are present in the sump discharge, the sump contents may be directed to the Liquid Waste Monitor and Disposal System. Assuming the flow into the Conventional Waste Water Treatment System is not terminated upon the receipt of a high radioactivity alarm, adequate holdup capacity is available in the Conventional Waste Water Treatment System to contain the postulated spill. Ground water or contaminated liquids collected in sump C would be pumped to a free outfall at the storm drain system which discharges into the SNSW Pond. Inflow to the SNSW Pond is passed to the Waste Water Collection Basin through the SNSW Pond outlet facility. Sufficient holdup capacity is provided in the Waste Water Collection Basin to contain the postulated spill. As shown on Figure 2B-2A, the SNSW Pond and the Waste Water Collection Basin are down gradient from all wells in the area, and contamination of the groundwater supply is not possible.

If the rate of flow into the three sumps is assumed equal to the capacity of the three sump pumps (750 gpm), the total postulated spill can be pumped from the Auxiliary Building sumps to the Turbine Building sumps and/or the SNSW Pond in approximately 15 minutes.

2.4.13.4 Monitoring or Safeguard Requirements

The potential for groundwater contamination is very low thus groundwater is not routinely sampled for radioactivity as part of the environmental radiological monitoring program.

A discussion of the environmental radioactivity monitoring program is provided in Section 11.6.

2.4.13.5 Design Bases for Subsurface Hydrostatic Loadings

As shown in Former Appendix 2B, Figure 2B-2, preconstruction groundwater levels were approximately 10 to 35 feet below plant yard grade. Reactor, Auxiliary and Turbine Building excavations in soil and weathered rock below plant yard grade were dewatered by eductor wellpoints located on the western, northern and eastern perimeter of the excavation. Excavations in rock below plant grade were dewatered by excavated sumps located at convenient construction locations.

A permanent Category I underdrain groundwater system is installed as shown on Figure 2-62 and Figure 2-63 to maintain the groundwater level below elevation 717.0 for the Reactor Building and elevation 712.0 for the Auxiliary Building. The underdrain system consists of a grid of interconnected flow channels at the top of rock or top of fill concrete below the foundation slabs. The grid of flow channels drains the entire foundation of the Reactor Building, and Auxiliary Building complex except for deeper pits which are designed for hydrostatic loads. Drilled holes through fill concrete into rock, at a maximum spacing of 8 feet on center, permit groundwater to flow from beneath the fill concrete slabs into the flow channels. All channels in the grid system drain by gravity to three sumps located in the Auxiliary Building. (A and B, 10 ft. x 10 ft. x 15 ft. deep, and C, 17 ft. x 17 ft. x 12 ft. deep.) As shown on Figure 2-62, an exterior wall drain system composed of two separate flow mediums or pathways, extends around the foundation perimeter and drains directly to sump C. This wall drain consists of a 2 ft. minimum thickness zoned sand and stone filter, placed vertically from the bottom of the excavation to a point 5 feet below yard grade, and an 8" perforated metal pipe which is continuous horizontally around the exterior wall at the bottom of the filter. Groundwater collected in the sumps is pumped to the yard storm drain system or to Turbine Building sumps. Two 250 gpm Category I pumps, each capable of handling the total flow into the sumps for a pump cycle, maintain the water level automatically in each sump. In the unlikely event a pump fails to start and water rises above the normal operating level of the sump, the second pump will automatically start and will continue to operate as required. If either or both pumps fail to start, an alarm will alert the operator. Since the three sumps are interconnected by the grid drain channels at Elevation 712 msl, all six pumps are available to discharge groundwater. In the unlikely event that two pumps become inoperable in any one sump, groundwater would flow through the many redundant channels to the other sumps. Details of the underdrain system, wall drain system, and sumps are shown on Figure 2-63. Calculations for estimating the groundwater flow are presented in Former Appendix 2D Section 5.1.1. System description and instrumentation and control for the groundwater sump pumps are presented in Sections 9.5.8 and 7.6.11.

Four independent discharge lines, each capable of handling the system capacity are provided to discharge groundwater from the Auxiliary Building sumps. The A pump in A sump pumps groundwater to Unit 1 Turbine Building sump. The B pump in A sump pumps groundwater to Unit 2 Turbine Building sump. The A pump in B sump pumps groundwater to Unit 2 Turbine Building sump. The B pump in B sump pumps groundwater to Unit 1 Turbine Building sump. This logic is provided to have the maximum sytem flexibility. Groundwater collected in sump C is pumped to a free outfall at the storm drain system through separate discharge lines for each pump. The free outfall drains to the storm drain system and prevents siphoning to the groundwater sump. In the event the storm drain system becomes blocked, the sump discharge would flow to adjacent catch basins or would discharge off the yard by sheet flow. The invert of the free outfall is located two feet above yard grade to prevent flooding of the Groundwater Drainage System during the local Probable Maximum Precipitation.

Multiple redundancy of vital system components will assure the ability of the system to function over the life of the plant. In the unlikely event that a single flow channel or wall drain becomes blocked, groundwater will flow to the sumps through any of the many redundant drain routes *available. Six Category 1 pumps, each capable of handling the total estimated flow, will assure the function of the sump. Monitoring of pump operation provides assurance that the zoned wall filter, drains and pumps are properly functioning.*

Although the wall drain and underdrain prevent a rise in the groundwater around the exterior building wall, calculations are presented in Former Appendix 2B Section 2.7 for the water level recovery based on the postulated simultaneous blockage of all drains into the sumps. Figure 2B-8 shows the results of the groundwater level recovery calculations. Based on design calculations, uplift and overturning of the Auxiliary Building will occur when groundwater rises above elevation 737 Feet Mean Sea Level (MSL). The maximum groundwater elevation is considered to be 760 Feet MSL. This level is based on a full lake level of 760 Feet MSL. The Reactor and Diesel Generator Buildings are designed to withstand overturning if groundwater rises to the maximum elevation. In addition, the Auxiliary, Reactor, and Diesel Generator Building walls are designed for maximum groundwater elevation. The time required for a rise in groundwater from elevation 712 Feet MSL (maintained by the under drain system) to 737 Feet MSL under the most adverse soil conditions is approximately 20.3 days. A groundwater level monitoring program, as described in Section 2.1.2 of Former Appendix 2B, has verified the expected construction stage drawdown and the stabilized elevation of groundwater outside the Reactor and Auxiliary building walls. The layout of these groundwater level monitoring wells is shown on Figure 2-62. The zone of influence of groundwater drawdown by the groundwater control system is discussed in Former Appendix 2B, Section 2.1.2.

Eleven permanent groundwater monitors are installed around the perimeter of the Auxiliary and Reactor Building exterior walls to monitor the groundwater level in the zoned wall filter. Seven interior monitors, instrumented through holes in the wall, are mounted inside the Auxiliary and Diesel Generator Building. One exterior groundwater monitor, instruments in a drilled cased well, drilled into the zoned filter located inside the Unit 2 Equipment Staging Building. Three exterior monitors, instrumented in cased wells drilled into the zoned wall filter, are located outside the Reactor and Auxiliary Building (See Figure 2-62 for location of monitors).

All eleven monitors provide three points of alarm to alert operators to a rise in groundwater. The seven interior groundwater monitors are located 2 Feet 3 Inches above the top of the adjacent floor slab. The four groundwater monitors in the Diesel Generator Building have a first alarm point set at 2 Feet 8 Inches above the top of the adjacent floor slab. The second alarm point is set at 5 Feet, and the third alarm point is set at 15 Feet above the adjacent floor slab The three groundwater monitors located in the Auxiliary Building have a first alarm point at 5 Feet above the adjacent floor slab. The second alarm point is set at 10 Feet, and the third alarm point is set at 15 Feet above the adjacent floor slab.

For the two exterior groundwater monitors located adjacent to the Reactor Buildings, the first alarm point was set at the elevation of the top of the adjacent floor slab, but is disabled. The second alarm point is set at 5 Feet, and the third alarm point is set at 15 Feet above the top of the adjacent floor slab for Unit 1 (11' - and 21' for Unit 2). The exterior groundwater monitor located inside the Unit 2 Equipment Staging Building has a first alarm point set at elevation 731 Feet MSL. The second alarm point is set at 4 Feet 9 Inches, and the third alarm point is set at 14 Feet 6.5 Inches above elevation 731 MSL. The exterior groundwater monitor located adjacent to the Unit 1 Auxiliary Building has a first alarm point set at elevation 716 Feet MSL with the second alarm point set at 5 Feet and the third alarm point set at 15 Feet above 716 Feet MSL.

Any single alarm or any combination of individual alarms will alert the plant operator to a groundwater rise. Orginally, McGuire incorporated all 11 monitors as Tech Specs monitors. Subsequently an analysis performed by Design Engineering demonstrated that the Reactor and

Diesel Generator Buildings were designed to withstand groundwater stress up to 760 feet Mean Sea Level (MSL). Therefore, a Tech Spec revision was sought and obtained that removed all but 5 of the Auxiliary Building level monitors from the Tech Specs. The other 6 Reactor and Diesel Building level monitors were placed in Chapter 16, SLC 16.9.8. The Auxiliary Building level monitors were placed in SLC 16.9.8 when McGuire converted to improved Tech Specs. The SLC limits for the Auxiliary Building are provided to ensure that the groundwater levels will be monitored and prevented from rising to the potential failure limit for McGuire Units 1 and 2 Auxiliary Buildings. This potential failure limit is based on engineering calculations that have determined that the Auxiliary Buildings are susceptible to overturning due to buoyancy at Elevation 737 feet MSL. Under the requirements of SLC 16.9.8, if groundwater level exceeds Elevation 731 feet MSL for 3 out of 5 of the SLC Auxiliary Building groundwater monitor alarms, and cannot be reduced in one hour, McGuire must begin reducing Units 1 and 2 to Mode 5, cold shutdown.

Since the zoned filter wall drain system is confined by building walls and the compacted earth backfill (or rock excavation at the foundation level) the wall drain system will remain passive during earthquake as will the underdrain system. Since the top of the zoned wall filter is 5 feet below plant yard grade there is no credible flood that will affect the underdrain system.

There is no credible risk to the underdrain system from non-Category 1 piping systems. Cutoff plates are provided around CCW pipe to prevent seepage along the soil-pipe interface. Plate stiffeners, 7 inches high and spaced at a maximum of 7'-6" along the pipe, provide additional assurance the seepage will not follow the CCW pipe. The postulated failure of the CCW pipe in the yard results in seepage of approximately 38 gpm to the underdrain system. Therefore, the failure of the condenser cooling water pipe will not flood the underdrain system.

The zoned wall filter and perforated pipe is not required on the south side of the Auxiliary Building because the adjacent Turbine Building will prevent a rise in groundwater above elevation 735.0. The radius of influence of drawdown due the underdrain system will limit the height that groundwater will rise on the Auxiliary Building walls. Additionally, the Auxiliary Building walls can withstand full hydrostatic loads to elevation 760.0 in combination with other loads described in Section 3.8.4.

The postulated failure of the Nuclear Service Water pipe has been evaluated to determine the potential for flooding the groundwater underdrain system. The pipes for this system penetrate the zoned wall filter and would result in the largest discharge of water into the underdrain system. The Nuclear Service Water System is a moderate energy fluid system and has been evaluated according to NRC Branch Technical Positions MEB 3-1 and APCSB 3-1. A throughwall leakage crack, one-half the pipe diameter x one-half the wall thickness, would result in a flow of 666 gpm to the underdrain system. This flow plus the calculated groundwater seepage would result in a total flow of 696 gpm. Since six - 250 gpm pumps are available to discharge groundwater the postulated failure of the Nuclear Service Water pipe will not flood the underdrain system.

Walls below plant yard grade are not waterproofed, but waterstops are provided in all construction joints. In the unlikely event that the groundwater level rises outside the Reactor and Auxiliary Building walls, seepage through the wall will not exceed the capacity of the floor drain system.

2.4.14 Technical Specification And Emergency Operation Requirements

With the exception of the Selected Licensee Commitments (UFSAR Chapter 16) describing remedial action required in the event of a rise in groundwater as described in Section 2.4.13.5, the hydrologic design bases presented in the preceding sections do not necessitate emergency *procedures to ensure safety-related plant functions. In an emergency situation, industries and municipalities that utilize Lake Norman and the downstream area as a water supply are notified of possible contamination.*

2.4.15 References

1. *A Forest Atlas of the South, Forest Service, U.S. Department of Agriculture, 1969, pg. 8*
2. *Engineering Manual for Civil Works, Part III, Chapter 5, April, 1946, Plate 3.*
3. *"Hydrology of Spillway Design: Large Structures - Adequate Data" by Franklin F. Snyder, Journal of the Hydraulics Division, ASCE, Vol. 90, No. Hy3, May, 1964, pg. 239-259.*
4. *HUR 7-96 "Interview Report - Meterological Characteristics of the Probable Maximum Hurricane, Atlantic and Gulf Coasts of the U.S.," Office of Hydrology, Corps of Engineers, 1968.*
5. *Engineer Technical Letter No. 1110-2-8, "Computation of Freeboard Allowances for Waves in Reservoirs", Corps of Engineers, 1966.*
6. *Local Climatological Data - 1971 Annual Summary, Charlotte, N.C., U.S. Department of Commerce, NOAA.*
7. *D. A. Molitor, "Wave Pressures on Sea Walls and Breakwaters" Trans. ASCE., 1935, 1984.*
8. *"Shore Protection Manual", Coastal Engineering Research Center, Department of the Army, Corps of Engineers, 1973.*
9. *"Seismic Failures of Chilean Tailings Dams" by Ricardo Doyey and Leonardo Alvanez, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM6, November 1967, pp 237-260.*
10. *Chow, Ven Te, Open Channel Hydraulics, McGraw-Hill, Inc., New York, 1959.*
11. *Henderson, F. M., Open Channel Flow, Macmillan Company, New York, 1966, pp. 304-321.*
12. *Linsley, Ray K. and Franzini, Joseph B., Water Resources Engineering, McGraw-Hill, Inc., New York, 1964, pp. 493-530.*
13. *Streeter, Victor L., Fluid Mechanics, McGraw-Hill, Inc., New York, 1971, pp. 474-479.*
14. *Snyder, Franklin F., Hydrology of Spillway Design: Large Structures-Adequate Data, Journal of the Hydraulics Division, Paper 3915, Proceedings of the American Society of Civil Engineers, May, 1964, p. 243.*
15. *King and Brater, Handbook of Hydraulics, 5th Edition, McGraw-Hill Book Company, New York, 1963, p. 5-24.*
16. *Ibid. p. 4-30.*
17. *House Research Paper 19, "Snow Load Studies", Housing and Home Finance Agency, May, 1952.*

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2.5 Geology And Seismology

[HISTORICAL INFORMATION NOT REQUIRED TO BE REVISED]

Note:

This section contains references to Former FSAR Appendix 2A through 2H. These appendices contain historical site data needed by the NRC reviewers prior to Construction. This historical information is now archived as hard copies stored in a fireproof cabinet under the control of the Regulatory Compliance Group for the lifetime of the plant.

The station site is located 17 miles northwest of Charlotte, North Carolina. It is bordered on the west by the Catawba River and on the north by Lake Norman which is formed by Cowans Ford Dam adjacent to the site. The site is underlain by metamorphosed sedimentary, volcanic, and intrusive igneous rocks ranging in age from Paleozoic Era to the Triassic Period. Law Engineering Testing Company conducted the subsurface and seismic investigations for the design of the station. Approximately one hundred soil and rock borings have been made at the site, as well as several shallow test pits. Conventional seismic refraction techniques were utilized to aid in defining the nature and stratification of the soil and rock. Laboratory tests were conducted on soil and rock samples to determine their strength characteristics and other engineering properties. Results of these investigations are included in the attached appendices (2C Geology, 2D Subsurface and Foundations and 2E Seismology) prepared by Law Engineering Testing Company. These appendices provide a more detailed discussion of the subjects covered in the following sections. From these extensive investigations it was concluded that the subsurface conditions and seismic history of the site and region have no adverse effect on the design, construction, or operation of the station. During the design and construction phases of the station there were no unexpected or unusual conditions encountered, and the foundation recommendations were found to be adequate.

2.5.1 Basic Geologic And Seismic Information

2.5.1.1 Regional Geology

1. *The general site area lies near the center of a region known as the Piedmont Geologic Province. the Piedmont is a northeast trending zone from Georgia through Virginia which varies in width from about 80 to 120 miles. The region is marked by rolling hills and numerous small streams and rivers. A major river in the Piedmont region, the Catawba River, borders the site. Figure 2-1 is a map of the region showing its physiographic features.*
2. *The Piedmont Geologic Province is bordered on the east by the Coastal Plain Province and on the west by the Blue Ridge Province. The Coastal Plain generally consists of poorly consolidated sediments which include gravels, sands, clays, limestones, and marls. The Blue Ridge is a belt of meta-sedimentary rocks of the amphibolite facies in which igneous rocks were emplaced. Former Appendix 2C, Figure 2C-2 shows the geographic relationship between these geologic provinces.*
3. *The site is located in the Charlotte Belt, one of five northeast trending rock belts within the Piedmont Geologic Province. This belt consists of metamorphosed sedimentary and volcanic rocks with igneous rocks emplaced by several intrusive episodes during its early history. Former Appendix 2C, Figure 2C-2, Regional Geologic Map, shows the five belts within the Piedmont. For a more detailed discussion of these belts see Former Appendix 2C,*

Section 1.1. Former Appendix 2C, Figure 2C-4 is a geologic map of the area immediately surrounding the site.

4. *The majority of the rocks of the Piedmont were probably formed during lower to middle Paleozoic time or about 400 to 600 million years ago. (See Former Appendix 2C, Figure 2C-1 for a geologic time scale.) These metamorphosed rocks were formed by sequences of sediment and volcanic flows. During middle to late Paleozoic time, there were several episodes of intrusive activity in the Charlotte Belt. At the end of the Paleozoic Era, the Appalachian Orogeny built the Blue Ridge Mountains and elevated the Piedmont region to the east. During this disturbance, faulting took place on a major scale in the southern Appalachians and to a lesser degree in the Piedmont. By early Triassic time, the accelerated erosion had worn down much of the Piedmont to a broad undulating plain that cut across older complex rock structures. In late Triassic time, the earth's crust began to be broken by great normal faults that produced a chain of troughs and basins into which there was a rapid accumulation of sediments from the uplands. At the end of Triassic time, these new sediments were sheared and faulted. At the same time, considerable amounts of basic igneous rock material in the form of diabase dikes were intruded into these sediments and the surrounding crystalline rocks. The close of the Triassic Period marked the end of geologic activity in the Piedmont. A more complete discussion of regional history is contained in Former Appendix 2C.*

5. *The five northeast trending belts of the Piedmont are postulated to be stratigraphic units since they have several features in common. This stratigraphic sequence indicates that the various belts probably represent zones of different grades of regional metamorphism imposed on a thickness of Paleozoic volcanic and sedimentary rocks. The Charlotte Belt consists of gneiss, pegmatite, and schist with granitic gneiss being the principle intrusive unit which was probably emplaced during middle to late Paleozoic time. Distinctive masses of gabbro, diorite and syenite were similarly intruded into the existing Charlotte Belt rocks probably before the end of the Paleozoic Era. Most of the rocks within the belt have a distinctive granitoid texture and strong compositional layering, which strongly suggests they were probably derived from sediments. Geologic profiles at the site are shown in Former Appendix 2C, Figures 2C-17 thru 2C-26. Former Appendix 2C, Figure 2C-10 shows the location of these sections. A discussion of the other four belts is included in Former Appendix 2C, Section 1.1.*

6. *Several Pre-Triassic faults or structural belts were associated faults described in published literature are located within 75 miles of the site and are listed on page 2C-4 of Former Appendix 2C. These structures probably occurred during or immediately following the Appalachian Orogeny at the close of the Paleozoic Era, and there is no evidence of their movement since Triassic time, or 180 million years ago. The closest of the above mentioned structures to the site in the King's Mountain Belt. Regional mapping of this belt postulated several strike faults, however, none of these faults have been found in subsequent detailed mapping or in mining operations. Only a few very small thrust faults and transverse faults have been exposed in the quarries of this belt. For a detailed discussion of the King's Mountain Belt, see Former Appendix 2C, Section 1.2.*

7. *A discussion of regional groundwater conditions is provided in Section 2.4.13.*

2.5.1.2 Site Geology

1. *The physiography of the site is typical of that of the surrounding Piedmont Geological Province. The original land form of the site before construction began was that of rolling hills with a small seasonal stream running through it. This stream is now the site of the*

Stand-by Nuclear Service Water Pond and the Waste Water Collection Basin. The Catawba River borders the west side of the site and Lake Norman, which is formed by Cowans Ford Dam, borders the north side. Figure 2-4 is a topographic map of the site area showing the locations of the principal station facilities.

2. *The four major types appearing at the site are dark green meta-gabbro, light gray fine to medium grained granite, black and white fine grained diorite, and black and white coarse grained diorite. The petrographic analysis or detailed descriptions of the major rock types are shown in Former Appendix 2C, Table 2C-1. An examination of the rock cores at the site generally confirms the published geologic literature of the placement order and relative age of the rock types at the site. Former Appendix 2C, Figures 2C-17 through 2C-26 are geologic sections of core borings taken at the site, and Former Appendix 2D, Table 2D-1 is a summary log of the borings. For further discussion of these four major rock types see Former Appendix 2C, Section 2.2.*
3. *The geologic structure of the site is very old and complex. Due to the various episodes of igneous intrusions, the site area is typified by bodies of the various rock types highly interlayed both horizontally and vertically. Several joint patterns or orientations were detected from visible rock outcrops at the site. Former Appendix 2C, Section 2.3 further discusses these joint patterns and the structural geology of the site. Several minor shear zones, including slickensided surfaces, were noted in the core borings at the site. Tests show that the movement at these surfaces occurred during a geologic time when the various rocks were still in a semi-plastic state, or early Triassic Period. A test pit revealed a Triassic dike which cut across the slickenside surfaces. This independently dates the movements which provided the slickenside surfaces. A further discussion of these minor shear zones is given in Former Appendix 2C, Section 2.4. An ancient flood plain or high level terrace, probably of the Catawba River, exists in the higher portions of the site. Former Appendix 2C, Section 2.5 provides a discussion of this terrace. Former Appendix 2C, Figures 2C-11 and 2C-12 show contours of top of partially weathered rock and sound rock, respectively, and the location of major station structures superimposed on the rock topography.*
4. *The surface geology at the site with the location of the major station structures is shown in Former Appendix 2C, Figure 2C-9.*
5. *The geologic history of the site is typical of that of the region. During the middle to late Paleozoic time, there were several periods of intrusion of granite and diorite which left only small amounts of the parent rock, meta-gabbro and mica schist, in the Charlotte Belt including the site area. The slickenside surface at the site was caused during one of the last episodes of intrusion. The last intrusive activity in the area was in the form of Triassic diabase and other mafic dikes like the one found at the site. Former Appendix 2C, Figures 2C-17 through 2C-26 are geologic stratigraphic columns taken from core borings at the site.*
6. *For a plot plan showing the locations of the major structures of the station and the locations of all borings, see Former Appendix 2D, Figure 2D-2.*
7. *A geologic profile showing the relationship of the major foundations of the station to the subsurface materials as they are constructed, is shown in Former Appendix 2C, Figures 2C-17 thru -26.*
8. *Extent of excavations and backfill at the site are shown in Former Appendix 2D, Figure 2D-1 and Figure 2-64 and Figure 2-65. The compaction for all engineered backfill at the site was carefully controlled. In general, suitable backfill material, free of all brush, logs, boulders, etc., with a satisfactory moisture content was placed in nine inch layers, and each layer was*

compacted by the necessary number of passes of approved rollers required to attain not less than 95 percent of the Standard Proctor Density, ASTM D698-66T, Method C.

9. *From an engineering geology standpoint there are no local geologic features which adversely affect the station structures. A description of physical evidence concerning the behavior during prior earthquakes of the surficial geologic materials and the substrata underlying the site is given in Section 2.5.2.3. Joint patterns and shear zones have been discussed above, and it is concluded that there are no active geologic structures in the vicinity of the site. Where zones of irregular weathering of bedrock occurred, the weathered material was excavated and fill concrete was used under foundation structures, or piles were driven to suitable rock bearing for Category I structures. There are no rocks that are considered unstable because of their mineralogy, lack of consolidation, water content, or potentially undesirable response to seismic or other events. The construction activities at the site has no adverse effect from an engineering geology standpoint on the Category I station structures. There have been no known evidences of unrelieved residual stresses, such as "rock squeeze", or "pop-ups", or "rockbursts" in the Piedmont Region. Furthermore, no evidence of such occurrences was seen in the construction excavations at the McGuire site. Therefore, if unrelieved stresses do exist in the bedrock, they are of no consequence to the stability of the station structures.*
10. *Site groundwater conditions are discussed in Section 2.4.13.*
11. *Former Appendix 2C, Figures 2C-13, -14, and -15 are seismic refraction profiles located in Former Appendix 2C, Figure 2C-1 and used in the subsurface investigation. Results of up-hole seismic data are given in Former Appendix 2D, Section 4.1.*
12. *Elastic constants, for sound rock underlying the site, used in the foundation designs are given in Former Appendix 2D, Section 4, and a discussion of investigation and results is given in Former Appendix 2D, Section 2. Soil properties used in foundation designs are discussed in Former Appendix 2D, Section 2. Former Appendix 2G discusses soil properties used in the Stand-by Nuclear Service Water Pond Dam.*
13. *Safety-related criteria, analysis techniques used and safety against failure for materials underlying the foundations of the major station structures are discussed in Former Appendix 2D and for the Stand-by Nuclear Service Water Pond Dam are discussed in Former Appendix 2G.*

2.5.2 Vibratory Ground Motion

2.5.2.1 Geologic Conditions of the Site

A description of the lithologic, stratigraphic, and structural geologic conditions of the site is provided in Section 2.5.1.2 and those conditions of the region surrounding the site in Section 2.5.1.1. Refer to Sections 2.5.1.1 (Reference 4) and 2.5.1.2, (Reference 5) for regional and site geological history.

2.5.2.2 Underlying Tectonic Structures

Identification of tectonic structures underlying the site is provided in Section 2.5.1.2 and those for the region surrounding the site in Section 2.5.1.1. Regional and site tectonic structures are discussed further in Former Appendix 2E, Sections 2.3, 2.5, 2.6 and 2.7.

2.5.2.3 Behavior During Prior Earthquakes

From the lithologic, stratigraphic, and structural geologic studies, no physical evidence has been produced concerning the behavior during historically recorded earthquakes of the surficial geologic materials and the substrata underlying the site.

2.5.2.4 Engineering Properties of Materials Underlying the Site

Engineering properties of materials underlying the site are discussed in Former Appendix 2D, Sections 2 and 4 and Former Appendix 2G.

2.5.2.5 Earthquake History

Listed below are the historically reported earthquakes which probably produced an intensity III Modified Mercalli Scale (MM) or more at the ground surface in the site area.

Location	Date	Reported Intensity (MM) At Epicenter	Estimated Intensity (MM) at Site
New Madrid, Mo.	Dec. 1811, Jan. 1812, Feb., 1812	XII	VI+
Arvonia, Va.	Dec. 22, 1875	VII	III
Charlotte, N.C.	Dec. 12, 1879	V	IV
Charleston, S.C.	Aug. 31. 1886	IX	VI-VII
Giles, Co., Va.	May 31, 1897	VII-VIII	IV
Union Co., S.C.	Jan. 1, 1913	VII	V-VI
Western, N.C.	Feb. 21, 1916	VI	IV
Western, N.C.	Aug. 26, 1916	V	III-IV
Western, N.C.	July 8, 1926	VI	IV
Western, N.C.	Nov. 2, 1928	VI	IV
Lake Murray, S.C.	July 26, 1945	VI	IV
Western, N.C.	May 13, 1957	VI	IV
Chester, S.C.	Sept. 9, 1965	V	III-IV

Former Appendix 2E, Figure 2E-1 is a regional earthquake map showing the locations of the epicenters of the above earthquakes. A further discussion of regional and local seismicity is provided in Former Appendix 2E, Sections 2 and 3, respectively.

2.5.2.6 Correlation of Epicenters with Geologic Structures

There has been no evidence to indicate correlation between tectonic structures of the region surrounding the site and the historically reported earthquakes listed in the previous Section. For further discussion see Former Appendix 2E, Sections 2.1, 2.5, and 2.6.

2.5.2.7 Identification of Active Faults

There is no evidence of movement along the regional faults since Triassic time or about 180 million years ago. Therefore, it is concluded that there are no identifiable active faults in the region of the site. See Former Appendix 2E, Sections 2.5 and 2.6.

2.5.2.8 Description of Active Faults

This Section is not applicable to the region of the site. See Section 2.5.2.7.

2.5.2.9 Maximum Earthquake

Historical records indicate that the maximum earthquake intensity experienced at the site was the Charleston earthquake of August 21, 1886 with an estimated site surface intensity between VI-VII (MM). The maximum earthquake intensity which has occurred within the region is VII to VIII (MM). Within the region earthquakes of epicentral intensities of V and VI have occurred at average intervals of about 5 years, and earthquakes of epicentral intensities of VII have occurred in the region about once every 50 years. Therefore, the maximum expected earthquake intensity in the region is estimated to be between VII and VIII (7.5). Refer to Former Appendix 2E, Sections 4.1 and 4.2 for intensity and acceleration correlation.

2.5.2.10 Safe Shutdown Earthquake

The Safe Shutdown Earthquake for foundations on jointed rock and slightly weathered rock is 0.15g. Former Appendix 2E, Section 4.2 gives a complete discussion of the Safe Shutdown Earthquake. The Ground Response Spectra are shown on Former Appendix Figure 2E-4.

2.5.2.11 Operating Basis Earthquake

The Operating Basis Earthquake for foundations on jointed rock and slightly weathered rock is 0.08g. Apendix 2E, Section 4.1 gives a complete discussion of the Operating Basis Earthquake. The Ground Response Spectra are shown on Former Appendix Figures 2E-2A through -2D and 2E-3.

2.5.3 Surface Faulting

There is no geologic evidence of surface faulting within the Piedmont region or adjacent geologic regions that is even remotely related to the earthquakes that have occurred in historic times. Therefore, a design basis for surface faulting for this station is not applicable. See Former Appendix 2E, Section 2.6.

2.5.4 Stability of Subsurface Materials

2.5.4.1 Geologic Features

Geologic features of the site are described in Section 2.5.1.2.

2.5.4.2 Properties of Underlying Materials

Soil and rock properties of materials underlying the site used in the foundation designs are given in Former Appendix 2E, Section 2.6. Former Appendix 2D, Sections 2 and 4 and Former Appendix 2G.

2.5.4.3 Plot Plan

Former Appendix 2C, Figure 2C-10, 2D-1 and 2D-2 are plot plans of the site showing location of borings, seismic lines, and piezometers with the station structures superimposed. Former Appendix 2C, Figures 2C-17 through 2C-26 are profiles through the station showing the relationship of the subsurface materials to the foundations of structures.

2.5.4.4 Soil and Rock Characteristics

Former Appendix 2C, Figure 2C-15A provides a summary of the up-hole seismic data used to determine the shear wave velocity and other engineering values as shown in Former Appendix 2D, Section 4.1. Seismic line profile showing relation of soil penetration and rock RQD values to compression wave velocities are shown on Former Appendix 2C, Figures 2C-13 through 2C-15. Geologic sections of core borings are provided in Former Appendix 2C, Figures 2C-17 through 2C-26.

2.5.4.5 Excavations and Backfill

Extent of excavation and backfill at the site is shown in plan in Former Appendix 2D, Figure 2D-1 and profiles in Figure 2-64 and Figure 2-65. Compaction criteria are discussed in Section 2.5.1.2.

2.5.4.6 Groundwater Conditions

Groundwater levels are governed by the level of Lake Norman, which borders the site on the north. All excavations from about elevation 735 to 750 in the plant area required dewatering prior to and during the construction period. Refer to Former Appendix 2B, Section 2 and Former Appendix 2D, Section 3.2 for discussion of groundwater condition beneath the site.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

Foundation responses to dynamic loading are discussed in Former Appendix 2E, Section 4.4 and Section 3.7.1. Response spectra for the Operating Basis Earthquake and the Safe Shutdown Earthquake are shown on Former Appendix 2E, Figures 2E-2A through 2E-4.

2.5.4.8 Liquefaction Potential

As discussed in Former Appendix 2D, Section 4.1, soils beneath the site are not considered susceptable to liquefaction. These materials are weathered in place, well-graded soils which possess true cohesion as has been determined by triaxial shear tests.

2.5.4.9 Earthquake Design Basis

Former Appendix 2E, Section 4.4 discusses earthquake design basis.

2.5.4.10 Static Analysis

A discussion of foundation conditions and settlement of the various structure foundations is contained in Former Appendix 2D, Sections 2 and 4. Lateral pressures are discussed in Former Appendix 2D, Section 5.2 and shown graphically on Former Appendix 2D, Figure 2D-20.

2.5.4.11 Criteria and Design Methods

Design methods for Class I structures and a listing of applicable criteria and codes are discussed in detail in Sections 3.7 and 3.8.

2.5.4.12 Techniques to Improve Subsurface Conditions

All major Class I structures are supported on relatively unweathered rock with a RQD of 75 percent or better. Where such materials are below the nominal foundation level, the weathered rock-residual soil above was removed and replaced by fill concrete. This conforms to Duke Power practice on its larger fossil fuel steam stations.

2.5.5 Slope Stability

The raised portion of Cowans Ford Dam immediately north of the plant area is discussed in Former Appendix 2H. This section discusses the slope stability of the Standy-by Nuclear Service Water Pond Dam. Additional information on this dam is contained in Former Appendix 2G.

2.5.5.1 Slope Characteristics

Cross sections, as well as other details are provided in Former Appendix 2G, Figure 2G-1. Summaries of the properties of embankment and foundation soil and rock are furnished in Tables 6.1 and 6.2 and in Figures 6.1 through 6.14, all of Attachment 2G-A in Former Appendix 2G.

2.5.5.2 Design Criteria and Analyses

The slopes were designed for steady seepage at anticipated pool elevations due to probable maximum flood with maximum wave height (FS=1.5) and for steady seepage during a Safe Shutdown Earthquake (FS=1.05). The downstream slope was also designed for steady seepage and a railroad surcharge (FS=1.5), and the upstream slope was designed for the construction condition (FS=1.25). Sudden drawdown does not occur. Static design conditions were checked by computer using a modified version of the IBM 1620 General Program Library 9.2.020, "Slope Stability Analysis," originated by the Albuquerque District Corps of Engineers. The seismic analysis was performed according to the dynamic method described by N. M. Newmark, "Effects of Earthquakes on Dams and Embankments," Geotechnique, Volume XV, No. 2, June, 1965. An additional check for the factor of safety against seismic loading was made using the pseudo-dynamic analysis as recommended in the foundation report prepared by Law Engineering Testing Company, Section 6.10 of Attachment 2G-A of Former Appendix 2G.

2.5.5.3 Logs of Core Borings

Logs of core borings in dam foundation are furnished in Figure 6.3 of Attachment 2G-A of Former Appendix 2G. Engineering properties of embankment soils from all potential borrow areas are discussed in Section 6.6.5 of Attachment 2G-A of Former Appendix 2G.

2.5.5.4 Compaction Specifications

When the moisture content and other conditions in any layer are satisfactory, that layer is compacted by the necessary number of passes of approved rollers required to attain not less than 95 percent of the Standard Proctor Density, ASTM D698-66T, Method C. Further *discussion of compaction requirements and laboratory tests on which these requirements are based is contained in Section 6.6.5 of Attachment 2G-A of Former Appendix 2G.*

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2.6 Former Appendix 2 A-H

This material has been removed and is now archived in an electronic format in the Duke Application Environment (DAE), NEDL Portal, ELL Documents under the following file names:

Chpt. 2 Appendices removed from MNS FSAR in 91 Update-2A, 2B, 2C

Chpt. 2 App. Removed from MNS FSAR in 91 Update-2D

Chpt. 2 Appendices removed from MNS FSAR in 91 Update-2E & 2F

Chpt. 2 Appendices removed from MNS FSAR in 91 Update-2G

Chpt 2 Appendices removed from MNS FSAR in 91 Update-2G Con't & 2H

for the lifetime of the plant.

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