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Docket Nos. 50-413
and 50-414

Mr. H. Tucker
Vice President
Nuclear Production Department
Duke Power Company
422 South Church Street
Charlotte, North Carolina 28242

Dear Mr. Tucker:

Subject: Transmittal of Preliminary Draft SERs - Catawba Nuclear Station

Enclosed for your review and comment are the preliminary draft SERs for the following areas:

1. Hydrologic Engineering (Enclosure 1)
2. Geotechnical Engineering (Enclosure 2)
3. Geology and Seismology (Enclosure 3)

Your attention is directed in particular to any open items contained within these preliminary drafts. A principal objective of this transmittal is to provide for timely identification and resolution of any additional analysis, missing information, clarifications or other work necessary to resolve outstanding issues. Please contact the staff's Project Manager, Kahtan Jabbour, regarding the need for any meetings and telephone conferences to this end.

Your comments, including schedules for completion of any further analyses or other work associated with resolution of open items, are requested within four (4) weeks of this letter.

Sincerely,

Thomas M. Novak, Assistant Director
for Licensing
Division of Licensing

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Docket Nos.: 50-413/414

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for Licensing
Division of Licensing

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CATAWBA

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HYDROLOGIC ENGINEERING SUMMARY

CATAWBA NUCLEAR STATION

HYDROLOGIC ENGINEERING SUMMARY

CATAWBA NUCLEAR STATION

2.4 HYDROLOGIC ENGINEERING

2.4.1 Introduction

The staff has reviewed the hydrologic engineering aspects of the applicant's design, design criteria and design basis of safety-related facilities for Catawba. The acceptance criteria used as a basis for staff evaluations are set forth in Sections 2.4-1 through 2.4-14 of the Stahdard Review Plan (SRP), NUREG-0800. The acceptance criteria include the applicable GDC reactor site criteria (10 CFR 100), and standards for protection against radiation (10 CFR 20, Appendix B, Table II). Guidelines for implementation of the requirements of the acceptance criteria are provided in Regulatory Guides, ANSI Standards and Branch Technical Positions identified in SEP Section 2.4-1 through 2.4-14. Conformance to the acceptance criteria provides the basis for concluding that the site and facilities meet the requirements of Parts 20, 50 and 100 of 10 CFR with respect to hydrologic engineering.

2.4.2 Hydrologic Description

The Catawba Nuclear Station (CNS) is located in north-central South Carolina approximately 6 mi north of the town of Rock Hill, South Carolina. As shown on figure 2.4.1, the site is on the western edge of Lake Wylie on a

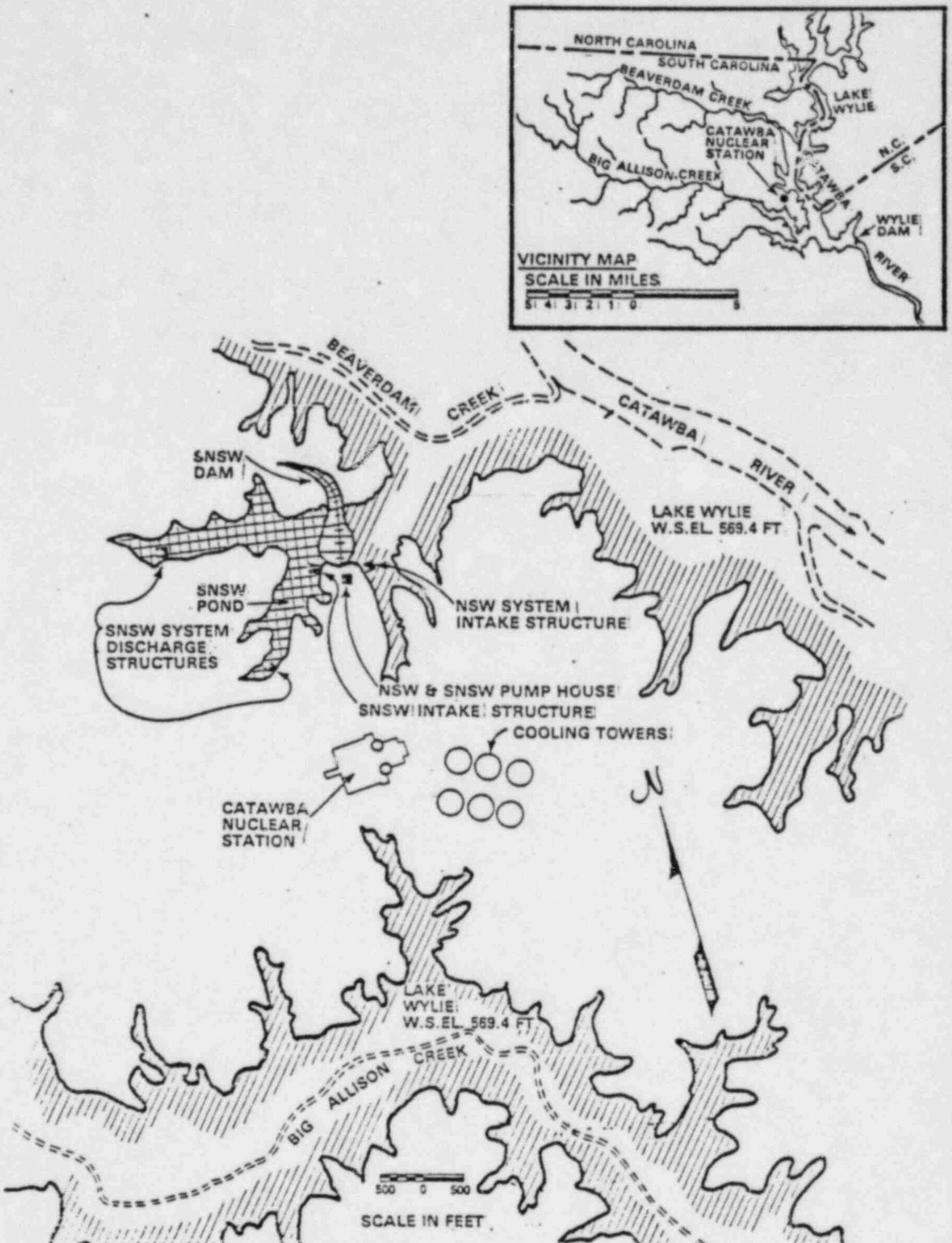

Figure 2.4-1 Principal Hydrologic Features

peninsula surrounded by the Lake Wylie backwater of Beaverdam Creek to the north and Big Allison Creek to the south. The yard grade is 593.5 ft above mean sea level (MSL) with a floor elevation of 594.0 ft, MSL.

The princip~~le~~al hydrologic features in the vicinity of Catawba Nuclear Station are Lake Wylie and the Catawba River system. Lake Wylie was formed in 1904 by the construction of Wylie Dam Across the Catawba River. Lake Wylie extends north from Wylie Dam for 28 mi upstream along the course of the Catawba River to Mountain Island Dam. Lake Wylie also extends approximately 5 mi up the South Fork Catawba River. At full pond elevation of 569.4 ft MSL, Lake Wylie has a volume of roughly 281,900 acre-ft and an average depth of 22.5 ft. The total watershed of the lake is 3020 sq. mi yielding a mean discharge of 4400 cfs.

The Catawba River heads in the Blue Ridge Divide near Old Fort, North Carolina and flows approximately 240 mi east and south to Lake Wateree where it joins the Wateree River near Camden, South Carolina over 35 mi south of the plant site. There are eleven hydroelectric dams and reservoirs in operation on the Catawba River system with no additional structures planned. There is only one uncontrolled reach of the Catawba River upstream from Lake Wylie.

Significant hydrologic safety-related plant features include (1) the Nuclear Service Water (NSW) System intake structure, (2) the Standby Nuclear Service Water (SNSW) Pond, (3) the SNSW Dam, and (4) the SNSW intake and discharge structures. The SNSW Pond is designed to function as the ultimate heat sink (UHS) providing essential cooling water for safe plant shutdown in the event water is not available from Lake Wylie. Two seismic Category 1 intake structures provide water for the NSW System and the SNSW System. The NSW System intake is located in Lake Wylie and the SNSW System intake is located in the SNSW Pond. The NSW and SNSW Pumphouse is located near the south abutment of the SNSW Dam on a peninsula of land separating one arm of the UHS from Lake Wylie. The pumphouse is connected to the NSW intake in Lake Wylie by a single seismic Category 1 transport line and to the UHS intake by two seismic Category 1 redundant lines.

The nearest (to the plant) downstream municipal surface water users are Rock Hill, South Carolina and Fort Mill, South Carolina. Surface water intakes for these two communities are located on the Catawba River approximately four river miles downstream of Wylie Dam or nine river miles downstream from the plant discharge. Two communities, Mount Holly and Belmont, North Carolina, take their raw water supplies directly from Lake Wylie roughly 31 mi and 20 mi upstream from the plant discharge point, respectively. Including Rock Hill and Fort Mill, South Carolina there are 13 municipal and/or industrial Catawba River Surface water users with intakes between Lake Wylie Dam and Wateree Reservoir Dam. Their combined average daily use exceeds 35 million gallons per day (mgd).

There are no "aquifers" identifiable by any name in the area. Ground water is usually found at water table conditions near the site. Soil permeabilities range from almost 400 ft/yr to near 0 ft/yr. Rock permeabilities are variable with high values near 1000 ft/year and low values near 50 ft/yr. Ground water is seldom found at depths greater than 300 ft because the frequency and size of fractures in the rock decrease with depth. Ground water will not be used in station operations; however, one well drilled during construction will be maintained as an emergency source of potable water. This well is located upgradient from the radwaste building.

There are few users of ground water in the vicinity of the Catawba Power Station. A survey of the area counted 12 wells and one spring within a one mile radius of the site. All of these wells are located near the shores of Lake Wylie and draw water that is hydraulically connected to the lake water. Local ground water use is limited to domestic users. Ground water use is not likely to increase significantly in the future due to the low yields of wells and the proximity of the lake.

The staff has reviewed the applicant's hydrologic description in the FSAR in accordance with SRP Section 2.4.1 and concludes that the description satisfies the requirements of GDC-2 and 10 CFR Part 100.

2.4.3 Flood Potential

The largely controlled Catawba River and its tributaries are the hydrologic features which may impact Catawba Nuclear Station safety-related components. The Catawba River is approximately 240 stream miles long and, along with its tributaries, drains approximately 4750 sq. mi of watershed above Wateree Dam. In downstream order, the sequence of control structures on the Catawba River is as follows: (1) Bridgewater Dam at mile 206; (2) Rhodhiss Dam at mile 170.5; (3) Oxford Dam at mile 152; (4) Lookout Dam at mile 143; (5) Cowans Ford Dam at mile 109; (6) Mountain Island Dam at Mile 94; (7) Wylie Dam at mile 66; (8) Fishing Creek Dam at mile 27; (9) Great Falls Dam at mile 23.5; (10) Rocky Creek Dam at mile 22; and, (11) Wateree Dam at mile 0. An 18 mi reach of the Catawba River downstream from Bridgewater Dam to the backwater of Rhodhiss Reservoir is the only reach of the Catawba River above Lake Wylie not affected by the backwater of a downstream reservoir.

The applicant analyzed the potential for site flooding which could occur as a result of the five following flood producing phenomena:

1. Probable maximum flood (PMF) resulting from the probable maximum precipitation (PMP) positioned appropriately over the Lake Wylie drainage area;
2. PMF resulting from the PMP positioned critically over the tributary area that contributes to the SNSW pond.
3. Locally intense precipitation occurring over the immediate project site;
4. The standard project flood (SPF) passing through Lake Wylie combined with the failure of one of the upstream dams due to an operating basis earthquake (OBE); and,
5. Probable maximum surge and seiche flood caused by the probable maximum hurricane.

The staff has reviewed the material presented by the applicant and concludes that these are the only credible sources of potential flooding at the site.

2.4.3.1 Stream Flooding

The maximum flow recorded for the Catawba River near Rock Hill, South Carolina is 151,000 cfs on May 23, 1901. The period of record for the gauge near Rock Hill is 1895 to 1903 and 1942 to present. Two major flood events not recorded occurred in 1916 and 1940 with peak flows estimated at Lake Wylie Dam of 299,400 cfs and 169,160 cfs, respectively. The maximum surface water level for Lake Wylie as a result of the August 1940 flood was 575.0 ft, MSL which is 5.6 ft above the full reservoir elevation of 569.4 ft, MSL. During the 1940 flood, all upstream reservoirs (i.e., Lake Wylie and above) were operating except Lake Norman which become operational in 1963 when Cowens Ford Dam was completed. The applicant states that the six reservoirs above the plant and Lake Wylie, including Lake Norman, have a combined storage capacity of nearly 1.5 million acre-ft and therefore the flood peaks on the main stem of the Catawba River are significantly modified and do not represent the uncontrolled flood potential of the Catawba River Basin. The staff concurs with the applicant and further notes that the maximum surface water elevation of 575.0 ft, MSL for Lake Wylie as a result of the 1940 flood of record is 18.5 ft below the plant yard grade of 593.5 ft, MSL. The staff reviewed the applicant's PMF analyses for Lake Wylie at the construction permit (CP) stage. At that time the staff concurred with the applicant's analyses and concluded that the plant site would not be flooded during a PMF on the Catawba River. The applicant reevaluated the PMF for Lake Wylie in the FSAR. The resulting maximum static water surface elevation of Lake Wylie at the Plant is somewhat lower (581.1 ft, MSL) than that reported at the CP stage (583.0 ft MSL). The staff has reviewed the material presented in the FSAR in accordance with the procedures described in SRP Sections 2.4.2 and 2.4.3. There is no new information that would lead the staff to change its earlier concusion. The staff therefore concludes that the station meets the requirements of GDC 2 with respect to flooding from the PMF in Lake Wylie.

The applicant determined the PMP occurring over the 410 acre SNSW Pond drainage and routed the resultant PMF through the pond to determine the maximum static water surface elevation. The applicant calculated a maximum static water surface elevation of 581.3 ft, MSL for the pond assuming a Lake Wylie elevation at its normal maximum of 569.4 ft, MSL. The applicant calculated wave height and runup caused by a 40 mph wind at 1.0 ft for a maximum water elevation of the pond at the SNSW Dam of 582.3 ft, MSL which is 12.7 ft below the crest. The staff performed an independent analyses of the local PMF on the SNSWP drainage area during the CP stage and found that the applicant's analyses were conservative. The staff has reviewed the FSAR and found no additional information that would lead them to change their earlier conclusion.

2.4.3.2 Local Intense Precipitation

The CNS yard drainage system is separated into subcatachments, each of which has a catch basin and a runoff inlet. Plant yard runoff is conveyed from the catch basins to Lake Wylie via a pipe network. Conservatively assuming 100 percent rainfall runoff and an instantaneous time of concentration, the yard drainage system is designed for a rainfall intensity of 4.0 in./hr. Rainfall intensities greater than 4.0 in./hr will exceed the drainage system capacity and cause ponding of water in the plant yard. The applicant evaluated the effect of runoff from the local probable maximum precipitation (PMP) on safety related structures. The applicant computed the inflows to the plant yard during the PMP by the rational method assuming 100% runoff and that water flows directly from roofs of plant buildings to the plant yard. Drainage from the plant yard occurs as orifice flow through the catch basin inlets and as sheet flow over the east and south ends of the yard. The applicant's analysis assumed the catch basin inlets were 61% clogged. The sheet flow over the east and south ends was computed as weir flow assuming a weir coefficient of 3.13. Based on these assumptions, utilizing the Puls graphical routing method, the applicant computed the maximum water elevation during the local PMP to be 593.7 ft MSL, 0.3 ft below the plant floor level.

The applicant did not provide any justification for assuming a 61% blockage of the catch basin inlets. There is no program to assure that the inlets do not become clogged with silt and debris; consequently, the staff concludes that assuming 61% blockage may not be conservative. The staff also concludes that use of a weir coefficient as high as 3.13 may also not be conservative.

As a result the staff performed an independent PMP analysis conservatively assuming:

1) 100% clogging of the catch basin inlets, and 2) a weir coefficient of 2.7 for the sheet flow over the east and south ends of the plant yard. Using these assumptions and the same approach as the applicant, the staff computed a maximum water surface elevation of 594.3 ft MSL, 0.3 ft above the plant floor level. Auxiliary flood protection measures such as sealed entryways or interior sumps are not discussed by the applicant. Therefore, in conclusion, the applicant has not demonstrated that safety related facilities are adequately protected against effects of the local PMP. The staff will require that the applicant analyze the effects of a ponded water depth of 594.3 ft MSL, which is 0.3 ft higher than the plant floor elevation, on safety-related structures and components. This analysis should identify all plant openings which are below elevation 594.3 ft msl and any safety-related components or equipment which could be affected by 0.3 ft of water. Alternately, the applicant may consider modifying the final site grading plan to assure more rapid runoff of precipitation away from safety related structures.

The roofs of safety-related structures, except the Reactor Building, are designed so that water runs directly off the roofs with no accumulation. The Reactor Building roof drainage system is designed for a rainfall intensity of 5 in/hr beyond which pondage occurs. Above elevation 711.34 ft MSL water flows directly off the roof of the Reactor Building. The Reactor Building roof is designed for live loadings due to roof pondage.

2.4.3.3 Dam Failure Flooding

The applicant investigated the effect of combinations of upstream dam failures, coincident with the standard project flood (SPF) occurring over each drainage area, on the water surface elevation of Lake Wylie. The seismic failure of Cowans Ford Dam, which forms Lake Norman, coupled with the SPF over the Wateree drainage area with Lake Norman control elevation at 761 ft, MSL (1.0 ft above full pond elevation) resulted in a maximum static water surface elevation for Lake Wylie of 592.4 ft, MSL. This is the highest flood elevation of all flood producing phenomena investigated by the applicant, and thus, is the Design Basis Flood (DBF). The applicant superimposed on the maximum still water surface elevation the maximum wind setup and wind-induced wave runup which resulted in a maximum water elevation on the plant yard of 593.9 ft, MSL. Although this is 0.4 feet above yard grade, all openings to safety related systems and components are located at a minimum elevation of 594.0 ft, MSL. The staff reviewed the applicant's DBF analyses at the CP stage and concluded that the input assumptions were conservative and that the analysis was representative of the most severe flooding conditions that may be expected to occur at The Catawba site. The staff's independent analysis indicates that the wave runup on the downstream face of the SNSW Dam is slightly higher than calculated by the applicant and results in insignificant overtopping of the dam. The staff calculated a maximum run-up elevation at the SNSW Dam of 595.2 ft, MSL resulting from a 40 mph sustained overland wind superimposed on the maximum Lake Wylie still water elevation. The staff concludes that the SNSW Dam crest riprap is sufficient to withstand the 0.2 ft of overtopping caused by wind-wave activity coincident with the Lake Wylie maximum water surface elevation.

2.4.3.4 Surge and Seiche Flooding

The applicant considered two possible hurricane tracks for determining the maximum surge and seiche water levels at the plant site. The maximum wind speeds calculated by the applicant for the two cases are 101.5 mph and 116.0 mph, respectively. The fastest mile wind speed observed in Charlotte, North Carolina, approximately 15 mi northeast of the site, is 74 mph caused by the hurricane that moved across South Carolina, July 14, 1916. The applicant determined the maximum water surface elevation gain due to the combined effects of wind tide, wave runup and differential pressure caused by the occurrence of the maximum probable hurricane to be 8.4 ft. This gain superimposed on the Lake Wylie full pond elevation of 569.4 ft, MSL results in a maximum surface water elevation of 577.8 ft, MSL which is well below the elevation of any safety-related features at the plant. The staff concludes, based on the procedures presented in SRP Sections 2.4.2, 2.4.5, and 2.4.10, that the threat of hurricane induced flooding has been adequately considered by the applicant and that the plant has been satisfactorily protected against flooding caused by any hurricane.

Because of the plant's non-coastal location, there is no threat of tsunami flooding. Also, ice accumulation occurs only over short and infrequent time periods due to the moderate climate. There is no threat of ice flooding severe enough to cause concern for safety-related plant components.

The staff reviewed the information regarding floods presented by the applicant in the Catawba Nuclear Station FSAR in accordance with procedures established in SRP Sections 2.4.2 to 2.4.7. Where it was considered necessary, the staff performed independent analyses and evaluations. The staff concludes, except as noted, that the applicant's analyses of flood impacts on safety-related features of the Catawba Nuclear Station are reasonable and complete, and furthermore, that the requirements of GDC 2 are met with respect to all potential types of flooding with the exception of flooding due to local intense precipitation.

2.4.4 Cooling Water Supply

Lake Wylie provides cooling water for use during both normal and emergency operation. To provide a backup source of emergency cooling water in the event that Wylie Dam should fail, the SNSW pond shown on figure 2.4.1, has been formed by constructing a seismic category I dam across an arm of Lake Wylie.

The staff has reviewed the material presented by the applicant using the procedures described in SRP section 2.4.11 and concludes that the two water sources (Lake Wylie and the SNSW pond) meet the guidelines of Regulatory Guide 1.27, "Ultimate Heat Sink for Nuclear Power Plants", with regard to providing a high level of assurance that at least one cooling water source will be available for emergency operation of the CNS.

2.4.4.1 Normal Cooling Water Supply

The normal source of cooling water for the CNS is Lake Wylie. There are seven reservoirs, including Lake Wylie, on the Catawba River upstream from Wylie Dam. All seven reservoirs are owned and operated by Duke Power Company and no additional impoundments are contemplated for the Catawba River above the plant site. The applicant states that the minimum flow downstream of Mountain Island Dam into Lake Wylie is 314 cfs. According to the applicant, the release from Mountain Island Dam is roughly 50 percent of the inflow to Lake Wylie with the remaining 50 percent divided equally between the South Fork Catawba River and lesser tributary streams. During normal operation of two units at full power, the CNS will consumptively use a flow of approximately 59 cfs which will be supplied by Lake Wylie.

The applicant calculated a minimum average expected inflow to Lake Wylie of 516 cfs by combining the minimum release required, by Federal Power Commission (FPC) license, from Mountain Island Dam (314 cfs) with a 7 day-10 year low flow of 202 cfs from the South Fork Catawba River and other tributaries. The total consumptive use of water from Lake Wylie due to power generation was estimated to be 70 cfs. This total includes a use of 11 cfs by Plant Allen, which is a fossil fuel plant that uses Lake Wylie as a source of cooling water, and a use of 59 cfs by the CNS. The natural evaporation from Lake Wylie was estimated to be about 60 cfs. This results in a total water loss from Lake Wylie of 130 cfs (70 cfs + 60 cfs). The minimum release required by FPC license from Lake Wylie is 411 cfs. Thus the total water lost from the Lake is 541 cfs (130 cfs + 411 cfs). The minimum expected inflow to Lake Wylie (516 cfs) minus the total water loss (541 cfs) results in an overall loss of 25 cfs. The useable Lake Wylie storage between full pool (569.4 ft MSL) and maximum drawdown (559.4 ft MSL) is 107,200 acre-ft. This volume provides sufficient water for almost 6 years of operation assuming a water loss of 25 cfs. In the above analysis, it was assumed that the inflow to Lake Wylie would be 516 cfs. A more conservative assumption is zero inflow to Lake Wylie. Under this condition there would still be sufficient water in Lake Wylie to permit the CNS to operate for about 100 days.

Using the procedures described in SRP Section 2.4.11, the staff concludes that Lake Wylie provides a highly reliable source of cooling water so that the SNSW pond will be needed only on a very infrequent basis. The staff concludes that the requirements of GDC-44 with respect to normal operating conditions have been met.

2.4.4.2 Emergency Cooling Water Supply

Lake Wylie will be the normal source of emergency cooling water for use in the NSW system. In the unlikely event that Lake Wylie is not available, the SNSW Pond will provide the cooling water needed to dissipate the heat rejected during either a LOCA in one unit and the coincident normal shutdown in the other unit or normal shutdown of both units.

The SNSW Pond which is located about 2800 ft north of the station, was formed by construction of a seismic category I earthfill dam (SNSW dam) across an arm of Lake Wylie. The SNSW pond is designed to fluctuate between a full pond elevation of 571.0 ft MSL and maximum drawdown elevation of 567.0 ft MSL. At elevation 571.0 ft MSL, the SNSW pond has a useable storage volume of about 560 acre-ft and a surface area of about 46 acres. The applicant estimates that the volume will be depleted by about 10 acre-ft of sediment during the 40-year plant life. The staff has reviewed the applicant's sediment analysis and concludes that a 10 acre-ft reduction in the storage volume of the SNSW Pond is a conservative estimate. However, the staff will require an analysis or discussion on how this sediment accumulation will affect the operation of the SNSW intake structure.

The applicant analyzed the ability of the SNSW Pond to provide a 30-day supply of cooling water at or below a design-basis temperature of 95°F under the most severe meteorological conditions of record. The applicant's analysis predicted a maximum temperature of 95°F and a maximum 30-day water loss of about 51 acre-ft. Since the design-basis temperature is 95°F and the SNSW Pond has a maximum volume of about 560 acre-ft, the applicant concluded that the SNSW Pond is capable of providing emergency cooling water for at least 30 days. The applicant thus concludes that the CNS meets all the recommendations set forth in Regulatory Guide 1.27, "Ultimate Heat Sinks for Nuclear Power Plants".

Using the conservative methods in NUREG-0693, "Analysis of Ultimate Heat Sink Cooling Ponds", the staff also analyzed the performance of the SNSW Pond. Various thermal mixing conditions were considered including completely mixed, plug flow, and thermal stratification. Although the simulations indicated that the volume of water present in the pond is sufficient to supply the NSW System for more than 30 days, the resultant peak pond temperatures exceeded the design maximum temperature of 95.0°F for each thermal mixing condition analyzed. The maximum simulated pond temperature was 106.4°F assuming a completely mixed pond.

The staff recognizes that the procedures described in NUREG-0693, for use in simulating performance of cooling ponds, are intended to give conservative (high side) estimates of water loss and/or temperatures. The staff is continuing its analysis of the performance of the SNSW Pond; however, based on the results of its analysis to date, the staff concludes that the SNSW Pond may not be capable of maintaining the service water system temperature below the design basis as recommended in SRP Sections 2.4.11 and 9.2.5 and Regulatory Guide 1.27. The staff therefore concludes that at this time, the SNSW Pond does not meet the requirements of GDC-44.

2.4.5 Ground Water

The pre-construction depth to the water table ranged from 10 ft to 40 ft. The water table followed the ground surface, mounding near the plant site and intersecting the ground surface at the lake. The regional ground water flow system closely follows the stream system. Ground water in the plant locality usually flows for very short distances before being intercepted by a surface-water body.

The post construction ground water table has been considerably changed due to the operation of the permanent dewatering system. The dewatering system has lowered the water table at the plant to a depth of approximately 50 ft below ground surface, which is 25 ft below the normal level of Lake Wylie. It has therefore altered the groundwater gradient near the site. However, based on groundwater elevations taken from observation wells surrounding the reactor building since the dewatering system has been operating, the staff concludes that the radius of influence of the dewatering system will not extend outside the plant boundaries.

The permanent dewatering system is designed to keep ground water levels at or below the level of the foundation mat and basemat walls. This is accomplished by using a system of seismic category I underdrains and exterior walls drains connected to sumps. The underdrain system consists of a series of interconnected flow channels spaced approximately 20 ft apart~~,~~. These channels are placed under the structural mats to relieve residual hydrostatic pressure which may develop in the foundations away from the exterior wall drains. Each flow channel has a minimum cross sectional area of 0.157 sq. ft and is constructed of lumber treated with a preservative. The flow channels are located on the surface of the excavated rock except in areas where unfavorable jointing of the rock resulted in an irregular rock surface. It was not practical to fit the wood-framed flow channel to these irregular surfaces so the flow channels were placed on a leveling course of concrete. The channels located on fill concrete rather than directly on rock are provided with 2 5/8 in. diameter holes that penetrate the concrete and the underlying rock a minimum of 3 ft. The purpose of the drilled holes is to provide a means for ground water to flow into the underdrain system. The channels are laid out in a grid pattern under the reactor building and the auxiliary building with the exception of some low-lying pits. The drains terminate at the walls of the plant with the drainage toward the perforated pipes that carry the flow to the sumps. All of the flow channels drain to these pipes which carry the water to three sumps located adjacent to the auxiliary building. Two of the sumps are 10 ft by 10 ft by 15 ft deep while the third sump is 17 ft by 17 ft by 12 ft deep. The storage capacity of the sumps is 48,000 gallons. The exterior wall drains are continuous 2 ft zoned sand and stone filters that extend from the bottom of the excavation to an elevation of 589.0 ft, MSL. These drains are connected to the same perforated pipes that lead to the three sumps. Two 300 gallons per minute (gpm) seismic category I pumps are used to maintain the water levels in each of the sumps. One pump starts automatically when the water level rises to

536.0 ft MSL. If the first pump fails to start or the water level rises to 538.0 ft, MSL, the second pump will automatically start and an alarm will sound. Since the three sumps are interconnected, there are six pumps available, each capable of pumping more than eight times the measured infiltration rate. Ground water collected in the sumps is pumped to the yard drainage system which drains to Lake Wylie.

If the dewatering system were to fail, the estimated time for ground water to recover to yard grade is about 56 days. Even if the plant dewatering system could not be repaired in this length of time, according to the applicant's analysis, the plant is capable of withstanding the resultant hydrostatic and uplift forces. Any leakage into the plant would be handled by the floor drain system sumps and pumps, which discharge into Lake Wylie.

In accordance with SRP Section 2.4.12 the applicant postulated breaks in underground piping and analyzed the effects of these breaks on the dewatering system. The applicant concluded that failure of a NSW pipe would induce the greatest quantity of water into the dewatering system but concluded that the pumping and storage capacity of the dewatering system would be sufficient to handle the additional water from the NSW pipe break. The applicant also analyzed postulated breaks in the Condenser Circulating Water (CCW) piping both inside and outside the Turbine Building. For a postulated CCW pipe break inside the Turbine Building, the applicant has designed a reinforced concrete wall to contain the water from the pipe break to prevent flooding of the Auxiliary Building. For a failure of the CCW piping outside the Turbine Building, the applicant concluded that there would be no effect on the permanent dewatering system because it is isolated from the CCW piping by a nominal 17 ft minimum thickness of impermeable backfill material.

The staff has reviewed the applicant's analysis of a postulated failure of the NSW pipe and concludes that the permanent dewatering system would be capable of maintaining groundwater levels at or near the base of the foundation mat as designed.

The staff has also reviewed the applicant's analyses of pipe breaks in the CCW piping. A description of the staff's review and conclusions regarding a CCW pipe break inside the turbine building is presented in section 10.4.5. For a break outside the turbine building the staff does not agree with the applicant's comment that the CCW pipes are isolated by impermeable backfill. Section 2.5.4.5.4.2 of the FSAR describes the backfill as crushed stone. This material is highly permeable. The staff will require an analysis of a postulated failure of the CCW piping at critical locations outside the plant. This analysis should also evaluate the floatation (buoyancy) forces which would be induced, particularily on the diesel generator building, by water from a CCW pipe break.

As described above, some flow channels have been placed on fill concrete instead of on the excavated rock. Drain holes have been drilled through the fill concrete to rock. The staff concludes that unless these drain holes intersect rock joints, they may not be effective in relieving the residual hydrostatic pressures under the foundations. Thus the staff will require that the applicant show ~~by pertinent analysis~~ that the drain holes that penetrate the fill concrete are effectively relieving the hydrostatic pressures as intended.

Based on the review procedures presented in Section 2.4.12 of the SRP, including Branch Technical Position HGEB-1, the staff is unable, at this time, to conclude that the dewatering system meets the requirements of GDC 2, 10 CFR 100, and Appendix A thereto, 10 CFR 50 and GDC 4.

2.4.6 Technical Specifications and Emergency Operation Requirements

According to the FSAR the applicant has committed to establishing a continuous monitoring system of the dewatering system. The monitoring system consists of 12 monitoring wells located around the perimeter of the reactor and auxiliary buildings. Six of these wells have continuous monitoring devices with 3 points of alarm that will alert the plant operator to any rise in ground water in the dewatering system. The other 6 wells are available to dewater the filter system if water levels were to rise. Fifteen additional wells are located around the plant site to complement the monitoring system. Although the applicant has committed to the dewatering system in the FSAR, the monitoring system procedures have not been included in the plant's Technical Specifications. The technical specifications should include provisions for plant shut-down and emergency action to reduce the water levels should the dewatering system fail.

2.4.7 Dispersion, Dilution, and Travel Time of Accidental Releases of Liquid Effluents

SRP Section 2.4.13 sets forth criteria and procedures for the analysis of accidental releases of liquid effluents into ground and surface waters. Using these the staff analyzed a postulated failure of the Recycle Evaporator Bottoms Tank to determine the potential for contamination of surface and ground water supplies. As described in Section 15.7.3, this tank was selected for analysis because it contains the highest potential concentrations. The staff's analysis assumed water from the tank spill reaches the Allison Creek arm of Lake Wylie through the plant underdrain system within hours of the spill. This requires that the fluid has immediate access to the underdrain system via cracks in the auxiliary building concrete floor or exterior walls. The plant underdrain system is capable of discharging the waste in a maximum of eight minutes and it was assumed that mixing of the waste with Lake Wylie water is rapid.

Assuming no adsorption occurs, all mitigation is dependent on dilution. Water enters the Allison Creek arm of Lake Wylie via Allison Creek, Big Branch Creek, and plant service water effluent with a conservatively estimated total discharge of 150 cfs. Given the volume of the Allison Creek arm of Lake Wylie and the effective volume of the main portion of Lake Wylie downstream of the plant site the concentrations of all contaminants at the outlet of Lake Wylie from the postulated tank spill would be less than 20 percent of the limits shown in Table II of Appendix B in 10 CFR 20.

The staff concludes, based on conservative estimate of contaminant concentrations, that an accidental spill of radioactive liquid will not result in concentrations above 10 CFR 20 limits at the nearest downstream water intakes. This conclusion was reached in accordance with acceptance criteria set forth in Section 2.4.13 of the SRP.

2.4.8 Conclusions

According to procedures outlined in the SRP, the staff has reviewed the design of the CNS in regard to hydrologically and hydraulically-related plant safety features. On the basis of this review, the staff concludes that any large-scale river flooding, either naturally occurring or seismically induced, poses no threat to the safe operation of the plant or the integrity of the site. The staff, however, is unable to conclude that local flooding will not threaten the CNS. Therefore, the staff concludes that the station meets the requirements of GDC 2 with respect to potential flood hazards except for the unresolved issues concerning local flooding.

The staff has analyzed the availability of water for plant cooling purposes during diminished flow periods and concludes that adequate storage is present in Lake Wylie to maintain safe plant operation over any reasonable drought period as required by GDC 2 with respect to cooling water availability.

The staff further concludes that the Catawba Nuclear Station UHS has been properly designed to withstand any flooding event and that sufficient supply is available for the safe shutdown of the plant. However, based on the staff's review of the thermal performance of the UHS in accordance with procedures described in Section 9.2.5 of the SRP, they conclude that the SNSW Pond may not be capable of satisfying the maximum design temperature requirements established in Regulatory Guide 1.27.

The staff has not completed its review of the permanent dewatering system. The staff will require additional information concerning the effects of underground piping failures on the dewatering system and assurances that the drain holes that penetrate fill concrete are effectively relieving residual hydrostatic pressures as intended. In addition, the plants Technical Specifications should describe shutdown procedures, and emergency actions to be used to reduce water levels should the dewatering system fail.

Finally, the staff concludes that the concentration of radionuclides passing Wylie Dam following a postulated liquid radwaste tank spill will be below the 10 CFR 20 limits. Therefore, the plant meets the requirements of 10 CFR 100 with respect to potential accidental release of contaminated liquid effluents.

CATAWBA NUCLEAR STATION, UNITS 1 & 2

DOCKET NOS. 50-413/414

WORKING DRAFT SER INPUT - GEOTECHNICAL ENGINEERING

The following sections are for inclusion in the working draft Safety Evaluation Report (SER). The stability of subsurface materials (FSAR Section 2.5.4), the stability of slopes (FSAR Section 2.5.5), and the stability of embankments and dams (FSAR Section 2.5.6) have been evaluated in accordance with the criteria outlined in Appendix A of 10 CFR, Part 100, Reg. Guide 1.70, Revision 3, (Nov. 1978) and in Sections 2.5.4 and 2.5.5 of the Standard Review Plan (SRP), NUREG 0800 (Rev. 2 - July 1981).

2.5.4 Stability of Subsurface Materials and Foundations

2.5.4.1 Site Conditions

The plant site is located in the northeastern portion of York County in South Carolina on a peninsula bounded by Beaver Dam Creek to the north, Big Allison Creek to the south, the main body of Lake Wylie to the east, and private property to the west. Rock Hill, South Carolina, is located approximately six miles south of the site, and Charlotte, North Carolina, is located approximately 10 miles east-northeast of the site. Surface elevations in the site vicinity range from about 570 feet (Lake Wylie) to 640 feet above mean sea level (El. 570 to El.640). The powerhouse yard grade is at

El.593.5. Lake Wylie is the normal source of Nuclear Service Water (NSW). The emergency cooling water supply is obtained from the Standby Nuclear Service Water Pond (SNSWP) that is formed by a north-south oriented earth dam across an existing cove of Lake Wylie, as shown in Fig. 2.4-1 (Section 2.4) of this report.

A list of seismic Category I structures of the two-unit Catawba Nuclear Station is given in Table 3.2.1-1 of the applicant's Final Safety Analysis Report. The major category I structures are listed below:

A. Structures Founded on Rock

Reactor Buildings

Auxiliary Building

Diesel Generator Buildings

Outside Dog Houses

New Fuel Storage Pools

Spent Fuel Pools

NSW Pump Structure

Main Steam Line Supports

B. Structures Founded on Partially Weathered Rock

Above ground storage tanks (reactor make-up and refueling water storage tanks)

Pipe trench to "Above ground storage tanks"

SNSW and NSW intake structures

SNSW and NSW discharge structures

SNSWP dam (portions of the dam rest on saprolite)

C. Structures Founded on Residual Soil or Compacted Backfill

Diesel Fuel Oil Tanks (buried)

SNSW and NSW pipelines (buried)

SNSW pond outlet pipe (buried)

NSW electrical conduit manholes (some are founded on partially weathered rock).

The bedrock at the site consists primarily of adamellite that is a metamorphosed igneous rock of the Charlotte belt. A secondary rock type also exists in the form of discontinuous and irregular mafic dikes within the adamellite. The soils overlying the bedrock are primarily residual soils formed by chemical weathering of the bedrock. Alluvial soils occur in the drainage swales at the site. No seismic Category I structures are founded on alluvial soils.

Detailed descriptions of the geologic features of the site are given in FSAR Section 2.5.1.2.2 and in the applicant's Final Geologic Report on Brecciated Zones (FSAR Reference 101). Based on observations of construction excavations in rock and performance of rock-supported structures in the Piedmont region, the applicant has stated that there is no record of adverse effects of unrelieved residual stresses in Piedmont rock and that none were noted at the Catawba site. The NRC staff has concurred in the applicant's conclusion concerning the absence of the effects of such unrelieved stresses at the site, as stated in Section 2.5.1 of this draft safety evaluation report.

2.5.4.2 Properties of Subsurface Materials

2.5.4.2.1 Field Investigations

Approximately 160 borings were drilled in the main plant area and at the SNSWP dam site. Table 2.5.4-1 in FSAR Vol. 2 summarizes the elevations of the top of continuous rock and groundwater levels found at the various borings. The preconstruction water table in the powerhouse area ranged from El.585 (in boring A-64 at Unit 1 Reactor Building) to El. 577 (in boring A-60 at Unit 2 Reactor Building). The elevations of the top of continuous rock noted at these two boring locations are 567 and 549 feet respectively. The bases of the foundation mats of the two reactor buildings are located at approximately El.510.

The field investigations included standard penetration tests (SPT) and split-barrel sampling performed generally according to ASTM D-1586, and undisturbed sampling using Shelby Tubes generally according to ASTM D-1587. In situ permeability tests were also performed in the powerhouse area and at the SNSWP dam site. In hard soils and partially weathered rock, undisturbed samples were obtained with either a coring pitcher barrel sampler or a Denison sampler. Rock coring was performed in general accordance with ASTM D-2113. Several test pits and three deep trenches were excavated to obtain bulk samples for laboratory testing.

Geophysical studies consisting of seismic refraction profiling, and up-hole and cross-hole surveys were performed to determine the seismic wave velocities of soil and rock in the powerblock and intake structure areas and along the Category I pipelines. The seismic compression wave (P-wave) velocities of residual soils measured in the area of the NSW pipelines were ~~ranged~~ about ~~between~~ 1000 ft/sec near the ground surface (about El. 620), 2700 ft/sec at about El.560, and 8400 ft/sec below El. 560. The P-wave velocities measured by refraction profiling on the exposed foundation rock ranged from 5400 ft/sec to 18100 ft/sec. These values of seismic wave velocities for the subsurface materials at various elevations are reasonable and appropriate for the design of buried structures.

2.5.4.2.2 Subsurface Profile

A thin soil stratum of fine grained red or tan sandy silts or clayey silts, is seen below the organically stained top soil in the plant area. This thin layer of silty soil, formed by advanced weathering near the surface, quickly grades into the residual soils. The upper residual soils (found in the upper 1 to 5 ft zone in the plant area) consist of fine-grained sandy silts and clayey silts that are stiff to very stiff in consistency and have SPT values of 10 to 30 blows per foot. The deeper soils are saprolites that retain the relict micro- and macro-structure of the parent rock. Texturally, these materials are coarse grained silty fine to medium sand (10 to 30 percent passing the number 200 sieve and a Unified Soil Classification of SM) and are generally of very low plasticity. Some weathered seams of mafic rocks, from 1 ft to 5 ft thick, are found in the form of numerous steeply dipping dikes. The weathered dike materials are generally fine to medium sandy silts (ML).

Residual soil (saprolites) having standard penetration resistances greater than 100 blows per foot have been designated by the applicant as partially weathered rock. The depth to the top of partially weathered rock varies from several feet to 30± feet below the preconstruction surface at the plant area. In general, the partially weathered rock has been excavated from beneath most major Category I structures. The Refueling Water Storage Tanks, Diesel Fuel Oil Tanks and the SNSW intake and discharge structures, however, are underlain by partially weathered rock.

Below the partially weathered rock is the primary parent bedrock, adamellite, which is a metamorphosed igneous rock of the Charlotte belt. The applicant has assumed the top of continuous rock to correspond to the elevation where a Rock Quality Designation (RQD) of about 75 percent on rock cores is obtained. This moderately hard to hard bedrock supports all major Category I structures.

2.5.4.2.3 Laboratory Investigations

The applicant conducted the following laboratory tests generally in accordance with accepted engineering standards to determine the engineering properties of soil and rock materials:

- grain size tests (ASTM D-421 and D-422)
- index properties tests (ASTM D-423 and D-424)
- compaction tests including optimum moisture-content and maximum dry density (ASTM D-698)
- consolidation tests (ASTM D-2435)
- permeability tests similar to ASTM D-2434
- static and dynamic triaxial tests and resonant column tests.

Static triaxial tests were conducted on samples of compacted residual soils and partially weathered rock materials from the plant area and on similar materials at SNSWP Dam to obtain the shear strength parameters. The test results are discussed in Sections 2.5.6.4 and 2.5.6.6.2 of this report.

Dynamic triaxial tests were made on several undisturbed soil samples, partially weathered rock materials, and remolded samples to ~~(1)~~ evaluate dynamic failure potential of the site soils, and to ~~(2)~~ determine the dynamic soil modulus. For evaluating the dynamic failure potential of the site materials, triaxially-confined compression tests were performed on saturated test specimens of both undisturbed and remolded soil samples, using 10 cycles of loading under controlled stress conditions. Since liquefaction was not observed during cyclic testing of the site materials, the applicant has adopted a value of 5% axial strain to represent "failure" of the test samples during cyclic triaxial testing. This failure criterion is acceptable to the staff for these materials, and for the anticipated loading conditions at this site.

The dynamic soil (Young's) modulus values were obtained in the laboratory from the results of stress-controlled, dynamic triaxial tests. The range of axial strains varied from about 0.01 percent to 1.0 percent and the cyclic load was maintained for 5 to 10 cycles. The shear modulus was calculated from the Young's modulus using an assumed value of 0.5 for Poisson's ratio of the saturated soil samples.

The dynamic shear modulus and damping values of soil at low strain levels (i.e., in the order of 0.001 to 0.01 percent) were determined by resonant column tests performed on undisturbed and remolded solid cylindrical samples. The variation of normalized shear modulus (i.e., the ratio of shear modulus at a given strain to the maximum shear modulus) with shear strain for the embankment and foundation materials was developed from the results of resonant column tests and cyclic triaxial tests. For the damping ratio variation with shear strain, only the resonant column test results were used. These relationships for shear modulus and damping ratio agree well with published relationships (Reference 1)* for these soils.

The value of shear modulus parameter, K_2 max, for dense and very dense residual foundation soils (saprolites), determined by geophysical methods in the field, exceeds the published typical value of 70 for a comparable material (very dense sand with relative density of 90 percent). The applicant has used the K_2 max values shown in Table 2.5.4-1 of this report to compute the shear modulus values by the formula given below that Table. These values are generally appropriate for the different materials identified. However, in certain situations, the use of the lower bound value of 70 for K_2 max for dense to very dense residual (saprolite) soils may not be conservative. The applicant must justify the selection of this value because ~~when~~ the field data shown in FSAR Fig 2.5.6-40 (Revision 3) range from about 55 to 155 for the saprolites.

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*Ref. 1. Seed, H. B., and Idriss, I. M., "Soil Moduli and Damping Factors for Dynamic Analyses" Earthquake Eng. Research Center Report No. EERC-70-10, December 1970.

Table 2.5.4-1 Shear Moduli* of Subsurface Materials and Embankment Soils

Material	K_{2max}	σ'_m (psf)	G_{max} (psf)	Source**
Partially weathered rock	140	1) 2500 2) 4000	1) 7.00×10^6 2) 8.85×10^6	Seismic test
Residual Soils (Saprolites)	70	1) 2500 2) 4000	1) 3.50×10^6 2) 4.43×10^6	Seismic test
Embankment Soils	40	1) 2500 2) 4000	1) 2.00×10^6 2) 2.53×10^6	Resonant column and cyclic triaxial tests.

*(1) $G_{max} = 1000\ K_{2max}\ (\sigma'_m)^{1/2}$, psf

where σ'_m = mean principal effective stress, psf

(2) G_{max} values given above correspond to assumed σ'_m values of (1) 2500 psf and (2) 4000 psf.

**Reference: FSAR Vol. 2, Fig. 2.5.6-40, Revision 3.

The applicant has evaluated the consolidation characteristics of the in-place materials and compacted soil in the plant area by means of one-dimensional consolidation tests on undisturbed samples and remolded samples. Typical consolidation test data presented in the FSAR do not show sufficient details of test conditions (e.g., water content, conditions of testing such as saturation, etc.).

The data do not show whether the samples were tested to investigate the possibility of rapid compression or 'collapse' of these soils upon saturation. Since saprolite soils are known to have caused problems due to this type of behavior, the applicant must demonstrate with supporting consolidation test data and stress-strain data from triaxial tests that such excessive settlements will not occur at Catawba site. The applicant must describe the consolidation test procedures in detail and discuss any special procedures that might have been followed in testing the saprolite soils.

2.5.4.3 Excavations and Backfill

2.5.4.3.1 Excavation and Foundation Preparation

Excavation of the residual soils and partially weathered rock in the main plant area was carried out using conventional methods from original ground surface (that varied from El. 600 to 620) to approximately El. 570. Beyond El. 570, blasting was used to remove the hard rock. When overbreak caused the excavation to extend below the proposed bases of foundation mats at approximately El. 515, fill concrete was poured to bring the bearing area up to the required elevation. Groundwater seepage and surface runoff into the foundation excavations during construction were controlled by gravity drainage through ditches leading to sumps where pumps removed the accumulated water.

2.5.4.3.2 Backfill

Three types of materials were used as backfill for the safety related structures: (1) fill concrete, (2) earth backfill, and (3) granular (coarse grained) backfill.

Fill concrete was used beneath the base of foundation mats where necessary to smooth the rock surface after blasting, as described previously. The 28-day compressive strength of this fill concrete was 3,000 psi.

The on-site residual soils obtained from general grading cuts, foundation excavations and borrow areas were used as Group I earth backfill materials; the applicant has not identified the locations where Group I earth backfill materials were used. Accepted standard procedures were used in spreading the materials in 9-inch horizontal layers and compacting the fill at moisture contents within plus or minus three percent of optimum, based on standard Proctor tests for the particular soils. Each layer was uniformly compacted to obtain densities not less than 96 percent of the Standard Proctor maximum dry density in accordance with ASTM D698. Field determination of compaction was performed in accordance with ASTM D2937 (Shelby Tube Method). The applicant must provide statistical data to verify that the specified compaction was achieved in the field.

The coarse-grained granular backfill materials consisting of crushed stone were spread in 12-inch layers and compacted to a minimum relative density of 80 percent in accordance with ASTM D2049. Field Measurement of density was made by Sand Cone method (ASTM D1556). The applicant must provide statistical data to verify that the sepcified compaction was achieved in the field.

The applicant must indicate the separate locations where the coarse-grained granular backfill and earth backfill are used.

2.5.4.4 Ground Water Conditions

The preconstruction water table in the powerhouse area ranged from El. 577 to El.585 whereas the bases of the foundation mats of the two Reactor Buildings are at approximate El. 510. Therefore a permanent groundwater drainage system has been installed to relieve hydrostatic uplift loading on the Category I structures in the power block area excepting very low pits in the Reactor and Auxiliary Buildings. The low areas not relieved of groundwater pressure are designed to withstand the resultant uplift and hydrostatic loads. The drainage system consists of foundation underdrains and continuous wall drains that maintain the groundwater levels at or near the bases of the foundation mats and basement walls. The details of the drainage system are given in FSAR Section 2.4.13.5 wherein the applicant has also described the filter design criteria.

A review of the gradation limits of the fine and coarse filter materials as well as those of the adjacent backfill material indicates that the filter design criteria (Ref. 2 and 3)* have been generally satisfied. However, the applicant has incorrectly stated the definitions of various filter design criteria. The criteria verifying calculations are done correctly as shown in FSAR Section 2.4.13.5. The applicant must correct the filter design criteria statements that have been stated incorrectly.

2.5.4.5 Response of Soil and Rock to Dynamic Loading

2.5.4.5.1 Liquefaction Potential

Most of the plant structures are founded on bedrock or partially weathered rock that is not susceptible to liquefaction. Only a few Category I structures (~~e.g.~~ the Diesel Fuel Oil Tanks, ~~and~~ portions of NSW pipelines and some conduit manholes) are founded on residual soil or compacted backfill described earlier. Based on the results of cyclic shear strength tests performed on remolded fill soils, the staff concurs in the applicant's conclusion that the Group I fill soils compacted to 96 percent standard proctor maximum dry density and the firm

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*Ref. 2. U.S. Navy, Design Manual - Soil Mechanics, Foundations and Earth Structures, No. NAVFAC DM-7, Oct. 1971, p. 7-8.14.

Ref. 3. Lambe, T. W., and Whitman, R. V., "Soil Mechanics", John Wiley & Sons, 1969, p. 293.

saprolite soils will not undergo liquefaction or excessive deformation under the safe shutdown earthquake having a peak acceleration value of 0.15 g.

2.5.4.5.2 Buried Pipelines

The applicant must docket a plan (or plans) showing the longitudinal sections of the SNSW pipelines and discharge pipelines showing therein the subsurface profile. The applicant must also show the locations of all the test borings along with the SPT blow counts and water table elevations along the longitudinal sections.

The applicant must justify the use of the average calculated value of 1580 psi/in as the coefficient of subgrade reaction in the soil-pipe interaction analysis. Apparently this value is not based on plate load tests. The applicant may compare the above calculated value with those available in the published literature.

2.5.4.6 Stability of Foundations

2.5.4.6.1 Foundations on Rock

Major Category I structures (except ~~the~~ buried Diesel Fuel Storage Tanks, buried NSW Pipelines and ~~the~~ SNSW Pond Outlet Works) are supported on mat foundations that bear on rock or fill concrete to rock. The average static bearing pressures on the mats range from 3 to 10 ksf while the maximum gross total static bearing pressures range from 10 to 20 ksf. The applicant has determined the static ultimate bearing capacity of the foundation on rock by assuming that the rock mass is comprised of rock columns, formed by vertical and near vertical and slightly open jointing surfaces, and then summing the compressive strengths of rock columns under the bearing area. The results of unconfined compressive strength tests performed on rock obtained from plant area substructure excavation were used in these calculations to arrive at the allowable bearing capacity of mat foundations. Excluding a low value of 915 psi obtained for one sample out of 27 samples reported in the FSAR (Fig. 2.5.4-12), the mean of the tested unconfined compressive strength values is about 10,000 psi. Based on this method of bearing capacity evaluation (Ref.4)* for the large mat foundations, the minimum safety factors for static loading are shown to exceed 30 for major Category I structures. The staff is satisfied with the above method of bearing capacity evaluation of mat foundations in moderately hard to hard rock at this plant.

*Ref. 4. R. E. Goodman, "Introduction to Rock Mechanics," John Wiley & Sons, New York, 1980.

The applicant has checked the stability of Category I foundations against overturning and sliding due to earthquakes, wind, and tornadoes. The minimum factors of safety against overturning and sliding for load combinations including the OBE and the design wind for the plant site are 1.5. These factors are reduced to 1.1 when the effects of the SSE and the design basis tornado are considered. The applicant has not shown the actual safety factors; the applicant has, however, stated that no tension reaction from the rock was assumed in calculating the safety factors and that the allowable toe pressure was up to 121 ksf when lateral loads due to SSE were considered. (FSAR Table 2.5.4.4, page 2). Since the calculated allowable bearing capacity of rock at the site is 965 ksf while the maximum static bearing pressure is 20 ksf and the maximum toe pressure due to overturning is 121 ksf, adequate margin of safety exists against bearing capacity failure.

For the range of the bearing pressures exerted by the structures at this site, the calculated settlement of mat foundations on rock is negligible. Using elasticity theory, the elastic deflection of less than 0.1 in. was calculated for the foundation beneath the Unit 2 Diesel Generator Building. The actual settlement measured at four corners of the roof of this building showed a settlement of about 1/4 in. after one year. The applicant has reported that this settlement has stabilized at this value. The staff concurs in the applicant's conclusion that foundations on rock will not experience significant additional settlements.

2.5.4.6.2 Foundations on Partially Weathered Rock

Some Category I structures (intake and discharge structures and above-ground storage tanks) have mat foundations bearing on partially weathered rock. The applicant has evaluated the bearing capacity of such foundations using the traditional bearing capacity equations and found the safety factors to range from 30 to 86. These safety factors are high because of the low applied static foundations loads.

The ultimate bearing capacity of the foundations on partially weathered rock ranges from 45 ksf to 64 ksf while the applied static loads range from about 0.6 to 2.1 ksf. The applicant has not given the actual dynamic bearing pressures for these foundations. However, the information given in FSAR Table 2.5.4-4 indicates that the maximum toe pressures under the SSE loading may range from 15 ksf to 22 ksf; this gives a safety factor of 3 against overturning under seismic loading conditions. The applicant must give the actual safety factor against overturning and sliding due to the SSE, wind and tornado loading conditions.

Computations of settlement of Category I mat foundations on partially weathered rock indicated negligible total settlements. Because of the rigid mat foundations and small total settlements, no differential settlement problems exist in these cases. Settlement measurements, taken at four locations on the foundation of each Refueling Water Storage Tank, have essentially stabilized at about 1/4 inch in each case. This compares well with the calculated settlement of 1/3 to 1/2 inch.

2.5.4.6.3 Foundations on Soil

Only a few Category I structures are founded directly on soil at this site. They are the buried Diesel Fuel Oil Tanks, some buried NSW pipelines and conduit manholes, and the buried SNSW Pond outlet pipes. The Diesel Fuel Oil Tanks are embedded inside Group I Earth Fill. There is a relatively thin zone of this fill between the tank bottom and the partially weathered rock on which the compacted fill rests.

The NSW pipelines are generally bedded directly on residual soils. However, in several low areas that were filled to bring them to yard grade elevation, these pipes are embedded in compacted backfill. The applicant must report whether any soft alluvial soils found in these areas were removed before placing the Group I backfill. The locations of the SNSW discharge pipelines are not shown in the plans received from the applicant. The applicant must furnish the subsurface profile along the longitudinal sections of all Category I pipelines indicating therein the locations of the borings. Conduit manholes have mat foundations bearing on residual soil, partially weathered rock and compacted backfill.

The ultimate bearing capacity of the soil supporting the above structures range from about 13 ksf to 77 ksf, giving safety factors ranging from 6 to 52 for static loading conditions.

Generally, construction of the yard fills was completed before commencing the excavation for the pipelines, tanks and manholes. Since the loading due to the installation of pipelines, tanks, etc. is less than the pre-excavation overburden pressure, the applicant has concluded that the static settlement will be small for these structures. In Section 2.5.4.2.3 of this report, the staff has requested the applicant to furnish additional consolidation test data to support this conclusion.

2.5.4.6.4 Subsurface Lateral Loading

The static lateral soil pressures acting against the rigid substructure walls of Category I facilities were calculated by the applicant using an at-rest earth pressure coefficient of 0.5 for compacted (silty sand) backfill. This value is appropriately based on published data on this subject. Provision has been made in structural design for hydrostatic pressures against the Reactor Buildings and Auxiliary Buildings; however, these pressures were not combined with seismic loading because of the installation of a permanent groundwater drainage system described in FSAR Section 2.4.13.5. Continuous monitoring devices have been installed in six of the twelve permanent groundwater wells in the zoned wall filter around the perimeter of the Reactor and Auxiliary Buildings. The latest results of monitoring of this permanent groundwater drainage system indicated that this system is functioning as expected.

The applicant has used empirical methods for calculating dynamic (seismic) lateral earth pressures. For design of walls with moderate height, the applicant has stated that the increase in lateral earth pressure may be assumed to be 10% of the normal design pressure, as recommended by W. C. Teng in his book, "Foundation Design." In the case of high outside walls, the applicant has stated that combined pressure may be determined approximately by the trial wedge method. The applicant has, however, not furnished the dynamic lateral soil pressures. The applicant must furnish these lateral dynamic soil pressures and demonstrate that these are conservative when compared to the values that may be obtained by following other state-of-the-art methods available in the published literature.

2.5.4.7 Conclusions

Subject to ths submission of additional data discussed in the preceding paragraphs, the results of the applicant's investigations, laboratory and field tests, and analyses indicate that the plant foundations will safely support the seismic Category I structures, equipment and components. The applicant must docket the following information/data to support some of the conclusions/statements made in the FSAR.

1. Docket consolidation-time curves for saprolite soils and also furnish details of testing to show that rapid compression or 'collapse' of these soils will not occur upon saturation;

2. Correct the statements defining the filter design criteria for the permanent groundwater drainage system for the main plant area;

3. Furnish dynamic lateral soil pressures acting on Category I structures.

4. Identify the locations where Group I earth fill and coarse grained granular backfill were used.

2.5.5 Stability of Slopes

The applicant's stability analysis of the slopes of the SNSWP Dam is evaluated in the following section. No other nuclear safety related slopes exist at the site.

2.5.6 Embankments and Dams

2.5.6.1 Standby Nuclear Service Water Pond Dam (SNSWP Dam)

General Design Criteria 44 and 45 of Appendix A to 10 CFR Part 50 describe the requirements for assuring a redundant source of cooling water supply for nuclear power plants. In the event of postulated loss of Lake Wylie* water impounded by the Standby Nuclear Service Water Pond (SNSWP) Dam within a cove of the lake provides the emergency cooling water for the plant. The SNSWP Dam located 2800 ft north of the plant, is an impervious,

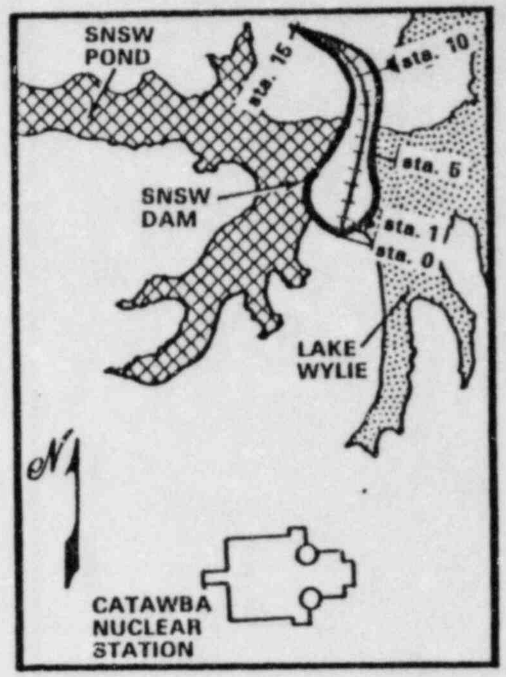
*Lake Wylie Dam is regulated by Federal Energy Regulatory Commission; it is not a seismic Category I structure and therefore was not evaluated by the NRC Staff.

homogeneous, rolled, earthfill structure. The dam has a crest width of 35 ft and extends 1710 ft between abutments at crest elevation 595. The maximum height of the dam is 75 ft with upstream and downstream faces sloped at 3 to 1 (horizontal to vertical). All surfaces including the crest are protected against wave action with stone riprap. A typical cross section of the SNSWP Dam is provided in FSAR Figure 2.5.6-1 and is included herein (SER Figure 2.5.1). The centerline of the dam is oriented North 22° East from the south abutment for about 656 ft, then follows a circular curve for approximately 272 ft, and then runs North 39° West approximately 782 feet to the north abutment. The SNSW Pond is operated between full pond El. 571 and maximum drawdown El. 567, while the full pond elevation and maximum drawdown elevation of Lake Wylie are El. 569.4 and El. 550 respectively. (The applicant has stated in FSAR Section 2.5.6.6.2 that the Lake Wylie pool level may drop to El. 550 while the maximum drawdown elevation is shown as El. 559.4 in FSAR Figure 2.5.6-1. The applicant may clarify this discrepancy).

2.5.6.2 Subsurface Conditions

The site exploration for the SNSWP Dam involved drilling about 22 borings in the dam area. The results of the test borings indicated that the subsurface materials at the dam site included alluvial soils, residual soils, partially weathered rock, and rock.

The alluvium consisting of very soft to stiff sandy silts (ML) and very loose to dense silty fine sands (SM) ranged in thickness from 0 to 14 ft and was completely removed from under the dam base.



LEGEND:

SM 1 Thru SM 12 – Settlement Markers
P 1 Thru P 8 – Piezometers

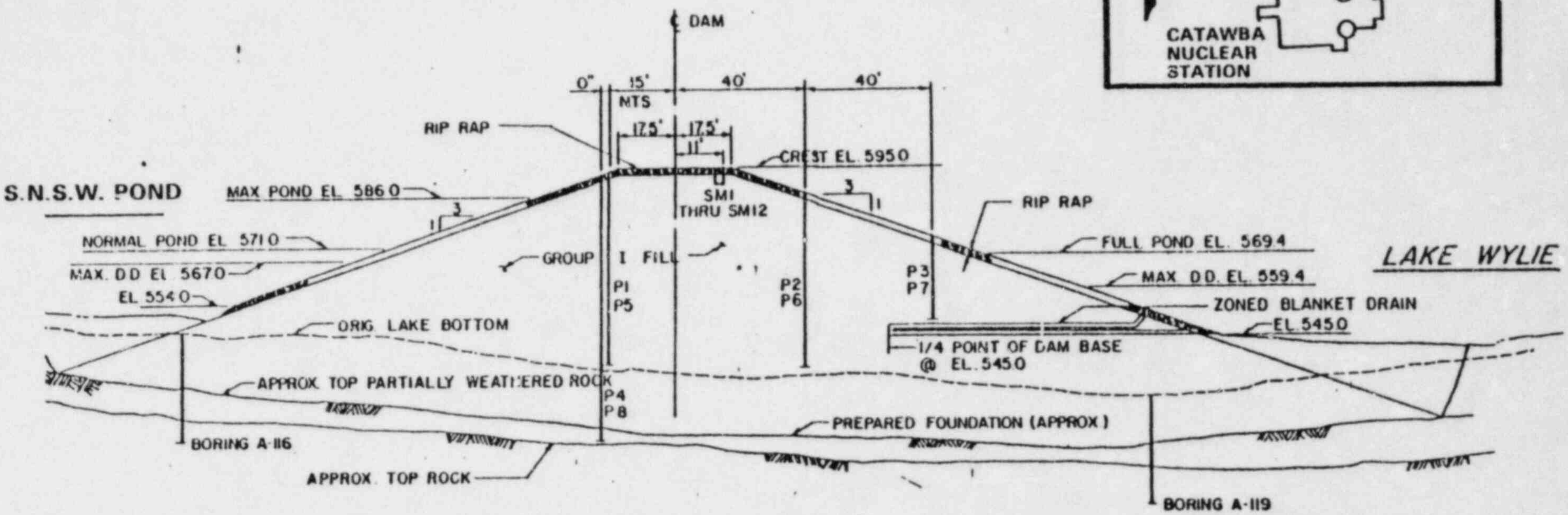


Figure 2.5-1. Typical Section of SNSWP Dam at Catawba Nuclear Station (Ref. FSAR Vol. 2)

The residual soils (saprolites) were derived by weathering in-place from the underlying adamellite bedrock. In-situ seismic wave velocity measurements were made at two locations - one at the Lake bed and another on the shore. There was a reduction in compression and shear wave velocities at a depth between 20 and 30 ft and a rapid increase with depth beyond 30 ft at the location in the lake. The residual soils (represented by such reduced wave velocities) and a near-surface stratum of fine grained sandy silts (ML) in the north abutment area were removed from beneath the dam during foundation preparation.

A zone of silty sand saprolite (SM) exists below the fine grained sandy silts (ML) at the north abutment and below the alluvium at the lake bottom and on the south abutment as shown in FSAR Fig. 2.5.6-6. This figure indicates that the thickness of the saprolite soils, that have not been removed as discussed in the previous paragraph, ranges from 10 to 30 ft.

Below the zone of residual soils (described above) exists the zone of partially weathered rock, and the unweathered bedrock (adamellite). The partially weathered rock materials exhibit standard penetration resistance (N) values in excess of 100 blows per foot. The thickness of the partially weathered rock ranges from 5 to 20 ft as shown in FSAR Fig. 2.5.6-6.

2.5.6.3 Foundation Preparation

The SNSWP Dam was constructed in a cove of Lake Wylie. The dam construction area was dewatered by building cofferdams in Lake Wylie and controlling groundwater by directing it to ditches and sumps with pumps where necessary so as not to degrade the foundation material. The hydraulic head on the cofferdams with respect to the stripped foundation varied from 30 to 50 ft. Groundwater flow from springs in the foundation area was controlled by means of granular drains in shallow trenches and vertical drainage pipes for pumping. The vertical pipes were constructed of open-ended barrels placed end to end as the embankment fill progressed. Low water levels were maintained in the drains by periodic pumping during foundation preparation and initial fill placement. The foundation drains were grouted when no longer required and the vertical barrels were filled with concrete.

After clearing the topsoil, all the alluvial soils and all other soils having shear strengths less than the design shear strength, were removed. The shear strength of materials left in place were verified by using dynamic penetrometers calibrated for the site and standard penetration tests (ASTM D1586). Foundation materials having a Standard Penetration Test (SPT) resistance (or equivalent dynamic penetrometer resistance) of less than 15 blows per foot were removed. The applicant has not demonstrated the adequacy of this field control criterion for removing the unsatisfactory materials from the foundation bed for the dam. Using the results of the triaxial shear test data given in FSAR (Fig. 2.5.6-12), the staff made a preliminary check and found that the consolidated, undrained shear strengths of two samples with SPT values of 15 and 16 blows per foot were reasonably close to the design shear strength under an assumed normal stress of 2.5 ksf. The applicant must demonstrate the adequacy of its field control criterion by providing calculations using the test data for several samples under different normal stresses corresponding to the actual locations of the samples, or by furnishing correlation of dynamic penetrometer resistance with SPT resistance.

Before placing the main embankment material, irregularities in the foundation surface were cleaned and filled with dental concrete; on the south abutment, dental concrete varying from 3 to 12 inches in depth was placed to reduce seepage. Slush grouting was also done where necessary to fill any minor surficial irregularities and provide a bond between the foundation and embankment materials.

2.5.6.4 Embankment Geometry and Materials

At its maximum cross section, the SNSWP Dam is approximately 75 feet high from the prepared foundation surface that was about 25 feet below the bottom of Lake Wylie. At the same cross section the base of the embankment is approximately 470 ft wide. Each face of the dam is sloped at 3 to 1 (horizontal to vertical) from the 35-foot wide crest down to the toe. Material for the dam (that contains about 536,000 cubic yards of compacted earth fill) was brought from two major borrow areas: one located north of SNSW pond provided 351,000 cu yds and the other near the cooling tower yard area provided 172,000 cu yds of earthfill.

The main volume of the embankment material used from all the borrow areas consists of silty sand saprolites (SM) with standard Proctor maximum dry densities (ASTM D698) in excess of 105 pcf. Small quantities of the dike soils (having a Unified Soil Classification of ML) were found mixed with the saprolites during excavation from the borrow areas.

Based on the results of static triaxial tests on borrow soils compacted to 96 percent of the standard Proctor maximum dry density (ASTM D-698) the design shear strength parameters listed in Table 2.5.6-1 of this report have been assigned for the embankment fill. This Table also shows the static shear strength parameters assigned to the residual foundation soils (saprolites) and to partially weathered rock. The design shear strength parameters assigned to the embankment materials based on laboratory test data seem to be appropriate. However, the presence of relict joints in the foundation materials (residual soils and partially weathered rock) may greatly reduce the significance of the c and Ø values (Ref. 5) assigned to them. The applicant must examine the construction (foundation

*Ref. 5. Peck, R. B. et al, "Foundation Engineering", 2nd Ed. 1974, John Wiley & Sons, pp 153-154.

Table 2.5.~~1-2~~ 6-1 Design Static Shear Strength Parameters for SNSWP Dam Materials

Material	Saturated, Consolidated Undrained, R-test c, psf	Ø, deg.	Saturated, Consolidated, Undrained, $\bar{R}$-test c, psf	Ø, deg.	Unsaturated, Unconsolidated Undrained, Q-test(1) c, psf	Ø, deg.
Embankment Fill	900	29	400	34	900	24
Coarse grained Residual Soils (Saprolites)	750	25	400	30	1000	19
Partially Weathered Rock	1000	34	1000	44	1000	34

(1)Notes: 1) The Q-test data were obtained during PSAR investigations

2) R-test: results not corrected for pore pressures (Total stress)

3) $\bar{R}$-test: results corrected for pore pressures (Effective stress)

preparation) data and report on the presence or absence of such relict joints in the foundation. The applicant must also indicate the number of tests performed for defining each failure envelope in Mohr diagrams shown in the FSAR.

The crest and the upstream and downstream faces of the SNSWP dam are protected by riprap stones extending from abutment to abutment and underlain by a 12-inch thick layer of filter material. A zoned blanket drain is provided in the downstream side of the dam below the original ground elevation 570. A toe drain is provided on the downstream side above El 570

2.5.6.5 Embankment Fill Placement and Settlement

2.5.6.5.1 Embankment Fill

The near-surface soils in the borrow areas consisted of up to 1 ft of organic top soil and 0 to 3 ft thick silt or sandy silt (ML with occasional MH materials). The applicant has not used these materials for embankment fill. The major portion of the embankment material obtained from the deeper layers of all the borrow areas consisted of saprolite soils having a Unified Soil Classification of SM with maximum dr densities exceeding 105 psf. The deeper soils, however, include a minor soil grouping of material having an ML classification and maximum dry densities ranging from 95 to 105 pcf. The applicant has stated that these materials, formed by the weathering of the dikes, were an insignificant portion of the embankment fill. A comparison of the grain size distribution band of the silty sand saprolites sampled during the PSAR studies with the band of the fill samples taken from the embankment shows that the actual fill materials are similar, (although slightly finer) in terms of grain size distribution to those soils tested for the PSAR investigation. Similarly the optimum moisture and maximum dry density data for the PSAR soils appear similar to the corresponding data for the soils obtained from the test pits in the borrow areas used in actual embankment construction. Therefore, the staff concurs in the applicant's conclusion that the performance of the embankment will not be affected by the inclusion of the ML soils in the essentially SM fill materials.

The fill was placed in nine-inch thick layers and compacted at moisture contents within +3 to -1 percent of optimum, based on standard Proctor density tests (ASTM D698). Moisture checks were made in accordance with ASTM D2216 at a frequency of about 4 tests per day. Each layer was compacted to attain not less than 96 percent of the standard Proctor density (ASTM D698). Field compaction tests were performed by Shelby Tube Method (ASTM D2937) for each change in soil type of borrow source and at a frequency of one per every 2500 cubic yards of fill. The applicant must provide statistical data to verify that the specified compaction was achieved in the field. In addition to the field tests, laboratory triaxial shear tests were done to assure that the shear strength of compacted fill meets the design requirements.

2.5.6.5.2 Embankment Settlement

The applicant has calculated the consolidation settlement of the residual foundation soil in the area of the thickest soil foundation materials under the full embankment weight for the reservoir empty condition. The applicant states that much of the calculated total settlement of 5-1/2 in. at that location would occur during embankment construction. The applicant must docket the consolidation-time curves and furnish the settlement calculation along with the soil profile to justify the above statement.

Settlement calculations, made by the applicant for the maximum height of embankment material under its own weight and reservoir empty condition, have shown a total static embankment settlement of 16 inches (that is about 2 percent of the maximum height). It is not clear if this (16 in) settlement includes the 5-1/2 in. settlement of the foundation soil discussed above. To compensate for this settlement the dam crest was overbuilt two feet above the finished design elevation at the maximum dam section and the overbuild was proportional to the fill height at other sections. Post-construction settlement monitoring was started on March 3, 1978 a few months after end of construction in late 1977. The settlement recorded since March 1978 is only about one inch. Since no measurements were made from the end of construction to the beginning of settlement monitoring, the applicant must docket the as-built crest elevations along with the present crest elevations. The applicant must also compare the predicted and measured differential settlement of the dam.

2.5.6.6 Slope Stability

2.5.6.6.1 Static Stability Analysis

The SNSWP Dam cross section selected for stability analysis includes a foundation zone of firm to dense and very dense coarse grained saprolite (SM) soil between the embankment and partially weathered rock. This zone of material exists between Stations 1 + 75 and 6 + 50. Selection of such a cross section with a saprolite layer is more conservative than a cross section where the saprolite layer has been replaced by compacted fill because of the slightly lower static shear strength parameters assigned to the existing saprolite material than for the compacted fill.

Four cases of static loading were analyzed: (1) end of construction, upstream and downstream; (2) steady state seepage with the worst combination of water levels; (3) instantaneous drawdown of Lake Wylie from maximum flood level to El.559.4 and (4) instantaneous drawdown of SNSWP Dam.

Shear strength parameters shown in Table 2.5.6-1 of this report were used for the different cases of loading as shown below:

Loading Condition	Shear Strength Conditions
1. End of Construction (prior to pond filling)	Unsaturated, unconsolidated, undrained (total stress analysis using Q-test results).
2. Steady seepage (maximum pond level)	Saturated, consolidated, undrained, corrected for pore pressure -

	(Effective stress analysis, using $\overline{R}$ test results).
3. Sudden drawdown	
(a) downstream - loss of Lake Wylie	Saturated, consolidated, undrained, not corrected for pore-pressure -
(b) upstream - Loss of SNSW Pond	(Total stress analysis based on R-test results).

Stability analyses were performed by the applicant using the circular arc and the method of slices. Minimum safety factors ranging from 1.95 to 2.30 were obtained for the above loading conditions; these compare favorably with the applicants' specified minimum safety factors that ranged from 1.25 to 1.50. However, the applicant does not appear to have used the proper drawdown elevation for the "loss of Lake Wylie" condition. The maximum drawdown elevation for Lake Wylie should be El. 550 (FSAR Section 2.5.6.6.2) instead of El. 559.4 (as shown in FSAR Fig. 2.5.6-1). The applicant must also explain why a wedge failure mode was not considered since the existing saprolite foundation soil between the embankment and the partially weathered rock is somewhat weaker than the embankment material.

2.5.6.6.2 Dynamic Stability Analysis

The dynamic properties of the foundation materials in situ were evaluated by field measurements of shear wave and compression wave velocities. The dynamic properties of the embankment, and foundation, materials were determined by appropriate laboratory tests (i.e. ~~, resonant column and cyclic triaxial tests for dynamic shear moduli, and~~ resonant column and cyclic triaxial tests for dynamic shear moduli, and resonant column tests for damping values). The maximum dynamic shear modulus values of embankment materials, determined in the laboratory, range from about 22 ksf (at a confining stress of 4 ksf) to about 15 ksf (at a confining stress of 2 ksf). The damping ratio values proposed for the dynamic analysis range from about 2 percent (for shear strain of 10^{-3} percent) to about 24 percent (for shear strain of 1 percent). After determining the "most probable" values of shear modulus and damping ratio, the applicant varied them by $\pm$ 25% to evaluate the effects of such variation on the calculated shear stresses. A combination of higher-bound shear modulus and lower-bound damping ratio produced the highest shear stresses. Therefore, the final dynamic analysis was performed using this combination of dynamic material properties. The applicant must investigate and report whether the procedure of using upper-bound modulus and lower-bound damping ratio for all three materials simultaneously will give conservative results or whether some other combinations for different materials will be necessary.

The dynamic shear strength characteristics of the embankment materials were determined by conducting cyclic laboratory tests on remolded, (and some undisturbed) saturated, isotropically and anisotropically consolidated samples. Since no liquefaction of the materials was observed as stated in FSAR Section 2.5.4.8.1 and in Section 2.5.4.5.1 of this report, the failure criterion used in this analysis is the stress required to cause 5 percent axial strain of the sample in 10 cycles of loading in the laboratory tests. This failure stress is designated as the dynamic shear strength of the material.

The dynamic response of the SNSWP Dam under seismic loading (during a safe shutdown earthquake (SSE) acceleration of 0.15g) was evaluated using the computer program QUAD-4 based on finite element analysis technique. The various steps involved in this dynamic analysis are described in FSAR Section 2.5.6.5.4. Briefly, the initial embankment stresses under static conditions are determined using nonlinear stress-strain properties of the soils. Then, using the synthetic time histories of bedrock accelerations produced by the SSE and the strain-dependent shear modulus and damping values of the embankment materials as input to QUAD-4, the induced dynamic shear stresses in elements throughout the embankment are obtained as output. The sum* of the static and dynamic shear stresses are compared with the available shear strengths of the embankment materials to evaluate the safety margin of the dam elements under seismic loading.

The safety factor against shear failure of the materials of the dam during the SSE is calculated as the ratio between the dynamic shear strength of the material and the seismically induced shear stress. In the finite element analysis procedure followed in this case, the safety factors were calculated for each of the finite elements of the dam cross section and the lowest

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* The applicant must confirm this since it has not specifically stated ~~that the~~ sum was used to calculate the dynamic safety factor.

safety factor obtained was 1.06 as shown in FSAR Fig. 2.5.6-52. However, this figure does not give the safety factors for all elements in the Dam cross section. The applicant must provide the missing safety factors in this figure. Also, the applicant must furnish the synthetic time histories of bedrock accelerations (both horizontal and vertical motions) and describe the procedure used in combining the shear stresses produced by the three components of earthquake motions.

The applicant has stated that lower safety factors would be obtained when normal pond level is combined with an SSE event since this condition produces lower normal stresses within the embankment. However, it is necessary for the applicant to justify the above assumption by performing dynamic response analyses for the following two cases and examining the worst case: (1) SSE plus pond water level corresponding to 25 year flood, and (2) OBE plus pond level corresponding to standard project flood.

The applicant had stated in Appendix 2G to the Catawba PSAR that the seismically induced permanent displacement of the Dam calculated by Newmark's method would be less than 1.0 inch. Considering the relatively high minimum safety factor (about 2.0) obtained for static stability of this dam and the low acceleration level of SSE, the low value of permanent displacement predicted by Newmark's method (Ref. 6) is reasonable. The applicant

*Ref. 6. N. M. Newmark, "Effects of Earthquakes on Dams and Embankments," Geotechnique, Vol. 5, No. 2, June 1965.

should, however, include a brief discussion of this aspect of the behavior of the dam in FSAR Section 2.5.6.5.4, "Dynamic Stability Evaluation", since this has been omitted from the FSAR.

2.5.6.7 Embankment Drainage

A zoned blanket drain is provided in the downstream side of the SNSWP dam at El 545 and it extends to the one-quarter point of the dam base. (See Fig. 2.5-1 of this report). The primary purpose of the blanket drain is to control any rapid drawdown pore pressures on the downstream slope of the dam that faces Lake Wylie in the event of rapid lowering of the level of Lake Wylie. This blanket drain consists of a 6-inch layer of free draining material (coarse filter) sandwiched between 6-inch thick fine filter layers. The gradation limits of the fine and coarse filter materials generally satisfy the filter design criteria, as shown in FSAR Figs. 2.5.6-36 through Figs. 2.5.6-38.

A toe drain is provided on the downstream side of the SNSWP dam above El 570. The function of the toe drain is similar to that of the zoned blanket drain described above. The applicant ~~must~~ demonstrate that the gradation limits of the toe drain materials satisfy the filter design criteria.

2.5.6.8 Performance Monitoring

Performance monitoring is necessary to ensure that the SNSWP Dam will remain functional and permit safe shutdown of the plant in the event of loss of Lake Wylie that supplies the normal cooling water.

2.5.6.8.1 Seepage Test

A seepage test was carried out after initial filling of the SNSWP dam in late 1978 to demonstrate that there will not be excessive seepage losses from the SNSW pond. Using a temporary drainage ditch with V-notched weir along the downstream toe of the dam, seepage was monitored for a 60-day period. Of the total measured flow of 68 to 76 gpm that comprises of groundwater and seepage flow, the seepage rate was estimated by the applicant to range from 20 to 28 gpm. The loss of water in 30 days represented by this seepage rate is less than one percent of the storage volume of the SNSW pond corresponding to normal pond level.

Permeability of in situ sandy silts (that form only a minor portion of the dam foundation in the northern end) ranges from 3 to 50 ft per year. The residual coarse grained (silty sand) saprolites that form the major soil type in the dam and the partially weathered rock have a representative permeability value of 700 ft per year (7×10^{-4} cm/sec). The bedrock permeability (including the effects of rock jointing) ranges from 0 to 470 ft per year. The SNSWP dam will be subjected to relatively small differential hydrostatic heads across the dam under normal operating conditions and to only 21 ft differential hydrostatic head during a lake level drop due to Lake Wylie Dam failure. No theoretical seepage analysis was performed because of the relatively small differential hydrostatic head across the dam.

The applicant must describe the procedures used to plug the holes made by the test borings in the SNSWP dam area.

2.5.6.8.2 Instrumentation

Two sets of three piezometers each were installed at Stations 1 + 75 and 3 + 75 to monitor the phreatic surface within the embankment. One of these three piezometers at each station is installed at 15' upstream of centerline, while the other two are installed at 40' and 80' downstream of centerline respectively. Two piezometers have also been intsalled in the foundation material to monitor the foundation pore pressures at Stations 0 + 70 and 2 + 40; both piezometers are located at 15' upstream of dam centerline in the partially weathered rock, just above the top of bedrock. (The stations are numbered starting near the south abutment of the SNSWP dam). Twelve settlement markers were placed on the crest of the dam to monitor the post construction settlement.

2.5.6.8.3 Inspection

The applicant has instituted a periodic dam inspection program for the SNSWP Dam and discharge facilities. These inspections are reported to conform to the requirements of Regulatory Guide 1.127, with minor exceptions, listed in FSAR Section 1.8. The piezometric data and the settlement data furnished by the applicant indicate that the dam is functioning as expected. The measured phreatic elevations in all of the piezometers agree reasonably well with the estimated elevations - the measured data being a little less than the values estimated based on static conditions with SNSW pond at normal pond elevation (El. 571) and Lake Wylie at full pond elevation (El. 569.4). The results of settlement monitoring have been discussed in Section 2.5.6.5.2 of this report.

During the fourth inspection of the dam carried out in late 1981 localized erosion areas are reported to have been noticed in three small areas (with a total area of about 0.3 acre) and subsequently repaired.

2.5.6.9 Conclusions

The staff finds that the material properties and procedures used in the static and dynamic stability analyses of the SNSWP Dam, and the margins of safety for the different conditions of loading are acceptable. However, as stated in the above paragraphs the applicant must docket supporting data/calculations for confirming some of the conclusions. The required additional data include the following:

1. Consolidation - time curves for the embankment and foundation materials and typical settlement calculations for the foundation soil and the embankment fill;
2. Demonstrate the adequacy of field control exercised by the applicant for removing the unsatisfactory materials from the foundation bed for SNSWP dam;

3. The applicant must justify the following assumptions made in the dynamic analysis of SNSWP dam: (1) lower safety factors would be obtained when normal pond level is combined with an SSE event; (2) a combination of higher-bound shear modulus and lower-bound damping ratio for all three materials simultaneously would produce the highest shear stresses.
4. Docket the 'as-built' crest elevations of the dam along with the present elevations along the longitudinal axis of the dam, and compare the estimated and predicted differential settlement of the dam.
5. Describe the procedure used to combine the seismically induced stresses in the dam due to the three components of the SSE.
6. Commitment to follow Regulatory Guide 1.127 throughout the lifetime of the plant.

CATAWBA NUCLEAR STATION, UNITS 1 AND 2
DRAFT SAFETY EVALUATION REPORT
CHAPTER 2.5. GEOLOGY AND SEISMOLOGY

The NRC staff and its advisor, the U.S. Geological Survey (USGS) concluded after its review in 1975 of the Preliminary Safety Analysis Report (PSAR), that the earthquake design bases of 0.15 g for the SSE and 0.08 g for the OBE were adequate, and that there was no potential for surface faulting at the site.

The NRC reaffirmed that position during construction of the facility after its review of additional information regarding numerous faults that were discovered in the plant excavations. The faults were investigated in considerable detail and it was demonstrated by the licensee that they were no younger than 86 ± 30 million years old. This conclusion was supported by an independent panel of geologists assembled by the licensee. The staff's analysis is presented in "Safety Evaluation of the Brecciated Zones at the site of the Catawba Nuclear Station, Units 1 and 2," July 6, 1976.

The NRC staff has completed its review of the geological and seismological aspects of the Final Safety Analysis Report (FSAR). We find that our previous conclusions remain valid, i.e., the seismic design bases are adequate and there is no surface displacement hazard at the site.

Recently the USGS has stated that it is reassessing its position regarding the localization of the seismicity in the vicinity of Charleston, S.C., including the 1886 Modified Mercalli Intensity X Earthquake. A formal statement of that position is forthcoming. The staff has supported the existing USGS position on the Charleston Earthquake with respect to the seismic and structural uniqueness of the Charleston area. We continue to support that position and will examine any reassessment by the USGS.

In licensing decisions since about 1976 regarding the seismic design basis of nuclear power plants located in the Precambrian-Paleozoic crystalline section of the Appalachian Orogen, particularly in New England and the northernmost Piedmont, the staff has recognized the New England-Piedmont Tectonic Province. Because seismicity was relatively uniform throughout this province, and the maximum historic earthquakes were MMI VII, it was not important to subdivide it. However in the Southern Appalachian area the staff, in effect, has treated the southern Piedmont as a separate tectonic area. Although this is the case, on January 9, 1982, a magnitude (m_b) 5.7, MMI VI earthquake occurred in south-central New Brunswick, Canada in geologic terrain that is similar to that which characterizes the New England-Piedmont Tectonic Province (including the southern Piedmont). Extensive research is under way regarding that earthquake by the Canadians, the U.S. Geological Survey, universities, consulting firms, and the New England utility companies. The NRC Geosciences Branch has formed a panel to monitor the results of these studies and assess them with respect to nuclear power plant sites in the region. If it becomes necessary to consider this earthquake to be the largest historic earthquake for licensing purposes, then this concern ~~must~~ may need to be addressed as part of the much broader seismo-tectonic issue in which the validity of the Piedmont-New England tectonic province as a homogeneous unit must also be considered.

Based upon the available information, it is our position that the controlling earthquake for Catawba should be assumed to be equivalent to the 1913 Union County MMI VII earthquake which is the largest historic event in the Southern Piedmont. We base our seismic conclusion regarding the Catawba site on our experience in reviewing other sites in the region and on past review positions taken for sites in the Southern Piedmont (i.e., McGuire, Summer, Catawba, Perkins, Cherokee, etc.).

We conclude that the applicant has satisfied the requirements of 10 CFR 100, Appendix A. We also find that the FSAR conforms to the applicable sections of the following documents:

(1) Standard Review Plan - NUREG 0800, Sections 2.5.1, 2.5.2, and 2.5.3.

(2) Regulatory Guide 1.70, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants," Revision 2.

(3) Regulatory Guide 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants," Revision 1.

Based on our review of the FSAR and pertinent documents from the published scientific literature we conclude:

1. The applicant has conducted an adequate investigation of the site and region around the site, and there are no geologic conditions that pose a hazard to the site;

2. The maximum earthquake that should be considered at the site is defined by modified Mercalli Intensity VII, or magnitude $m_{blg} = 5.3$. The applicant's overall design criteria are acceptable, provided the effects of soil amplification on Category 1 structures not founded on continuous rock are further analyzed and documented. The staff will review that analysis when it becomes available;

3. The Operating Basis Earthquake of 0.08 g ZPA anchored to a Newmark, 1967, response spectrum is adequate, and

4. There are no capable faults at the site or in the site region.

2.5.1 <u>Basic Geologic and Seismic Information</u>

The paragraphs in this section contain a brief summary of the geological conditions of the Catawba Nuclear Site and the basis for our conclusion concerning the geological suitability of the site.

2.5.1.1 <u>Regional Geology</u>

The Catawba site lies in the Piedmont Physiographic Province (Fenneman, 1938 and Thornbury, 1965). Elevations in this portion of the Piedmont range from

400 feet mean sea level (ft MSL) at the eastern boundary to +1200 ft MSL near its western boundary. The Piedmont is bounded by the Coastal Plain about 50 miles southeast of the site and the Blue Ridge Physiographic Province about 70 miles northwest of the site.

The Piedmont is underlain by crystalline metamorphic and igneous rocks that were formed in the Late Precambrian and Early Paleozoic (800 million years before present (mybp) to 400 mybp). These rocks have been subjected to several periods of deformation during the Paleozoic Era (570 mybp to 240 mybp).

During the Mesozoic Era (240 mybp to 63 mybp), continental rifting caused the formation of large, sediment filled, fault-bounded basins in the Piedmont and Coastal Plain. This rifting was accompanied by the intrusion and extrusion of mafic rock which is present in the region in the form of diabase dikes, sills and flows.

The rocks of the Piedmont slope to the southeast and disappear beneath the southeasterly thickening wedge of unconsolidated to poorly consolidated sediments underlying the Coastal Plain physiographic province. The outcrop of the contact between the Coastal Plain deposits and the Piedmont rocks is called the Fall Zone. The Fall Zone is located about 50 miles southeast of the site. The Coastal Plain is comprised of Cretaceous to Recent (138 mybp to present) sands, gravels, silts, clays, shells, and limestones that thicken from the Fall Zone to up to 10,000 feet along portions of the Carolina coast. The Coastal Plain is 90 to 120 miles wide and ranges in elevation from +500 feet MSL in the west to sea level in the east.

The Blue Ridge physiographic province is underlain by highly deformed Precambrian (more than 570 mybp) igneous and metamorphic rocks. Elevations range from about 1500 ft MSL to more than 6,500 atop Mt. Mitchell in North Carolina. The southeast boundary of the Blue Ridge is the edge of the Piedmont and the northwest boundary is defined by thrust faults along which the Blue Ridge and Piedmont rocks have been thrust westward over sedimentry rocks of the Valley and Ridge Province.

During past licensing decisions the NRC and AEC have held to the position that the relatively high seismic activity in the vicinity of Charleston, S.C., including the 1886 MM Intensity X earthquake, is related to unique tectonic structure there and, therefore, for licensing purposes in the context of the tectonic province approach, should not be assumed to occur anywhere else. This conclusion is based primarily on the persistent seismicity that has characterized the meizoseismal zone of the Charleston Earthquake since 1886. It is also based on evidence, though not strong, of unique geologic structure. Lacking definitive information, the NRC-AEC based its conclusion to a very great extent on advice from the U.S. Geological Survey.

In 1973, with AEC funding, the USGS began extensive geologic and seismic investigations in the Charleston region. These studies are still underway. As a result of these investigations, a great deal of information has been obtained, but the source mechanism of the seismicity still is not known. Many working hypotheses have been developed based on the research data. These hypotheses are described in the Virgil C. Summer Safety Evaluation Report (NRC 1981), and will not be discussed here, only to say that some of these theories postulate that the Charleston Earthquake of 1886 could recur in other areas of the Piedmont and Atlantic Coastal Plain in addition to the epicentral area.

Because of the wide range of opinions within the scientific community concerning the tectonic mechanism for the Charleston seismicity, the USGS announced in January, 1982, that it will reassess its past position. We expect the result of that reanalysis to be available in the Fall of 1982.

A change in the USGS position could require a re-evaluation of the seismic design bases of the Catawba site assuming that the Charleston Earthquake could occur closer to the site than was previously considered. However, pending the announcement of the USGS position, the NRC staff continues to support its past position that the Charleston seismicity is associated with tectonic structure in the Charleston-Summerville area, and that for licensing purposes in the context of the tectonic province approach, should not be assumed to occur anywhere else. When the USGS statement is made we will consider it from a scientific and regulatory point of view.

Major structures in the region around the site include the Gold Hill-Silver Hill fault system, a southwest projection of which is about 11 miles southeast of the site; the eastern Piedmont fault system, 58 miles southeast of the site; the Kings Mountain Belt, 19 miles northwest of the site; and the Brevard zone, 70 miles northwest of the site.

The Gold Hill-Silver Hill fault trends northeast to north-northeast and dips to the northwest. It is 50 to 90 miles long and 1 to 4 miles wide. It is made up of brecciated zones that have been cut by diabase dikes of Triassic age (240 mybp to 105 mybp). The unfaulted diabase dikes demonstrate that this fault zone is not capable according to Appendix A, 10 CFR 100.

The Eastern Piedmont fault system includes the Goat Rock fault, the Towaliga fault, and the Modoc fault. The system strikes northeast to north-northeast and dips to the southeast. It extends from Alabama to Virginia and ranges from 15 to 40 miles wide. This fault system is interpreted to have originated as mylonitization, possibly related to folding, and later undergoing brittle deformation. An upper limit of the last movement is indicated by the presence of Mesozoic diabase dikes that cut across the southern part of the system in Georgia (Pickering and Murray, 1976).

Additionally, the Siloam granite of Permian age (more than 240 mybp) crosses the trend of the Goat Rock fault without being offset. Finally, the aeromagnetic map of the Carolinas, presented in the FSAR shows probable diabase dikes transecting the system. This information leads us to conclude that the Eastern Piedmont fault system is not capable.

The Brevard zone is a major structural feature that varies in width from one to four miles, strikes northeast for a distance of at least 600 miles, and dips to the southeast. Interpretations of recent CO-CORP data (Cook et al., 1979) indicate that the Brevard zone is one of the many thrust faults along which slivers of the Piedmont and Blue Ridge have been thrust northwestward over rocks of the Valley and Ridge Province. These regional faults are believed to be listric to a master decollement which is present at depths from 4 to 13 kilometers beneath the Appalachian Mountains, and, along which the

Valley and Ridge, Blue Ridge, and Piedmont, have been overthrust as much as 200 kilometers to the northwest from their original position.

The Brevard zone consists of many diverse types of rocks and there are many theories about its origin. It has been investigated extensively and no evidence has been found that indicate that tectonic deformation has occurred in the zone at least since the Triassic (205 mybp). The staff concludes that the Brevard Zone is not capable.

The Kings Mountain belt is structurally made up of multiple folds and brecciated zones that trend north to northeast. The zones dip vertically or steeply to the southeast. Iverson and Smithson (1982) suggest that the root of the decollement extends beneath the Kings Mountain Belt. Horton (1981) reports the presence of 5 zones of mylonitic deformation that later underwent semibrittle deformation within the Kings Mountain Belt. The closest approach to the site of one of these zones is 10 miles. Radiometric dating of pegmatites related to one of the shear zones indicated that deformation occurred at least 350 million years before present (Horton, 1981). This confirms the staff's conclusion made in regard to the McGuire Nuclear Site, and following the Catawba CP review: that the faults of the Kings Mountain Belt are not capable.

The interpretation of geophysical data suggests the presence of an east-west or east-northeast trending fault (Wilson, 1981) that terminates about 5 miles east of the Catawba site. Geologic mapping by the U.S. Geologic Survey in that area is reported to have found no evidence for this fault, therefore it will not be shown on the soon-to-be-published Charlotte 2 degree map (Duke Power, 1981).

The applicant has geologically mapped the area within a 10-mile radius of the site. The results of that mapping are shown on Figure 2.5.1-9. The average strike and dip of schistosity and foliation is N 44° and 72° SE. Jointing most commonly strikes between N 35° W and N 50° W and between N 30° E and N 45° E. The regional drainage system is strongly influenced by these trends, particularly the northwest system. These trends are strong in bedrock at the site.

The nearest significant structure of regional size to the site is the Nanny Mountain anticline, located about 3 miles northwest of the site. Radiometric dating indicates that deformation that created this northeast striking fold occurred more than 300 mybp.

2.5.1.2 Site Geology

The site is located on the western shore of Lake Wylie, a reservoir formed by the construction of a dam in 1904 across the Catawba River downstream from the site. This dam was rebuilt and the water level was raised to its present level in 1925 (FSAR, page 2.4-1) The terrain consists of low rounded hills with site elevations ranging from +570 ft MSL at the shore of Lake Wylie to +640 before construction. The site is in the Charlotte belt of the Piedmont. Bedrock beneath the site is adamellite (quartz monzonite) that has been dated as 532 $\pm$ 15 million years old. Mafic dikes comprise a minor part of site bedrock. These rocks have been subjected to moderate to high grade regional metamorphism (amphibolite facies).

The site bedrock has been sheared and brecciated through geologic time and been intruded by hydrothermal minerals. The applicant has determined the historical geology development of the site (Table 2.5.1-4). Several phases of faulting have been detected, and the youngest faults that affect the site rocks are no younger than 86 $\pm$ 30 million years as demonstrated by radiometric dating (potassium argon) of undeformed mineral assemblages within the shear zones. The most prominent trends of these shear zones are N-S to N 15° W, with dips of 80° SW to vertical; and N 40° W to N 45° W with dips at 80° SW to vertical. However, many shears strike north, and north-northeast.

Two main trends of joints were mapped at the site, N 30° E to N 45° E, and N 35° W to N 50° W. Both trends dip 65° to 85° NE, respectively.

The site was investigated in considerable detail by techniques that included core borings, testpits and trenches, seismic surveys, in-hole seismic exploration, geologic mapping, permeability tests, and laboratory testing. Based on these studies the applicant has identified a profile of the subsurface materials. The natural drainage areas are filled to some extent by alluvial, silty sands. Residual soil (saprolite) derived from the weathering of adamellite, and consisting of sandy silt, and silty sand, overlies the bedrock. The saprolite ranges in thickness from a few feet to more than 30 feet. The saprolite grades into what the applicant has classified as partially weathered rock. Partially weathered rock is less weathered and harder than residual soil, and is defined as that material having a standard penetration test blow count of at least 100 blows per foot.

Continuous bedrock is defined by the applicant as that rock which has a Rock Quality Designation (RQD) of 75% or greater. Depth to continuous rock ranged from about 10 feet to 110 feet. All major Category 1 structures are underlain by continuous bedrock. The SNSW Pond Dam, intake and discharge structures, Refueling Water Storage Tanks and Diesel Generator Fuel Oil Tanks are founded on partially weathered rock. Both continuous rock and partially weathered rock are competent foundation materials.

Based on our review, there are no geologic hazards at the Catawba Nuclear Site.

2.5.2 Seismology

2.5.2.1 Summary

The conclusions reached at the construction permit (CP) review by both the staff and its advisor, the U.S. Geological Survey (CP-SER Supp. 1), were that 0.15 g (SSE) and 0.08 g (OBE) accelerations when used with appropriate response spectra are adequate for representing the ground motion caused by the maximum earthquake for the site. In the review process for the Operating License the Staff noted that the applicant used a Newmark 1967 spectrum, which was anchored to 0.15 g zero period acceleration (ZPA) for the SSE and to 0.08 g ZPA for the OBE.

At the Operating License review stage, the staff evaluated the Catawba seismic design spectrum by comparing it to present NRC standard practices. In terms of its relationship to maximum earthquake Modified Mercalli intensity MMI = VII the Catawba design should be equivalent to the Reg. Guide 1.60, 0.13 g ZPA design spectrum.

In terms of its relationship to magnitude, it is the staff's position that the Catawba design should meet the 84% percentile of the site specific spectrum of earthquake records with a mean magnitude of 5.3, (M_{blg}).

The staff concluded that, with a few exceptions fully discussed in this section, the Catawba seismic design criteria are acceptable.

2.5.2.2 Maximum Earthquake

The largest historic earthquakes in the Southern Piedmont have estimated modified Mercalli intensities (MMI) of VII. Two of these, the February 21, 1774 earthquake and the ~~February 10, 1874~~ December 23, 1875 earthquake occurred near Arvonia, Virginia, approximately 260 miles northeast of the site. A third, the January 1, 1913 Union County, S.C. earthquake occurred at a distance of approximately 40 miles from the site. Bollinger (1973) lists these earthquakes as having an intensity VII (MMI) and its equivalent Rossi-Forel VII-VIII, Barstow (1981) lists these earthquakes with intensities VII (MMI), Coffman and Von Hake (1977) list these earthquakes with intensity VII or VI-VII (MMI).

In the more recent Safety Evaluation Reviews, the Staff has maintained that the magnitude is a more appropriate measure of source strength of an earthquake. Magnitude is usually determined from instrumental records; however, Nuttli et al (1979) derived a magnitude estimate for the 1774 and the 1875 events from felt area and isoseismal area information. The estimated magnitudes range from 4.5 to 5.0 (m_b). In another study, Nuttli and Hermann (1978) indicated that an appropriate equivalent magnitude for an epicentral intensity of VII (MMI) is an $m_b = 5.3$. It is the staff's conclusion that the maximum historic earthquakes in the Southern Piedmont can be defined as having an estimated (maximum) magnitude of $m_b = 5.3$.

The August 31, 1886 Charleston, S.C. earthquake is listed in the Coffman and Von Hake catalogue (1977) with a meizoseismal intensity of MMI = IX-X. Bollinger (1977) estimated the maximum intensity to be MMI = X. The Charleston, S.C. region is presently under intensive investigation. Interpretations that have emanated from these studies differ considerably as far as the possible mechanisms are concerned. An extensive overview of the published studies on the subject can be found in a Corps of Engineers report dealing with the seismic design evaluation of a hydroelectric dam on the Virginia-North Carolina border (RSACE, 1981). The NRC and its advisors, the USGS and NOAA, undertook a similar survey of the publications on the subject during the OL safety evaluation review of the V. C. Summer Nuclear Station (USNRC, 1981). As mentioned before, the Charleston, S.C. area is being subjected to an intensive study by the USGS. See also discussion in Section 2.5.1.1. The staff's position has been that the Charleston seismicity is associated with a (regionally unique) tectonic structure in the Charleston-Summerville area and should not be assumed in the tectonic province approach for licensing purposes to occur anywhere else. Thus, in accordance with the Tectonic Province approach (Appendix A to 10 CFR 100), the maximum earthquake which shall be considered to occur near the site has a maximum intensity of VII (MMI) or a maximum magnitude of 5.3 (m_b). In addition, the effects of a recurrence of an 1886 Charleston earthquake in the Summerville-Charleston area shall be postulated to assess its influence on the Catawba site.

2.5.2.3 Safe Shutdown Earthquake (SSE)

At the CP stage (CP-SER) the staff concluded that a horizontal acceleration of 0.15 g used with an appropriate response spectrum was adequate for representing the ground motion for the maximum earthquake. During the OL review the staff identified three potential problems which it deemed worthy of further clarification.

1. The shape of the spectra which were used to define the SSE design for the Category I structures and equipment.

2. The adequacy of the amplification factors used to account for the shearwave velocity contrast and layer of unconsolidated material underlying some of the Category I structures.

3. The effects of ground motion at the site generated by a distant earthquake similar to the 1886 Charleston, S.C. event.

Requests for clarification to this effect were forwarded to the applicant, who supplied the following information:

i. All Category I structure founded on "continuous" rock (8500 fps) were designed to the Newmark, 1967, spectrum anchored to 0.15 g zero period acceleration.

ii. The floor spectra for the Category I structures on rock were obtained from a synthetic earthquake time-history, normalized to the Newmark spectrum anchored to 0.15 g ZPA.

iii. The Category I structures founded on "partially weathered" rock (2700 fps) were designed to a Newmark Spectrum, anchored to 0.15 g ZPA amplified spectrum, which took in consideration the amplification through the "partially weathered" layer.

iv. The effects of a 1886 Charleston, S.C. type earthquake were considered to be less severe than those of the maximum earthquake. The applicant assumed the 1886 type events to be constrained to the immediate vicinity of Charleston, S.C. The ground motion at the site was estimated to be equivalent to that of a local earthquake of intensity VI-VII.

The staff evaluated the information provided by the applicant and its conclusions are as follows:

I. The maximum earthquake in the Southern Piedmont has an intensity of MMI = VII (e.g., 1913 Union County, S.C. earthquake). In accordance with the Standard Review Plan the corresponding "zero period" acceleration (ZPA) for seismic design may be obtained by using Trifunac & Brady's "trend of the mean" formula (Trifunac and Brady 1975). Hence, the seismic design spectrum for Category I structures on rock should be equivalent to a Reg. Guide 1.60, 0.13 g ZPA response spectrum.

The staff compared the Catawba seismic design spectrum to the standard Reg. Guide 1.60, 0.13 g ZPA spectrum and concluded the seismic design to be generally acceptable. The small exceedences that do exist are noted below are discussed in Section 3.7.

I-a. The Reg. Guide 1.60 0.13G ZPA spectrum exceeds the Catawba design spectrum by 10% or less in the 0.06 to 1.5 second period range (0.7 to 16.5 Hz) and by 7% or less in the 0.24 to 0.44 second period range (2.3 to 4.2 Hz).

I-b. Beyond the 1-second period ($f \leq 1.0$ Hz) the Reg. Guide 1.60, 0.13 g ZPA spectrum exceeds the Catawba to a progressively larger amount. The rationale for the relatively insignificant impact of this exceedence is discussed in Section 3.7.

II. In more recent Safety Evaluation Reviews (e.g., Clinton, Perry, and Wolf Creek, NUREG-0853, 0881, and 0887, respectively), the staff has indicated that site specific spectra obtained from statistical analyses of appropriate suites of earthquake records are more in accord with the controlling earthquake size, frequency spectrum, and local site conditions. In this method the use of peak acceleration and Reg. Guide 1.60 spectrum shapes are replaced by spectra obtained from statistical analysis (84th percentile) of a suite of earthquake records from earthquakes with magnitudes equal to within one half magnitude of the Safe Shutdown Earthquake recorded at distances equal to or less than 25 km at sites which exhibit geologic conditions similar to those of the site in question. It is the staff's position that spectra obtained by this method afford a more realistic evaluation of the seismic design for Category I structures and components. Although a site specific spectrum has not been generated for the Catawba site specifically, one that would be reasonably representative for the Catawba site may be obtained from other similar (rock) sites.

The Wolf Creek (NUREG-0881) and Perry (NUREG-0887) site-specific spectra are obtained from records of nearby earthquakes with magnitudes of $m_b = 5.3 \pm 0.5$ and at distances equal to or less than 25 km, recorded at rock site stations.

The staff constructed a site-specific spectrum by averaging these 84th percentile site-specific spectra. By comparing this site specific spectrum to the Catawba design spectra the staff observed the following:

II.a Between the frequencies of 10 to 30 Hertz the Catawba design spectrum (Newmark 0.15 g ZPA spectrum) matches the site specific spectrum reasonably well.

II.b Between the frequencies of 3 to 10 Hertz the site-specific spectrum exceeds the Catawba design spectrum by 15% to 16%.

II.c At frequencies less than 3 Hertz the Catawba design spectrum exceeds the site specific design spectrum.

II-d The synthetic earthquake time history spectrum, discussed in Section 2.5.2.3-ii, exceeds the site specific spectrum at all frequencies.

The significance of the observations in relation to the seismic design of the plant are discussed under Section 3.7.

III. The staff compared the Catawba design spectrum developed for Category I structures founded on "partially weathered" rock to the average (rock) site specific spectrum discussed under Section 2.5.2.3-II.

The rationale for this comparison is that the ratio between the two spectra is a measure of the (estimated) amplification caused by the unconsolidated material. The staff concluded that the procedure used by the applicant to account for a layer of 25 feet of soil above unweathered bedrock (FSAR p. 2.5-26) account for a 250% amplification (maximal) in the 2.5 to 5 Hertz region. In order to determine whether or not this amplification is sufficient to account for the indicated difference in shear wave velocities (2700 fps for "weathered" rock v.s. 8500 fps for "continuous" rock) the staff is requesting the applicant to provide further justification on the subject.

IV. The intensity at the site as a result of an 1886 Charleston, S.C. earthquake of epicentral intensity X can be estimated by several attenuation functions (USGS, 1977, Gupta and Nuttli, 1976 and McGuire, 1977). Assuming that Summerville is the closest approach from the Charleston - Summerville, S.C. earthquake zone to the Catawba site, this distance is approximately 200 km. Using this distance and the above mentioned attenuation functions, the site intensity, as a result of an 1886 Charleston, S.C.-type earthquake ranges from MMI = VI to MMI = VII.

V. The effects of a large distant earthquake upon the Catawba site may be estimated also by a method proposed by Nuttli (1981). The distant earthquake to be considered at the site is a recurrence of the 1886 Charleston S.C. earthquake with magnitude 6.6 (m_b) (Nuttli, 1979), at a distance of 200 km. To evaluate the predicted acceleration and velocity at the site, spectra can be constructed by using the amplification factors proposed by Newmark (1978). A comparison of the 5% damped spectra indicates that the resulting ground motion at the Catawba site from a magnitude 6.6 earthquake at 200 km distance is less than the seismic design values used.

2.5.2.4 Operating Basis Earthquake

The applicant supplied information which indicated that the Catawba OBE design response spectrum used is a Newmark (1967) response spectrum anchored to

0.08 g ZPA. In accordance with Appendix A to 10 CFR 100, the OBE should be at least one-half of the SSE design. The 0.08 g OBE design acceleration is somewhat greater than one-half of the SSE design and is approximately equal to one-half of the site-specific spectrum discussed in Section 2.5.2.3. Therefore, the staff concludes that the OBE design acceleration is acceptable.

2.5.3 Surface Faulting

The faults mapped in the plant excavations are briefly described in Section 2.5.1.2. A more detailed description is given in the "Safety Evaluation of The Brecciated Zones at the Site of the Catawba Nuclear Station, Units 1 and 2" (USNRC, 1976). In that report, the NRC staff concluded that the site faults were demonstrated by radiometric methods to be at least 86 ± 30 million years old. We further concluded that the faults were not capable according to the criteria set forth in Appendix A.

As a result of our review of the FSAR material, we reaffirm that there are no faults at the site that represent a hazard to the Catawba site.

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