SECTION 2.4
HYDROLOGIC ENGINEERING

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT.)

LIST OF TABLES

LIST OF TABLES (CONT.)

LIST OF FIGURES

LIST OF FIGURES (CONT.)

LIST OF FIGURES (CONT.)

2.4 HYDROLOGIC ENGINEERING

The information in this section of the referenced DCD is incorporated by reference with the following departures and/or supplements.

Insert the following sections following Section 2.4 of the DCD.

VCS DEP 2.0-1

Section numbering of this section is based on Regulatory Guide 1.206 down to the X.Y.Z level, rather than following the AP1000 DCD numbering. Left-hand margin annotations indicate where DCD COL Items (VCS COL X.Y-#) have been responded to or supplementary information (VCS SUP X.Y-#) has been added.

{DEPARTURE JUSTIFICATION: Section 2.4 of the AP1000 DCD is not organized in a fashion that readily supports NRC review or applicant presentation of the required information. This administrative change is necessary to present the required information in a regulatory accepted fashion. Marginal annotations direct the reader to the proper location for the information required to be provided. This change is acceptable since it does not alter the information required to be provided.}

DCD

The AP1000 is designed for a normal groundwater elevation up to plant elevation 98' and for a flood level up to plant elevation 100'. For structural analysis purposes, grade elevation is also established as plant elevation 100'. Actual grade will be a few inches lower to prevent surface water from entering doorways.

For a portion of the annex building the site grade will be 107 feet to permit truck access at the elevation of the floor in the annex building and inside containment. Subsection 3.4.1 describes design provisions for groundwater and flooding.

The Combined License applicant will evaluate events leading to potential flooding to demonstrate that the site meets the site parameter for flood level. As necessary, the Combined License applicant may propose measures to protect the plant according to the Standard Review Plan, Section 2.4.10. Events to be considered are those identified in Standard Review Plan, Section 2.4.2.

Adverse effects of flooding due to high water or ice effects do not have to be considered for site-specific nonsafety-related structures and water sources outside the scope of the certified design. Flooding of water intake structures, cooling canals, or reservoirs or channel diversions would not prevent safe operation of the plant.

VCS COL 2.4-1

The ultimate heat sink in the AP1000 reactor design is the atmosphere, and the safety system is passive. The required emergency water supply for the AP1000 reactors at VCSNS Units 2 and 3 is stored in two tanks per unit on the site. Both tanks are missile-protected, and the primary tank is seismic Category I, located physically above the reactor containment, while the auxiliary tank is seismic Category II. No cooling water reservoirs or other facilities off the site are used to supply water for safety-related cooling at the plant, although normal operating makeup water is sourced from the Monticello Reservoir.

2.4.1 HYDROLOGIC DESCRIPTION

2.4.1.1 Site and Facilities

Units 2 and 3 are located in Fairfield County, South Carolina, approximately 1 mile east of the Broad River and 2 miles northeast of the Parr Shoals Dam. In general, elevations in Section 2.4 are given relative to the official plant datum, NAVD88. Numerous studies relative to other works in the region were made relative to the earlier datum NGVD29. The NAVD88 datum is 0.696 feet higher than the NGVD29 datum. Thus, all elevations given relative to NAVD88 are numerically 0.696 feet smaller than the same elevations relative to NGVD29. NAVD88 datum is, in general, within 0.39 inch of local mean sea level (MSL) along the South Carolina coast.

The Units 2 and 3 site is situated on a hilltop with a design plant grade elevation (equivalent to the DCD design plant grade of 100.0 feet) of 400 feet NAVD88, about 150 feet above the Broad River floodplain. The site is located about a mile to the south of the Monticello Reservoir, the upper pool of the Fairfield Pumped Storage Facility and the source of makeup water for normal operation of Units 2 and 3 (see Figure 2.4-201 and Figure 2.4-202). In addition, the Monticello Reservoir provides cooling and makeup water for Unit 1.

Figure 2.4-203 shows the general site layout including safety-related structures (*i.e.,* nuclear island), topography, and changes to the natural drainage. The Units 2 and 3 site is not susceptible to flooding from the Broad River due to its relative height above the river. The design plant grade is at an elevation of 400 feet NAVD88 which is about 25 feet below the maximum operating level of the Monticello Reservoir of 425 feet NGVD29. As Figures 2.4-201 and 2.4-203 indicate, the site is bounded on the north by the Unit 1 site, which is at an average plant grade elevation of 435 feet NGVD29. The Unit 1 site is bounded on the west by an unnamed creek, the upper reaches of which have thalweg elevations between 300 and 360 feet—substantially lower than the general plant grades of both Unit 1 and Units 2 and 3. The Unit 1 site is bounded on the southeast by the upper reaches of Mayo Creek, which is also between El. 300 and 360. The Units 2 and 3 site is bounded on the west by another unnamed creek, the thalweg elevations of the upper reaches of which are also between El. 300 and 360 (40 to 100 feet below the design plant grade level of 400 feet NAVD88 for safety-related facilities). On the east, the site is also bounded by the upper reaches of Mayo Creek, the thalweg of which is also between 40 and 100 feet below design plant grade. Pre- and post-development runoff from the site discharges to the Mayo Creek and the unnamed creeks. Development of the site results in little alteration to predevelopment drainage patterns. Both the unnamed creeks drain into the

Parr Reservoir and the Mayo Creek drains into the Broad River downstream of Parr Shoals Dam. The Parr Reservoir has a normal full pool elevation of 266 feet NGVD29. Thus, the Monticello Reservoir is the only water body in the vicinity of the Units 2 and 3 site that could be an external source of flooding. It is clear from the topography of the area as shown in Figure 2.4-201 that there is no route by which any waters from the Monticello Reservoir could reach Units 2 and 3.

Units 2 and 3 are AP1000 nuclear-powered electric generating units, each of which includes a reactor with a core rating of 3400 MWt supplying steam to a turbine generator unit capable of supplying at least 1000 MWe to the electric grid. The plants are served by a system of four circular evaporative cooling towers arranged as shown in Figure 2.4-203. The maximum rate of makeup water withdrawal from the Monticello Reservoir for all Units 2 and 3 is about 61,600 gpm or 272 acre-feet per day. This volume is equal to about three times the evaporation loss from the reservoir, which has been estimated to be 109 acre-feet per day (Reference 227). This volume, however, is very small compared with the 29,000 acre-feet pumped daily into and out of the Monticello Reservoir by the Fairfield Pumped Storage Facility (Reference 227). As discussed in Subsection 2.4.11, there is sufficient storage in the Monticello and Parr Reservoirs to permit full operation of Units 2 and 3 through the 100-year drought low flow on the Broad River.

AP1000 nuclear reactors use safety-related passive ultimate heat sink systems with built-in water storages that do not require an external safety-related ultimate heat sink to reach safe shutdown. Therefore, the AP1000 units do not depend on the Monticello or Parr Reservoirs for safe shutdown during a design basis accident.

As described in Subsections 2.4.2 through 2.4.10, there are no significant flood risks from the rivers or lakes in the vicinity, and the site grading and drainage system will prevent flood damage to all safety-related facilities based on the design plant grade elevation of 400 feet NAVD88.

2.4.1.2 Hydrosphere

This subsection describes the location, size, shape, and other hydrologic characteristics of the streams and reservoirs comprising the surface water hydrosphere. A description of the groundwater environment influencing the site is included in Subsection 2.4.12.

2.4.1.2.1 Rivers and Streams

The region surrounding the site is characterized by a network of small tributaries and a few large rivers draining the rolling, low-profile terrain. The Broad River, the principal hydrologic feature in the site vicinity, drains an area of about 4,750 square miles upstream of the site. The drainage area is located between two southeast-northwest trending ridges stretching from Columbia, South Carolina to the headwaters about 100 miles northwest in North Carolina. The average annual precipitation is 45 inches (Reference 226) with a runoff of about 17.8 inches

(Reference 209), equivalent to an average annual runoff volume over the entire watershed of 4.5 million acre-feet. In the vicinity of the site, the Broad River at Parr Reservoir is about 2000 feet wide, with depths ranging from a few feet to around 15 feet (Reference 226). Many streams and creeks carry runoff and groundwater drainage into this watercourse. Important rivers draining into the Broad River upstream of the site include the Enoree River, the Tyger River, and the Pacolet River. Downstream of the site the Broad River joins the Saluda River near Columbia forming the Congaree River.

The nearest downstream active stream flow gauging station on the Broad River is USGS Station 02161000 at Alston, South Carolina. As shown in Figure 2.4-204, the Alston station is located about 1.2 miles downstream of the Parr Shoals Dam and has a contributing drainage area of approximately 4,790 square miles (Reference 209). Stream flow measurements at this station began in October 1896. They were discontinued at the end of 1907, and restarted in October 1980. The Alston station continues to operate to this date. The mean annual daily flow at Alston based on all available data (water years 1897–1907 and 1981–2005) is 6,302 cfs (Reference 209).

The next nearest downstream gauging station on the Broad River at Richtex (USGS Station 02161500) was discontinued in 1983. As shown in Figure 2.4-204, the Richtex station was located about 14 miles downstream of the Parr Shoals Dam and had a contributing drainage area of approximately 4,850 square miles (Reference 209). Stream flow data from this station exists from October 1925 to July 1928, and from October 1929 to September 1983. The mean annual daily flow for the available record is about 6,155 cfs.

The nearest active stream flow gauging station on the Broad River upstream of the site is USGS Station 02156500 near Carlisle, South Carolina. The Carlisle station is located about 21 miles upstream of the site, and has a contributing drainage area of approximately 2,790 square miles (Reference 209). Stream flow measurements at this station began in 1938 and continue today. The mean annual daily flow at this station for water years 1939 to 2005 is 3,880 cfs.

As shown in Figure 2.4-204, the Carlisle gauging station is located upstream of the confluences of the Tyger and Enoree Rivers with the Broad River. Its drainage area is about 4% less than the 4,750 square miles drainage area of the Broad River near the site, which is located downstream of these two tributaries. However, the drainage areas at the Alston and Richtex gauging stations are only about 1% and 2% greater, respectively, than the drainage area of the Broad River at its closest point to the site. This, combined with the fact that the Alston and Richtex stations have longer records, makes these two stations more suitable to be used to characterize the flow conditions of the Broad River near the site.

Table 2.4-201 summarizes the key hydrologic data for the Alston, Richtex, and Carlisle gauging stations (Reference 209).

2.4.1.2.2 Lakes and Reservoirs

The nearest body of water to the site is the Monticello Reservoir that has a drainage area of about 17.4 square miles (Reference 226), and is formed by the Frees Creek Dams (see Figure 2.4-202). The main dam, referred to also as Dam B, has a maximum height of 160 feet, measured from the original channel bottom of Frees Creek, and a crest length of approximately 4,700 feet. Three smaller saddle dams have lengths of 3,130 feet (Dam A), 2,000 feet (Dam C), and 1,300 feet (Dam D), with maximum heights from 50 to 90 feet. All four dams have crest elevations of 434 feet NGVD29 and are of earthfill construction with appropriate riprap protection (Reference 226). The Monticello Reservoir has a surface area of about 6,800 acres and a storage volume of about 400,000 acre-feet at normal maximum water surface El. 425 feet NGVD29. A part of the Monticello Reservoir (Monticello Sub-Impoundment), covering an area of about 300 acres, is used for recreation purposes. The maximum daily withdrawal for power generating purposes is 29,000 acre-feet, lowering the pool to El. 420.5 feet NGVD29 and reducing the reservoir surface area to approximately 6,500 acres (Reference 225). Pumping operations during periods of off-peak power demand refill the reservoir. Figure 2.4-205 presents the area and storage capacity curves for the Monticello Reservoir (Reference 225). Table 2.4-202 lists the data that the curves shown in Figure 2.4-205 are based on.

The Parr Reservoir constitutes the lower pool of the Fairfield Pumped Storage Facility, and is located approximately 1 mile to the west of Units 2 and 3 on the Broad River. This reservoir is formed by Parr Shoals Dam, constructed in 1914 and owned by SCE&G. Parr Shoals Dam, located about 2 miles southwest of the site, is 2,715 feet long, approximately 48 feet high, and has a 2,000-foot-long concrete gravity spillway section with 9-feet high spillway crest gates, with a crest (top of gate) elevation of 266 feet NGVD29. The dam is joined on the western end by an earth dike, about 300-foot-long with a crest elevation of 272 feet NGVD29, and on the eastern end by a 300-foot-long integral powerhouse section, a 90-foot-long concrete non-overflow section with a crest elevation of 271.1 feet NGVD29, and a 25-foot-long earth-fill section (Reference 226).

The Parr Reservoir originally had a surface area of 1,850 acres with normal pool elevation of 257 feet NGVD29, and extended about 8.5 miles upstream. In 1977, the Parr Shoals Dam crest was raised approximately 9 feet by the installation of spillway crest gates, mounted on top of the concrete portion of the dam. The gates are hinged at the bottom and are raised by means of hydraulic cylinders located on the downstream side of each gate. Raising the gates increases the effective height of the dam. With the gates in the raised position, a maximum pool elevation of 266 feet NGVD29 is achieved (Reference 226)

At El. 266 feet NGVD29, the Parr Reservoir extends approximately 13 miles upstream and has a usable storage capacity of 29,000 acre-feet with a surface area of approximately 4,400 acres. At normal minimum pool elevation of 256 feet NGVD29, the surface area is about 1,400 acres with a dead storage volume of about 2,500 acre-feet. The operating drawdown of the pool is 10 feet. Figure 2.4-206 presents the area and storage capacity curves for the Parr

Reservoir (Reference 226). Table 2.4-203 lists the data that the curves shown in Figure 2.4-206 are based on.

In addition to the Parr and Monticello Reservoirs, a number of reservoirs exist upstream and downstream of the site on the Broad River and its tributaries. Most of these reservoirs are small, low-head dams for hydroelectric power generation and water supply. The pertinent data for the 78 upstream reservoirs with 200 acre-feet of storage or more is included in Table 2.4-204 (Reference 239). Their locations are shown in Figure 2.4-207. Table 2.4-204 includes data on drainage areas, types of dam, dam length and height, year of construction, and regulating agency. Elevation-storage relationships for reservoirs other than the Monticello and Parr Reservoirs are not needed because these dams are very small and storage behind them is minimal. Because all of the upstream dams are run-of-the-river reservoirs, there is no distinction between their short- and long-term storage allocations.

Several studies for the construction of proposed major dams on the Broad River have been done in the past (Reference 240). The latest study (Reference 240), completed in 1969, reported that the only reasonably feasible location for a major dam in the Broad River watershed is at the Clinchfield site. As shown in Figure 2.4-208, the site of this proposed dam is located in the upper reaches of the Broad River basin in North Carolina, approximately 100 river miles upstream of the VCSNS site. If constructed as proposed in 1969 (Reference 240), this dam would have a drainage area of 571 square miles and a crest elevation of 830 feet NGVD29, 153 feet above the Broad River streambed. The conservation pool would be at El. 810.5 feet NGVD29, with 830,500 acre-feet of storage and 20,220 acres surface area. The flood control pool would be at El. 820 feet NGVD29, with 1,036,000 acre-feet of storage and 23,180 acres of surface area. A volume of 716,000 acre-feet would be allocated for water supply, 90,000 acre-feet for water quality management, and 205,000 acre-feet for flood control. The maximum water surface El. of 825 feet NGVD29 would be reached with the occurrence of the spillway design flood coincident with the full flood control pool (Reference 226 and Reference 240).

A search of different sources did not reveal any reference to plans for the construction of the Clinchfield dam, which suggests that such plans most likely have been abandoned at this time. Even though the construction of the Clinchfield dam is highly unlikely, this potential reservoir is considered in the safety analysis for Units 2 and 3, to account for the unlikely possibility that plans for the construction of this dam are revived within the life of these units. The dam failure analysis considering the impact of Clinchfield is reported in Subsection 2.4.4.

2.4.1.2.3 Surface Water Users

Downstream of Units 2 and 3 on the Broad River, surface water is withdrawn by a number of municipalities and industries (see Table 2.4-205). The largest downstream surface water users in 2005 were:

- The Eastman Chemical Voridian Division with an average daily use of 72.3 million gallons
- The Santee Cooper Cross Station on Lake Moultrie with an average daily use of 59.7 million gallons
- The Charleston CPW (now Charleston Water System) on the Back River Reservoir with an average daily use of 46.2 million gallons
- The city of Columbia Canal Water Plant with an average daily use of 34.5 million gallons.

The city of Columbia has two water treatment plants, one on the Broad River (the Columbia Canal) and one on Lake Murray. Each facility can produce enough water for the entire city system, but normally water is produced at both facilities. Only the facility on the Columbia Canal is directly downstream of Units 2 and 3. Lake Murray is located on the Saluda River, a tributary to the Broad River downstream of Units 2 and 3. Several smaller downstream users are also listed in Table 2.4-205, which also gives their average daily use in 2005 based on data from the South Carolina Department of Health & Environmental Control (SCDHEC) (Reference 222). The nearest downstream user to Units 2 and 3 is the Columbia Canal Water Plant, located approximately 28 miles downstream. Figure 2.4-209 shows the rivers and lakes where the downstream water users listed in Table 2.4-205 are located. The only major upstream users that have been identified are the municipalities of Spartanburg, which withdraws an average of 33 mgd from the Pacolet River and Lake Blalock, and Gaffney, which withdraws an average of 7.7 mgd from the Broad River and Lake Whelchel. Neither of the identified upstream users will impact the safety-related features of Units 2 and 3, nor will they significantly affect the low flows in the Broad River, as discussed in Subsection 2.4.11.

VCS COL 2.4-2

2.4.2 FLOODS

2.4.2.1 Flood History

Data on historical floods is available at Richtex (1925–1983) and Alston (1980–2003). Although there is a 10-year period of record at the Alston gauge from 1897 to 1907, the data from this period was not used because of the 73-year gap before the gauge was reinstituted in 1980. The length of the gap gave rise to concerns about consistency of data, so it was decided to exclude the early record from further analysis. The data used indicates two flood seasons—one from January to April and the other from July to October. Floods during the latter period are generally associated with hurricanes and have usually been of greater magnitude than those occurring from January to April (Reference 224). The highest flood of record at Richtex had a peak discharge of 228,000 cfs, which occurred on October 3, 1929. Table 2.4-206 summarizes the major historic floods at Richtex, their peak discharge rates, and their maximum water surface elevations. Also included in this table are the estimates of the corresponding discharges and water levels at Parr Shoals Dam. The discharges at Parr Shoals Dam were estimated by multiplying the recorded flow values at Richtex and Alston stations by the respective drainage area ratios. This is considered an appropriate method of adjustment because of the very small differences in drainage areas (Alston is 1% larger than Parr; Richtex is 2% larger). The water levels in Parr were estimated using the weir equation at Parr Shoals Dam with a crest length of 2,000 feet, and the historical crest elevation of 257 feet NGVD29 for floods before 1977. A weir discharge coefficient of 3.97 was adopted for the spillway section (Reference 214). Backwater losses were not included in this calculation, because the Units 2 and 3 site is only 2 miles upstream from the dam, and they would be minor when the river is in flood stage. The operation of the pumped storage project post-1977 may affect the computed water surface elevations in the Parr Reservoir. The elevations for post-1977 floods were computed based on the assumptions that the gates were in their lowered positions, and the reservoir was essentially full at the time of the flood, so that reservoir storage had a negligible effect on peak outflow. These assumptions are consistent with SCE&G operating rules for the Parr Shoals Dam spillway, which require that the gates be opened whenever the river flow exceeds 40,000 cfs.

The Monticello Reservoir is an off-stream pond operated as the upper reservoir of the Fairfield Pumped Storage Facility. Its historical operating range is from El. 420.5 feet NGVD29 to El. 425 feet NGVD29. Operation of the pumped storage project and the small contributing drainage area largely prevents the effects of floods from filling the reservoir above the operating maximum.

2.4.2.2 Flood Design Considerations

As described briefly at the beginning of Section 2.4, the AP1000 standard design is based on a design basis flood level of 100 feet. At Units 2 and 3, this corresponds to El. 400 feet NAVD88, the design plant grade.

Subsections 2.4.2.3 and 2.4.3 through 2.4.7 summarize and identify the individual types of flood-producing phenomena, and combinations of these events that were considered to establish the design basis flood level for the plant safety-related features. The design basis flood level is established by considering the worst single phenomenon identified in Subsections 2.4.2.3 and 2.4.3 through 2.4.7.

The probable maximum flood (PMF) on Frees Creek and the Monticello Reservoir, coincident with the maximum operating water level of the Monticello Reservoir and related wind setup and wave run-up, is addressed in Subsection 2.4.3. For this flooding scenario, the maximum PMF floodwater level for the Monticello Reservoir was estimated as 437.85 feet NGVD29 (437.15 feet NAVD88). However, the site of Units 2 and 3 is not located within the Frees Creek/Monticello Reservoir watershed. For the site to be impacted by flooding on the Monticello Reservoir, the water level would have to exceed 438 feet NGVD29, which is the crest of the dike (north berm) along the shoreline of the Monticello Reservoir north of the site of Unit 1 (Reference 226). The dike crest elevation is 0.15 feet higher than maximum estimated water level in the Monticello Reservoir because of the combined effect of the PMF and wind setup and wave run-up. As can be seen in Figure 2.4-201, the site of Unit 1 is between the Units 2 and 3 site and the Monticello Reservoir. The Unit 1 site is protected by the dike, which coincidentally will also protect Units 2 and 3.

Subsection 2.4.4 addresses the potential flooding from cascading dam failures upstream on the Broad River during a PMF event. The analysis performed in this section shows that the maximum floodwater surface elevation in the Broad River (adjacent to the site) was estimated as 379.6 feet NGVD29 (378.9 feet NAVD88). Because this water level is about 21 feet below the safety-related facilities' design plant grade elevation of 400 feet NAVD88, it is concluded that the plant site is safe from seismically induced potential upstream dam failures during a PMF event.

Subsection 2.4.5 describes the surge and seiche flooding potential at the Units 2 and 3 site. Although the Monticello Reservoir is the only water body in the vicinity of Units 2 and 3 that could be a source of surge or seiche flooding, it does not present a risk to Units 2 and 3. It is clear from the topography of the area and site as shown in Figures 2.4-201 and 2.4-203 that there is no route by which any waters from the Monticello Reservoir could reach Units 2 and 3. Moreover, because the plant site is located nearly 150 miles from the nearest coast, the site is not subject to any coastal surge and seiche flooding. Therefore, surge and seiche flooding was found to be no risk to Units 2 and 3.

Potential flooding from tsunamis is not applicable to Units 2 and 3 because of the location of the site, as discussed in Subsection 2.4.6.

Subsection 2.4.7 addresses the potential flooding due to ice effect at the Units 2 and 3 site. Due to lack of significant ice cover formation in the area, it is concluded that there is no risk of ice-related flooding at the Units 2 and 3 site.

In Subsection 2.4.10 under "Flood Protection Requirements," it is concluded that the safety-related structures at Units 2 and 3 are not subject to flooding. No additional flood protection measures, other than the dike already installed for Unit 1, and no emergency procedures are required.

The effect of flooding on safety-related structures because of a postulated occurrence of a local probable maximum precipitation (PMP) storm event coincidental with ice-related or any other blockage of the subsurface site drainage system is addressed in Subsection 2.4.2.3. The analysis performed in this subsection shows that the maximum floodwater surface elevation due to local intense precipitation is 399.4 feet NAVD88 near safety-related structures.

2.4.2.3 Effects of Local Intense Precipitation

The effect of local intense precipitation at the Units 2 and 3 site was evaluated by performing a site drainage analysis following the guidelines provided in Section 11 of American Nuclear Society, ANSI/ANS 2.8-1992 (Reference 201), which requires that the maximum water level associated with potential flooding resulting from the local PMP be determined. For this purpose, the performance of the site area storm drainage system was analyzed for the local intense PMP assuming complete clogging of the subsurface drainage system. Since no credit is taken for the site drainage system, it is not described.

PMP depths for the local site area were estimated for durations of 5, 15, 30, and 60 minutes up to a period of 6 hours, following the procedures outlined in Hydrometeorological Report No. 51 (Reference 242) and Hydrometeorological Report No.52 (Reference 243). The estimated values are given in Table 2.4-207. The temporal distribution of the six-hour local PMP was estimated following the procedure outlined in the Army Corps of Engineers Bulletin 52-8 (Reference 231). The six-hour PMP depth for the local area was estimated to be 30.4 inches.

The effect of the PMP on the site was analyzed following the Plant Site Drainage guidelines provided in Section 11 of ANSI/ANS 2.8-1992. Figure 2.4-210 shows the conceptual drainage grading plan including the surface and subsurface drainage systems for the plant site, as well as the slopes of key graded areas. The site area in the immediate vicinity of the plant buildings is bounded by the plant access roads, drainage ditches, and storm water basins. The plant site is graded to permit overland drainage flow away from the buildings.

As shown in Figure 2.4-210, the main plant site area is divided into four discrete subbasins, each of which has one or more distinct drainage outlets. Subbasin 1 in Figure 2.4-210 covers the western part of the site including Unit 3. Subbasin 2 covers the eastern part of the site including Unit 2. Subbasin 3 covers the northern part of the site, including the parking lot. Subbasin 4 covers the southern part of the site including the cooling tower pad. All directions mentioned in Subsection 2.4.2.3 are with respect to the Plant North shown in Figure 2.4-210. The drainage outlet for each subbasin is also shown in Figure 2.4-210.

As shown in Figure 2.4-210, there are two additional drainage areas (*i.e.,* Subbasin A to the east and Subbasin B to the north) located outside the main plant site area that may contribute runoff to the adjoining Subbasin 2 during an extreme storm event such as the PMP. For simplicity, it is conservatively assumed that during the PMP event, the entire runoff from Subbasin A flows into Subbasin 2, assuming that the culvert under the railroad to the south is blocked as in accordance with ANSI/ANS 2.8-1992. The runoff from Subbasin B may be blocked by the road coming out of the site and going towards the north. This was analyzed for the PMP storm event and the maximum water surface elevation for peak flow over the road was estimated to be about 387.2 feet NAVD88. This water level is about 7 feet below the lowest point of Subbasin 2. Therefore, Subbasin B will not contribute any runoff to the main plant site area (*i.e.,* through Subbasin 2) even during the PMP event.

The potential flooding of safety-related structures at the plant site was evaluated assuming that the plant site receives runoff from the entire local drainage area, including drainage from the roofs of all the onsite structures, and the sidehills surrounding the plant site. To maximize the effect of the PMP, it was assumed that during the PMP event, the ground would be fully saturated, which suggests that there would be no losses due to infiltration and all precipitation would run off.

These saturated ground conditions would represent the antecedent storm conditions before the PMP event (Reference 201).

For conservatism, it was also assumed that at the same time all road crossing culverts, underground storm drainage pipes, and storm water ponds would be completely blocked or impaired.

The analysis of flooding potential at the site and the determination of the maximum water levels at the safety-related structures (see Figure 2.4-210) resulting from the local intense precipitation, based on the six-hour PMP presented in Table 2.4-207, consisted of the following two steps:

- Estimation of peak runoff rates from the Rational Method
- Determination of floodwater levels using the HEC-RAS hydraulic model (Reference 236).

Conservative assumptions were made in determining the peak runoff rates from the plant site area. A five-minute duration local PMP of 6.2 inches was used to determine the rainfall intensity of 74.4 inches/hour and no precipitation losses were assumed in the estimation of the peak runoff rates at the plant site.

Maximum water levels at safety-related structures of the main plant site area were calculated with the hydraulic model HEC-RAS (Version 3.1.3) developed by the U.S. Army Corps of Engineers (USACE) (Reference 236). This model uses step-wise backwater equations to estimate hydraulic flow parameters such as water levels and flow velocities for open channel systems. The steady-state option in the HEC-RAS model was used with input parameters including cross-section geometry, Manning's roughness coefficients, and flow boundary conditions.

Conservative values of the HEC-RAS input parameters intended to maximize the calculated water surface elevations along each of the drainage flow paths were used in the analysis. A value of 0.04 for Manning's roughness coefficient was used to define the channel bed and the over-bank roughness characteristics. No credit is taken for storage in the three storm water ponds on site. The downstream model boundary condition at the drainage outlets of Subbasins 1, and 2 was set equal to the critical depth as the areas drain onto fairly steep fill slopes. Subbasin 3 includes the parking area at its most downstream end. The depth of flow at the two outlet points of this subbasin is estimated by considering the entire flow from this subbasin as flowing over a broad crested weir of length equal to the total length of the two outlets. This depth is applied then at the upstream (south) end of the parking lot and it is used as downstream boundary condition for the HEC-RAS analysis that gives the water depth further upstream, *i.e.,* towards the south end of Subbasin 3. A critical depth downstream condition is used at the farthest downstream end of Subbasin 4, near Storm Water Pond 3.

The maximum water surface elevation at the plant site was estimated as 399.8 feet NAVD88, which occurs at the upstream-most point of Subbasin 4. The maximum floodwater surface elevation near safety-related structures due to local intense precipitation is 399.4 feet NAVD88. This elevation is 0.6 feet below the design plant site grade elevation of 400 feet NAVD88. Therefore, the safety-related structures for Units 2 and 3 would be safe from potential flooding during the local intense precipitation event, even if the entire underground storm drainage systems is completely blocked.

2.4.3 PMF ON STREAMS AND RIVERS

The PMF was derived for the two water bodies located adjacent to the site of Units 2 and 3: the Broad River and Frees Creek. There are no other streams that could cause flooding at the site of Units 2 and 3. The site is considered as a flood-dry site; therefore, approximate methods were used combined with conservative assumptions to estimate the maximum PMF water levels in the Broad River and Frees Creek.

The PMF analysis performed for Unit 1 (Reference 226) was based on PMP estimates obtained using Hydrometeorological Report No. 33 prepared by the National Weather Service (Reference 250). Since 1976, the National Weather Service (now under NOAA) updated the PMP estimates and published new guidelines for estimating the PMP in Hydrometeorological Reports 51, 52, and 53 (References 242, 243, and 244). In general, PMP estimates obtained using later hydrometeorological reports are greater and of longer duration than those based on Hydrometeorological Report No. 33. Thus, for this section, the PMF analysis for Unit 1 (Reference 226) was updated to incorporate the latest PMP information from Hydrometeorological Reports 51, 52, and 53 and the most recent hydrologic data.

2.4.3.1 Probable Maximum Precipitation

2.4.3.1.1 Broad River

Figure 2.4-204 shows the Broad River watershed at the Richtex USGS gauging station. The total drainage area at this location is about 4,850 square miles. The shape, size, and orientation of the watershed were used in the PMP estimation for the Broad River watershed according to the procedures outlined in Hydrometeorological Reports 51 and 52 and the ANSI/ANS 2.8-1992 guidelines. The storm was oriented to maximize the precipitation volume over the watershed. Table 2.4-208 summarizes the PMP depths estimated for the watershed for durations of up to 72 hours. The 72-hour PMP for the Broad River watershed was estimated to be at 22.1 inches. The temporal distribution of the 72-hour PMP is presented in Table 2.4-209 in six-hour time increments.

2.4.3.1.2 Frees Creek

The PMP for the Frees Creek watershed was estimated using the procedures outlined in Hydrometeorological Reports 51 and 52. Figure 2.4-202 shows the Frees Creek watershed at the Monticello Reservoir. The total drainage area at this location is about 17.4 square miles (Reference 226). Because of the small drainage area, no adjustments to orientation or shape were required.

Table 2.4-210 summarizes the PMP depths estimated for the Frees Creek watershed in six-hour intervals for a duration of up to 72 hours. The 72-hour PMP for the Frees Creek watershed was estimated to be 48.6 inches.

2.4.3.2 Unit Hydrograph for the Broad River Watershed

The UFSAR for Unit 1 used the unit hydrograph method to determine the PMF for the Broad River watershed (Reference 226). This unit hydrograph was developed using historical rainfall and runoff records associated with the storm event that occurred on August 16, 1940. The validity of this unit hydrograph was checked by comparing it with the unit hydrograph derived from data for the most recent major storm event on October 14, 1990. The latter was derived using the parameter optimization routine of the HEC-1 model developed by the USACE (Reference 237).

Figure 2.4-211 shows the two-unit hydrographs, one developed from the data for the August 16, 1940 storm and the other from the data for the October 14, 1990 storm. The overall shape of these two-unit hydrographs is very similar. Additional validation of the unit hydrograph based on the August 16, 1940 storm was obtained by computing the flood hydrographs produced by two different more recent storms and comparing the computed with the measured hydrographs from these storms. The two storms used for this purpose were those which occurred on October 8, 1976 and on October 14, 1990. Figure 2.4-212 and Figure 2.4-213 compare the computed with the observed flood hydrographs for these two storms. As can be seen in these figures, the measured and the computed flood hydrographs are very close, which further confirms the validity of the 1940 unit hydrograph.

Since the 1940 unit hydrograph was calibrated based on a much smaller storm event (*i.e.,* the 1940 storm) compared to the PMP storm event, the calibrated 1940 unit hydrograph ordinates were adjusted to account for nonlinearity effects in the runoff process under PMP conditions. The 1940 unit hydrograph ordinates were adjusted by increasing the hydrograph peak by 20% (from 63,175 to 75,800 cfs) and decreasing the time to peak by 25% (from 48 to 36 hours) in accordance with recommendations in the USACE EM 1110-2-1417 (Reference 235).

2.4.3.3 PMF for the Broad River

The flood hydrograph of the PMF at Richtex was derived using the rainfall-runoff model HEC-HMS developed by the USACE (Reference 238). Input data used in the HEC-HMS rainfall-runoff model includes a rainfall hyetograph, a unit hydrograph, precipitation losses, and base flow data. The specific data used for the Broad River model is discussed below:

- A 72-hour duration rainfall hyetograph was specified as input to the model, using the estimated PMP depths for six-hour time increments presented in Figure 2.4-214. In accordance with ANSI/ANS 2.8-1992, before the 72-hour PMP storm event, a storm with rainfall depths equivalent to 40%

of the 72-hour PMP values was routed through the watershed. The 40% PMP storm ended 3 days before the PMP storm.

- The 1940 unit hydrograph obtained from the UFSAR, which was validated with data from the 1990 storm event (see Subsection 2.4.3.2), was used as input to the model to represent the rainfall-runoff response characteristics of the Broad River watershed.

- Precipitation losses were modeled using the "Initial and Constant" method in HEC-HMS. The model uses the losses, along with the precipitation data and unit hydrograph, to determine the direct runoff hydrograph corresponding to the excess rainfall. The initial rainfall loss represents the amount of infiltrated or stored rainfall before surface runoff begins. The constant rainfall rate determines the rate of infiltration that occurs after the initial loss is satisfied. As the PMF peak flow is often insensitive to the initial rainfall loss (Reference 238), this value was set equal to zero in the rainfall-runoff model. Based on typical constant rainfall loss rate values (Reference 238), and the loss rate values estimated for the 1976 and 1990 storm events, a constant rainfall loss rate of 0.06 inch/hour was adopted in the model.

- The subsurface flow or the base flow rate was calculated in accordance with the procedures in ANSI/ANS 2.8-1992. The base flow at the beginning of the storm was set equal to the monthly average flow rate for the stream, which is reported as 6,160 cfs at Richtex.

Figure 2.4-214 shows the PMP rainfall hyetograph and the PMF hydrograph for the Broad River watershed at Richtex derived from the HEC-HMS rainfall-runoff model. The estimated peak discharge of the PMF hydrograph is 1,132,879 cfs. The peak PMF discharge for the Broad River watershed at the Parr Reservoir, with a drainage area of 4,750 square miles, was estimated to be equal to 1,109,521 cfs. This estimate was obtained by multiplying the peak PMF discharge of 1,132,879 cfs at Richtex with the ratio of the two drainage areas (4,750/4,850). Since the two locations are close to each other, and the difference in drainage areas is only about 2%, adjustment of peak flows by simple drainage area ratio is considered appropriate in this case. Orographic effects are accounted for in the PMP calculations leading to the hyetograph used in this analysis. Antecedent snowpack considerations are not relevant to the Broad River watershed because of the mild climate of the region.

The PMF flood elevation at the Parr Reservoir was estimated using the following conservative assumptions for the Parr Shoals Dam spillway gate operation and the PMF flow hydrograph:

- During the PMF, the spillway crest gates on the Parr Shoals Dam remain closed at the raised position with a top elevation of 266 feet NGVD29, rather than being lowered to the concrete ogee dam crest elevation of 257 feet NGVD29.

- Any attenuation of the peak PMF discharge (1,109,521 cfs) as it flows through the Parr Reservoir is neglected.

Based on these assumptions, the peak flood stage at Parr Shoals Dam was estimated using the standard weir equation with a weir discharge coefficient of approximately 3.9 (Reference 214) for the gated section with a crest length of 2,000 feet. This weir coefficient was adopted based on review of the layout of the curved surfaces of the gates, which were taken to behave in a manner similar to an un-vented ogee crest. The non-overflow section to the west of the gated section was treated as a broad-crested weir with a crest elevation of 272.1 feet and a length of 300 feet. A weir coefficient of 3.1 was adopted to represent the broad crested weir. The concrete non-overflow section to the east of the powerhouse was also treated as a broad crested weir with a crest elevation of 271.1 feet and a length of 90 feet. A weir coefficient of 3.1 was adopted to represent the broad crested weir. The 25-foot-long earthfill section at the extreme east end of the dam was neglected. The peak flood stage was calculated to be 25.5 feet above the top of the gates, *i.e.,* at El. 266 + 25.5 = 291.5 feet NGVD29, or 290.8 feet NAVD88. Since this is well below the design site grade elevation of 400 feet NAVD88, it is not necessary to perform analysis of coincident wind wave activity including wave run-up and setup in accordance with Subsection 9.2.1.1 of ANSI/ANS 2.8-1992. It should be noted that the Units 2 and 3 site is bounded on the east and west sides by the Mayo Creek and a small unnamed creek, both of which are tributaries to the Parr Reservoir. Because of the very small drainages of these two creeks relative to that of the Parr Reservoir, and due to the steepness and width of the two creeks, backwater on the creeks has been neglected.

2.4.3.4 PMF for Frees Creek

As indicated in Figure 2.4-202, the Monticello Reservoir, with a surface area of 10.6 square miles at the maximum water level elevation of 425 feet NGVD29, inundates about 60% of the Frees Creek watershed (17.4 square miles). Therefore, a simpler and more conservative method than the unit hydrograph-based rainfall-runoff approach was used to determine the PMF flood elevation in the reservoir. The PMF flood stage was calculated by adding the volume associated with the direct 72-hour PMP depth over the reservoir area (10.6 square miles) and the surface runoff volume of total PMP depth less 0.06 inch/hour loss rate from the remaining watershed area (6.8 square miles) of Frees Creek. This volume was then added to the full-pool volume of 397,000 acre-feet to yield a total of 440,500 acre-feet. As indicated in Figure 2.4-205, the PMF still water elevation in the Monticello Reservoir is estimated to be 431 feet NGVD29. It should be noted that the normal evaluation of an antecedent flood of 40% of the PMF occurring three days before the PMF on Monticello Reservoir will not affect the PMF peak water surface in the reservoir. The antecedent event will comprise a volume of some 17,500 acre-feet. The hydraulic capacity of the Fairfield Pumped Storage Facility is 29,000 acre-feet/day, so the entire volume of the antecedent event will be discharged within a day of the storm, and before the PMF.

2.4.3.5 Wind-Generated Wave Setup in the Monticello Reservoir

In accordance with procedures outlined in Subsection 9.2.1.1 of ANSI/ANS 2.8-1992, the wave setup and run-up generated by a two-year return period wind speed was added to the PMF still water elevation of 431 feet NGVD29 to determine the maximum PMF water elevation at the Monticello Reservoir. The two-year overland wind speed for the site was obtained from data presented in ANS/ANSI 2.8-1992 and NUREG/CR-2639 (Reference 248). From these two references, a fastest-mile two-year wind speed of 50 mph, measured 30 feet above the ground surface over land, was selected as the design wind speed.

The design wind speed was then adjusted for height, wind duration, wind speed over water, and fetch length and was used to calculate the site-specific significant wave height and significant peak spectral period, based on the procedures given in the USACE Coastal Engineering Manual (CEM) (Reference 232). The effective fetch length was estimated to be 15,820 feet from the fetch diagram shown in Figure 2.4-215 (Reference 226).

Using these values and the procedures outlined in ANSI/ANS 2.8-1992 and the CEM, the maximum wave height in the reservoir was calculated to be 5.16 feet and the wave run-up 6.68 feet. The run-up depends on the slope of the shoreline where the waves are breaking. For this analysis based on information provided in the UFSAR for Unit 1, it was conservatively assumed that the slope of the Monticello Reservoir shoreline is quite steep, equal to 2:1 (horizontal to vertical). This applies to the riprap dike that was constructed to protect the Unit 1 site; it does not apply to the much milder natural slopes around the remainder of the reservoir.

Using the calculation procedures described in USACE Design Guideline EM 1110-2-1420 (Reference 234), a wind setup of 0.17 feet was calculated for the reservoir site. Adding the wind setup and wave run-up values to the PMF still water elevation for the reservoir (431 feet NGVD29) resulted in a maximum PMF elevation of 437.85 feet NGVD29 for the Monticello Reservoir. The site of Units 2 and 3 is not located within the Frees Creek/Monticello Reservoir watershed. For the site to be impacted by flooding on the Monticello Reservoir, the water level would have to exceed El. 438 feet NGVD29, which is the crest of the dike (north berm) along the shoreline of the Monticello Reservoir north of Unit 1 (Reference 226). The dike crest elevation is 0.15 feet higher than maximum estimated water level in the Monticello Reservoir because of the combined effect of the PMF and wind setup and wave run-up. As can be seen in Figure 2.4-201, the Unit 1 site is between the Units 2 and 3 site and the Monticello Reservoir. Therefore, the topography of the site will protect Units 2 and 3.

2.4.4 POTENTIAL DAM FAILURES

Units 2 and 3 are located on a hilltop within about a mile of Parr Reservoir and two miles upstream from Parr Shoals Dam. The design plant grade for all safety-related facilities is established at El. 400 feet NAVD88. This is approximately 135 feet above the elevation of the top of the spillway gates at Parr

Shoals Dam. Thus, the site is not expected to be affected by flood waves from dam failures upstream. Since the site is relatively high and close to Parr Shoals Dam, a simplified approach was used to estimate a very conservative flood elevation that will not be exceeded as a result of any potential dam failure events.

2.4.4.1 Dam Failure Permutations

Both potential site flooding and low water conditions related to a dam failure event have been evaluated. Low water conditions from a failure of the Frees Creek dams or the Parr Shoals Dam will not present a safety hazard to Units 2 and 3 because the water supplies from Parr Reservoir and Monticello Reservoir are not required to support safety-related functions. Water supply for the safety-related passive containment cooling system is stored independently in tanks on the site (two tanks for each unit). The passive containment cooling water storage tank is seismically Category I designed and missile-protected. The passive containment cooling ancillary water storage tank is designed in accordance with criteria for seismic Category II building structures, Category 5 hurricanes including the effects of sustained winds, maximum gusts, and associated wind-borne missiles. Thus, there is no site flooding hazard from these tanks. Other onsite water storage tanks are addressed in Subsection 2.4.10.

ANSI/ANS 2.8-1992 requires that the maximum water level associated with potential dam failures be determined based on the higher of the following two alternative combinations:

- Alternative 1: Dam failure caused by the safe shutdown earthquake coincident with the 25-year peak flood plus the wind wave actions resulting from the two-year wind speed applied in the critical wind direction.
- Alternative 2: Dam failure caused by the operating basis earthquake coincident with one-half of the PMF or the 500-year flood (whichever is less) plus the wind wave actions resulting from the two-year wind speed applied in the critical wind direction.

The simplified and conservative approach adopted for determining the maximum water level at the Parr Shoals Dam associated with the potential failures of the significant upstream dams consists of the following calculation steps:

1. Assume at Parr Shoals Dam there is a “hypothetical vertical wall” that can hold all the water stored in all upstream reservoirs on the Broad River, including both existing and proposed reservoirs. Since the proposed Clinchfield Reservoir, with a suggested storage capacity of 1,275,000 acre-feet, is two orders of magnitude larger than the largest of the other reservoirs in the basin (except Monticello, which is off-stream, and is dealt with separately), it dominates any dam failure scenario. Thus, the most severe credible event would involve the failure of Clinchfield and all the dams between it and Parr Shoals Dam on the Broad River. Dams on tributaries are not considered since their failure would have a negligible effect because of their very small storage volumes. The exception to this is

the Monticello Reservoir. Since it is located adjacent to Unit 1, and it is about 100 miles downstream from Clinchfield, should it fail because of the same seismic event that precipitated the failure of Clinchfield, the two flood waves would arrive at Parr Shoals Dam at very different times. They would, therefore, not be directly additive, but would constitute separate events.

2. Determine the combined storage volume of all the existing and proposed upstream reservoirs in Step 1. This implies that all upstream dams on the Broad River fail and their entire storage is transported instantaneously to the Parr Shoals Dam site, where it is held. This assumption provides an upper bound estimate of the water level at Parr Shoals Dam.

3. Add the reservoir volume associated with the peak PMF discharge at Parr Shoals Dam (see Subsection 2.4.3) to the volume of all the existing and proposed upstream reservoirs to yield a volume associated with the maximum estimated water surface elevation behind the hypothetical vertical wall in Step 2. This water surface elevation provides the most conservative estimate for the still water level in the hypothetical reservoir during a PMF event.

4. Determine the wind wave run-up and setup resulting from the two-year wind speed and add that to the estimated water surface elevation in Step 3 to get the maximum water surface elevation behind the hypothetical vertical wall.

5. Compare the maximum water surface elevation estimated in Step 4 against the design plant grade elevation of 400 feet NAVD88 for the safety-related facilities of Units 2 and 3.

2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

The simplified and conservative approach used in the analysis is based on the most severe credible dam failure sequence, which is a cascading, or domino-type failure of Clinchfield and all the dams on the Broad River between it and Parr Shoals Dam, adjacent to the VCSNS site. The water level estimated at Parr Shoals is based on all stored water from the upstream dams being transferred instantaneously to Parr Shoals without any attenuation, and without any storage outflow from Parr Shoals. The water level resulting is superimposed on the level associated with the PMF peak outflow at the dam and wave run-up and wind setup are added as well. This approach will yield an upper bound on the water level that can be achieved in Parr Reservoir under the most severe imaginable condition. Unsteady routing of flood waves in the river will yield much lower levels.

Figure 2.4-208 shows the general location of the existing reservoirs at Houser Lake, Gaston Shoals, Cherokee Fall, Ninety-Nine Islands, Daves Pond, Una S. Johnson, Lockhart, Neal Shoals, and Parr Shoals and the proposed Clinchfield Reservoir on the Broad River. The existing reservoirs, with the exception of the

Parr Reservoir, are small low head dams that were constructed between late 1800s and early 1900s, primarily for water supply and hydropower generation.

By far, the largest upstream reservoir on the Broad River is that of the proposed Clinchfield Dam. A report for the development of this dam was prepared by the USACE in 1969 (Reference 240). The UFSAR for Unit 1 of October 2005 reported this to be the most recent report on the Clinchfield Project, indicating that this project had not advanced from the early feasibility study level.

A search of other relevant sources did not reveal any references to plans for the construction of Clinchfield Dam. Although there are no plans for its construction, Clinchfield Dam is considered in the safety analysis for Units 2 and 3 to account for the unlikely event that such plans are revived in the future. The storage capacities for the reservoirs on the Broad River are summarized in Table 2.4-211.

2.4.4.3 Maximum Water Level at Parr Shoals Dam

The maximum water level at the project site (*i.e,* at Parr Shoals Dam) associated with the potential failure of the upstream dams during a PMF event was estimated using the simplified approach discussed in Subsections 2.4.4.1 and 2.4.4.2 as follows:

1. The total volume of all upstream reservoirs plus the Parr Reservoir was estimated from Table 2.4-211 to be equal to 1,318,500 acre-feet. In the approach described in Subsection 2.4.4.1, it is assumed that this entire volume can be held behind a hypothetical vertical wall at Parr Shoals Dam.

2. Figure 2.4-216 shows elevation-storage curves developed for the hypothetical reservoir considered at Parr Shoals Dam site (Reference 230). Two storage capacity curves are shown in this figure, one curve for the storage volume of the “main river only” and the other for the storage volume of “main river plus branches.” The elevation-storage curve for the “main river only” was used in this analysis because it gives the more conservative estimate for the water surface elevation behind the hypothetical “vertical wall.”

3. The peak floodwater surface elevation of 291.5 feet NGVD29 was conservatively calculated for the peak PMF discharge at Parr Shoals Dam. This level is equivalent to 210,000 acre-feet of water in Parr Reservoir as shown in Figure 2.4-216. This volume was added to the volume of 1,318,500 acre-feet from the dam failures to yield a total of 1,528,500 acre-feet behind the hypothetical vertical wall. The still water surface elevation equivalent to this volume was estimated from Figure 2.4-216 to be El. 362 feet NGVD29.

4. The wave run-up and setup for the hypothetical reservoir at Parr Shoals Dam were determined for a two-year wind speed by following the procedures outlined in the USACE CEM. Using the two-year fastest mile

wind speed of 50 mph (Reference 201) and an estimated fetch length of 17 miles (see Figure 2.4-217) for the reservoir formed behind the hypothetical vertical wall at Parr Shoals Dam, the wave run-up associated with the maximum calculated significant wave plus setup was calculated equal to 17.6 feet. Because of the hypothetical and very conservative nature of this analysis, it is not considered necessary to distinguish between significant and maximum wave heights. The maximum wave run-up plus setup value calculated above was then added to the still water surface elevation estimated in Step 3, to give a maximum water surface elevation of (362 feet +17.6 feet =) 379.6 feet NGVD29. This is equivalent to El. 378.9 feet NAVD88.

Based on this analysis, during seismically induced potential upstream dam failures coincident with the PMF, the maximum water surface elevation in the Broad River (adjacent to the plant site) cannot be greater than 378.9 feet NAVD88. Because this water level is about 21 feet below the safety-related facilities design plant grade elevation of 400 feet NAVD88, it is concluded that the plant site is safe from seismically induced potential upstream dam failures, even during a PMF event.

The analysis described above shows that the plant site is dry based on the most conservative approach to flooding. It is clear that sedimentation in either the Broad River or Frees Creek can have no impact on this hilltop site, nor can landslides associated with the river system.

2.4.4.4 Maximum Water Level Due to Potential Failure of Fairfield Dam

A dambreak analysis for the Frees Creek Dams was performed in 1990 as part of the relicensing of the Fairfield Pumped Storage Facility by the Federal Energy Regulatory Commission. The results of this analysis were presented for the area downstream of the dam (Reference 223). The analysis shows that in the event of failure of Frees Creek Dams, the maximum water level at the nearest inundated point to Units 2 and 3 is about 310 feet NGVD29, *i.e.,* well below the design plant grade level for safety-related facilities of 400 feet NAVD88.

2.4.5 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

The Units 2 and 3 site is located about one mile south of the Monticello Reservoir, as shown in Figure 2.4-201. As the figure indicates, the site is bounded on the north by the Unit 1 site, which is at an average plant grade elevation of 435 feet NAVD88. The Unit 1 site is bounded on the west by an unnamed creek, the upper reaches of which have thalweg elevations between 300 feet and 360 feet, substantially lower than both Unit 1 and Units 2 and 3. Unit 1 is bounded on the southeast by the upper reaches of Mayo Creek, which is also between El. 300 feet and 360 feet. The Units 2 and 3 site is bounded on the west by another unnamed creek, the thalweg elevations of the upper reaches of which are also between El. 300 feet and 360 feet NAVD88 (40 to 100 feet below the Units 2 and 3 design plant grade level of 400 feet NAVD88). On the east, the site is also bounded by the upper reaches of Mayo Creek, the thalweg of which is also between 40 and

100 feet below design plant grade. Both the unnamed creeks and Mayo Creek drain into the Parr Reservoir on the Broad River. All three creeks have large flood flow capacity to convey potential overtopping flow, if any, from the Monticello Reservoir. This is evident from the site topography that shows that the channel slopes of these creeks are on the order of 1% and steeper, and the cross-sections of the creeks open up to a width of more than 500 feet at El. 360 feet NAVD88 near and downstream of the Units 2 and 3 site. Further, backwater effect from the Parr Reservoir causing flooding at the site is not expected because the Parr Reservoir has a normal full pool elevation of 266 feet NGVD29, or 265.3 feet NAVD88. Thus, although the Monticello Reservoir is the only water body in the vicinity of the Units 2 and 3 site that could be a source of surge or seiche flooding, it does not present a risk to the Units 2 and 3 site.

It is clear from the topography of the area and site as shown in Figures 2.4-201 and 2.4-203 that there is no route by which any waters from Monticello Reservoir could reach Units 2 and 3. Moreover, because the plant site is located nearly 150 miles from the nearest coast, the site is not subject to any coastal surge and seiche flooding. It is concluded that the site is not at risk from surge or seiche flooding from any source. As described in Subsection 2.4.11, the Monticello Reservoir provides no safety-related water supplies; therefore, there is no safety impact to Units 2 and 3 from low water due to surging or seiching.

2.4.6 PROBABLE MAXIMUM TSUNAMI HAZARDS

The Units 2 and 3 site is located about 150 miles from the nearest seacoast (that of South Carolina), and is situated such that the design plant grade for safety-related facilities is at El. 400 feet NAVD88. This elevation corresponds to approximately 400 feet above MSL on the South Carolina coast. Thus, the site is not subject to any effects from tsunami events anywhere.

2.4.7 ICE EFFECTS

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The ice effects on the Broad River and Monticello Reservoir are evaluated because they are an integral part of the water supply system that provides makeup water to Units 2 and 3. The analysis of ice effects was performed based on water temperature data collected from the Broad River and the Monticello Reservoir and on air temperature data recorded at the Parr climate station. The climate in the vicinity of the Units 2 and 3 site is temperate (Reference 227). There are no records of ice jams on the Broad River based on a search of the “Ice Jam Database” of the USACE (Reference 233).

Water temperature data from the Broad River recorded on different occasions at the Carlisle, Alston, and Richtex stations (Reference 229) between October 1959 and December 1975 was used to evaluate the water temperatures in the river close to the VCSNS site. The minimum recorded water temperature at these stations was 38.3°F.

In general, surface water temperatures in the Monticello Reservoir are slightly higher than those in the Broad River because of the effect of waste heat discharge from the cooling water system of Unit 1. A review of five years (August 2001 through August 2006) of hourly water temperature data collected in the Monticello Reservoir near the intake of the Fairfield Pumped Storage Facility suggests that the minimum recorded surface water temperature in the reservoir was 37.6°F on January 26, 2003. The minimum daily average water temperature for the same day, however, was estimated to be 43.4°F.

Occasionally, the air temperature at the plant site falls below the freezing point, which may cause the formation of thin ice in any ponding water around the site. However, such low temperatures do not persist over long periods of time. Daily air temperature data from the Parr climate station (Reference 220) for the period from 1949 to 2001 showed a maximum Accumulated Degree Day Freezing of 42.5°F/day. This low magnitude of Accumulated Degree Day Freezing along with high heat load suggests that ice formation in the Monticello Reservoir is unlikely. In the unlikely event that thin ice forms at the surface of the Monticello Reservoir, it would not affect the water supply at Units 2 and 3 intake, which is located approximately 12.8 feet (3.1 meters) below the lowest operating reservoir water surface elevation. Moreover, the intake water supplies to Units 2 and 3 are not safety-related.

The measured temperatures in the Broad River and Monticello Reservoir suggest that the water temperature never approaches the freezing point. Therefore, the formation of frazil or anchor ice is considered highly unlikely.

If thin ice were to be formed in the plant site drainage system, it may cause blockage of the site drainage systems. The effect of flooding on safety-related structures because of a postulated occurrence of a local PMP storm event coincidental with the blockage of the site drainage system is addressed in Subsection 2.4.2.3.

2.4.8 COOLING WATER CANALS AND RESERVOIRS

The safety-related water supply for the passive containment cooling system used in the AP1000 reactors at Units 2 and 3 is stored in tanks on the site as described in Subsection 6.2.2 of the DCD. The tanks are missile-protected, and the primary one is seismic Category I, while the auxiliary tank is Category II. No cooling water reservoirs or canals are used to supply water for safety-related cooling at Units 2 and 3.

Units 2 and 3 use the Monticello Reservoir as a source of makeup water for normal plant cooling and other nonsafety-related uses as described in Sections 9.2 and 10.4. The maximum rate of makeup water withdrawal for the two units is 61,600 gpm or 272 acre-feet per day. Subsection 2.4.1.2.2 contains a more complete description of the Monticello Reservoir.

2.4.9 CHANNEL DIVERSIONS

The Units 2 and 3 site is located within the Piedmont Physiographic Province of central South Carolina. The Piedmont Physiographic Province is discussed in Subsection 2.5.1. As stated in Subsection 2.5.1.2.1, ". . . the site topography is characteristic of the region consisting of gently to moderately rolling hills and generally well-drained mature valleys. Within the vicinity of the site, topography ranges from about 520 to 220 feet MSL. All local tributaries drain into the Broad River. The local drainage pattern is generally dendritic; however, a subtle trellis pattern is also evident and probably a result of regional bedrock structure and joint systems. Steep gullies exist within the site area resulting from differential weathering of the basement rock and possible exacerbation by previous agricultural activity." This section discusses the potential for channel diversion into the Monticello Reservoir and the Broad River. Historical channel diversions, regional topography, local geology, local seismicity, ice effects, and human activity are considered to assess this potential.

The Monticello Reservoir is a pumped storage reservoir. It is the upper pool of the Fairfield Pumped Storage Facility. The Parr Reservoir constitutes the lower pool of the Fairfield Pumped Storage Facility, and is located on the Broad River. The Monticello Reservoir itself is not subject to upstream diversions because the reservoir occupies the majority of the original Frees Creek tributary drainage basin, which has a small drainage area of about 17.4 square miles (Reference 226). Because a relatively large fraction of the drainage basin is impounded, there are no streams of significance draining into Monticello Reservoir that could be subject to upstream channel diversion. Consequently, issues related to upstream channel diversion are not applicable to the Monticello Reservoir.

The Broad River, the principal hydrologic feature in the site vicinity, drains an area of about 4,750 square miles upstream of the site. The drainage area is located between two southeast-northwest trending ridges stretching from Columbia, South Carolina, to the headwaters of the Broad River about 100 miles northwest in North Carolina. To identify significant changes in the course of the Broad River over the past two centuries, the USGS GIS digital elevation maps files of the topography (Reference 230) (Figure 2.4-218) were compared with an 1838 map by Bradford (Reference 205) (Figure 2.4-219) and a 1773 map by Cook (Reference 208) (Figure 2.4-220). Although these maps are quite old and may not be as accurate as current-day USGS topographic maps, they do indicate that there have not been any significant channel diversions over the last two hundred years. These maps show the shape of the riverbank line. Geomorphic features such as oxbow lakes, meander cutoffs, abandoned meanders, and paleolandslide features that would indicate an actively changing river form are not present. It is therefore unlikely that the Broad River at the VCSNS site will be diverted from the lower pool of the Fairfield Pumped Storage Facility by natural causes.

An examination of the regional geology in the four geologic quadrangles surrounding the Units 2 and 3 site indicates that the Broad River is incised into various types of metamorphic rocks deformed during Precambrian and Paleozoic mountain-building events (Subsection 2.5.1). The site region has also been intruded by late Paleozoic (Carboniferous) igneous plutons like the Winnsboro Plutonic complex. The Units 2 and 3 site is located within this granitic terrane (Subsection 2.5.2.2). The geology of the four geologic quadrangles surrounding the Units 2 and 3 site is shown in Figures 2.5.1-224 and 2.5.1-225. The detail shown on the quadrangle maps indicates that the geologic fabric (foliations, cleavage, compositional layering and faults) mapped in metamorphic terrane strike in a northeast-southwest direction and generally dip toward the northwest or southeast (Figure 2.5.1-225). These structural geologic features represent potential planes of weakness along which rock slides or landslide could potentially occur. However, the Broad River generally flows in a northwest to southeast or north to south direction in the site vicinity (Figures 2.4-218, Figures 2.5.1-224 and 2.5.1-225). Since the river channel is predominantly perpendicular to, rather than parallel to, the dip slopes of potential failure planes, there is little likelihood for a rock mass failure that would result in a diversion of the Broad River. Subsection 2.5.1 provides a detailed discussion on erosion along ancient river channels based on geomorphic evidence such as uplift and subsidence.

No large-scale geologic fabric (potential slip planes) is mapped within the Silurian Newberry granitic complex and the Carboniferous Winnsboro granitic complex (Figures 2.5.1-224 and 2.5.1-225). However, northwest-trending Mesozoic diabase dikes have been mapped in the site vicinity (Figure 2.5.1-225). It is highly likely that northwest-trending fractures also occur in the site region. Since the channel slopes along the Broad River appear to have a relatively gentle gradient and well-developed alluvial deposits, and since the drainage is predominantly dendritic, there is little likelihood that slope failure could occur along northwest-trending joints or fractures that would result in channel diversion of the Broad River.

Impoundment of water within the Monticello Reservoir has resulted in minor reservoir-induced seismicity as discussed in Subsection 2.5.2. The reservoir-induced seismicity represents small, shallow earthquakes associated with the filling of the Monticello Reservoir in 1977 and 1978. These seismic events were too small to cause surface faulting and did not result in slope failures that resulted in channel diversion of the Broad River. Subsection 2.5.3 discusses current aerial and field reconnaissance activities conducted around the VCSNS site. Twelve bedrock faults are mapped within 25 miles of the site. As stated in Subsection 2.5.3, no deformation or geomorphic features suggestive of potential Quaternary activity have been reported in the literature for these twelve faults. Aerial and field reconnaissance and interpretation of aerial photographs and satellite imagery performed shows that no geomorphic features indicative of Quaternary activity exist along any of the mapped fault traces. Based on this analysis of the seismicity of the area, there is not expected to be any danger of channel diversion due to earthquake-induced landslide activity.

Subsection 2.4.7 discusses ice effects at the VCSNS site. This subsection states that the minimum recorded water temperature in the Broad River is well above freezing. Occasionally, the air temperature at the plant site falls below the freezing point, which may cause the formation of thin ice in any ponding water around the site. However, such low temperatures do not persist over long periods of time. Daily air temperature data from the Parr climate station (Reference 220) for the period from 1949 to 2001 showed an Accumulated Freezing Degree Day of 42.5°F/day. This low magnitude of Accumulated Freezing Degree Day suggests that ice formation in the Broad River is unlikely. Therefore, ice-induced channel diversion is not considered a possibility for this location.

Figure 2.4-203 shows the general site features, layout, topography, and changes to the natural drainage. The Units 2 and 3 site are not susceptible to flooding from any possible channel diversions within the Broad River because of their relative height above the river. Thus, there is no need to consider channel diversion-induced forces on systems, structures, and components important to safety.

In addition to Parr and Monticello, a number of reservoirs currently exist upstream and downstream of the site on the Broad River and its tributaries, as described in Subsection 2.4.1. Most of these reservoirs are small, low-head dams for hydroelectric power generation and water supply. There is the possibility of future additional dams and reservoirs on the Broad River because several studies for the construction of major dams have been completed. The latest study (Reference 240), completed in 1969, reports that the only reasonably feasible location for a major dam in the Broad River watershed is at the Clinchfield site. As shown in Figure 2.4-208, the site of this proposed dam is located in the upper reaches of the Broad River basin in North Carolina, approximately 100 river miles upstream of the VCSNS. As discussed in Subsection 2.4.4, construction plans appear to have been abandoned; however, if this dam were to be constructed, it should not influence the channel location of the Broad River at the VCSNS site. The primary impact of this dam, if built, would be to reduce the peak discharges of major flood events. There are no other activities planned for the Broad River that could result in human-induced channel diversions.

The above analysis of historical channel diversions, regional topography, local geology, local seismicity, ice effects, and human-induced causes of channel diversion indicates that no channel diversion effects at Units 2 and 3 will adversely affect safety-related facilities or water supply. As a result, no alternative water sources or operating procedures are needed.

VCS COL 2.4-6

2.4.10 FLOODING PROTECTION REQUIREMENTS

The design basis for flood protection of safety-related facilities is the establishment of a design plant grade elevation (400 feet NAVD88) that is above the most severe potential flood level. Thus, no safety-related facilities will be subject to flooding hazards.

Subsections 2.4.3 and 2.4.4 discuss the calculation of the maximum flood elevation of Parr Reservoir due to the PMF and failure of upstream dams on the Broad River, respectively. The resulting maximum flood elevations are considerably lower than the plant grade elevation of 400 feet NAVD88. Also, it is shown in Subsection 2.4.3.5 that under maximum wind setup and wave run-up in the Monticello Reservoir, there is no risk of flooding of the site of Units 2 and 3 because of the existing flooding protection measures for Unit 1, which were installed between the site of Units 2 and 3 and the Monticello Reservoir. Also, due to the presence of the creeks to the east and west of the site, there is no path by which flow from the Monticello Reservoir may reach Units 2 and 3, as described in Subsection 2.4.1.1.

The impacts of local intense precipitation on the plant site are addressed in Subsection 2.4.2.3. The site drainage analysis presented in Subsection 2.4.2.3 shows that even if the subsurface drainage system were completely blocked during a local PMP event, the maximum flood level at the site of Units 2 and 3 would still be below the design plant grade elevation for safety-related facilities of 400 feet NAVD88. Therefore, no special flood protection measures are required to prevent flooding of safety-related facilities from local intense precipitation.

The roofs of safety-related buildings are designed to preclude accumulating of water. The roofs are sloped such that rainfall is directed towards gutters located along the edges of the roofs. Therefore, ponding of water on the roofs is precluded (see DCD Subsection 3.4.1.1.1).

There are seven water storage tanks located in the vicinity of the central plant site. These are all described in Chapter 9 of the DCD. The largest of these tanks with a capacity of 485,000 gallons is assumed to fail and cause flooding. Based on a conservatively estimated peak outflow of 1,200 cfs (associated with one minute to drain the largest tank) and a flow width of 200 feet (the effective flow section at the location of the nearest safety-related structure—the containment structure) the maximum possible flow depth is 1.0 foot (critical depth). The grade elevation outside the containment structure is 398 feet NAVD88. Thus, it is not possible for the maximum flood level to exceed 399 feet NAVD88. Therefore, the design plant grade elevation of 400 feet NAVD88 is not reached.

It is concluded that the safety-related facilities of Units 2 and 3 are not subject to flooding. No flood protection measures, other than the one already installed for Unit 1, and no emergency procedures are required.

2.4.11 LOW WATER CONSIDERATIONS

VCS COL 2.4-3

2.4.11.1 Low Flow in Rivers and Streams

The safety-related water supply for the AP1000 reactor units at Units 2 and 3 is not dependent on either river flow or lake water level. The required emergency water supply for the plant is stored in two tanks for each unit on the site. Both tanks are missile-protected, and the primary one is classified as a seismic Category I structure, while the auxiliary tank is Category II. Thus, failure of any downstream dam will not produce any safety hazard due to the resulting low water in the Broad River. Similarly, failure of the Frees Creek Dams will not produce a hazard due to loss of Monticello Reservoir storage.

To demonstrate the adequacy of the water supply for nonsafety-related operational uses, a low flow frequency analysis was performed on annual daily-mean flows estimated at the Parr Shoals Dam. This was accomplished by graphically plotting a best-fit curve through the annual minimum daily mean flows, which was extrapolated to obtain the 100-year daily mean low flow in the Broad River. This analysis showed that the 100-year daily mean low flow is about 125 cfs (see Figure 2.4-221).

A similar analysis was performed on annual minimum seven-day average flows estimated at the Parr Shoals Dam. This analysis shows that the 10-year, 7-day average low flow (7Q10) and the 100-year 7-day average low flow (7Q100) are about 850 cfs and 430 cfs, respectively (see Figure 2.4-222).

These flows, combined with the storage at the Monticello and Parr Reservoirs, are more than adequate to support the maximum Units 2 and 3 makeup demand of about 272 acre-feet per day (137.2 cfs).

In the event the Fairfield Pumped Storage Facility is unavailable and there is no inflow of water to the Monticello Reservoir from direct precipitation or runoff, evaporation and withdrawal of makeup water for Units 2 and 3 will reduce the storage of the Monticello Reservoir by 390 acre-feet per day. This is based on an evaporation loss of 104 acre-feet per day when the reservoir is at El. 420.5 feet NGVD29. To obtain a conservative estimate of the available water volume it is assumed that the unavailability of the Fairfield Pumped Storage Facility starts when the Monticello Reservoir is at its lowest normal operation level. The water surface of Monticello Reservoir at El. 420.5 feet NGVD29 is about 95% of that at El. 425 feet NGVD29 (see Figure 2.4-205). The maximum water requirements for Units 2 and 3 are 272 acre-feet per day. This brings the total loss from the Monticello Reservoir at 104+272=376 acre-feet. Considering that the volume of the reservoir between El. 419.8 feet and 417.3 feet NAVD88 (420.5 feet and 418 feet NGVD29) is 16,000 acre-feet (see Figure 2.4-205), lowering the reservoir water level by 2.5 feet (from 419.8 feet to 417.3 feet NAVD88) would meet the makeup water needs of Units 2 and 3 for about 42 days.

2.4.11.2 Low Flow Resulting from Surges, Seiches, or Tsunami

The effect of surges, seiches, and tsunami are not applicable to this site as discussed in Subsections 2.4.5 and 2.4.6. As described in Subsection 2.4.7, no ice conditions are expected to affect flows in either the Broad River or the Monticello Reservoir. Low water due to ice conditions will not affect the safety-related facilities of the plant in any event, since no external water source is required.

2.4.11.3 Historical Low Water

Information on historic low flows is available at the Richtex (October 1925 to July 1928, October 1929 to September 1983) and Alston (October 1896 to December 1907, October 1980 to September 1984, October 1996 to September 2006) gauging stations. The lowest observed daily mean flow at Richtex was 149 cfs on October 13, 1935 and on September 2, 1957. The lowest daily mean flow at Alston was 48 cfs on September 12, 2002. However, this value is not considered representative of natural river flows because it was influenced by the upstream flow diversion from the Parr Reservoir to the Fairfield Pumped Storage Facility. Therefore, this value was not included in the low flow analysis. The next lowest flow at Alston was 156 cfs on August 13, 2002.

2.4.11.4 Future Controls

No future uses and/or controls for the Monticello Reservoir are planned. The Monticello Reservoir is not used for any safety-related facilities at the plant in any event. No future plans for control of the Broad River above Parr Shoals Dam that would affect the safety-related facilities at the plant are known to exist.

2.4.11.5 Plant Requirements

The AP1000 reactor's passive safety system does not require a water supply for safety-related use apart from the water stored in the primary and auxiliary tanks located on site. Each AP1000 unit has a nonsafety-related circulating water system to dissipate plant waste heat during normal plant operation. It also has a nonsafety-related service water system to provide cooling water to the component cooling water heat exchangers. Both the circulating water system and service water system use closed-cycle cooling system with mechanical draft cooling towers. Makeup water to the circulating water system and service water system is supplied from the Monticello Reservoir at a maximum flow rate of about 59,000 gpm (131.5 cfs) and 1840 gpm (4.1 cfs), respectively.

The intake system for the raw water system consists of the intake approach channel, the intake structure, the raw water pumps, and the support systems. Section 9.2 provides a description of the raw water system. The invert of the pump sump is located at El. 400 feet NAVD88 (400.7 feet NGVD29) with a design minimum reservoir water level of El. 414.3 feet NAVD88 (El. 415 feet NGVD29).

The Fairfield Pumped Storage Facility is licensed by the Federal Energy Regulatory Commission. It is assumed that the Federal Energy Regulatory Commission Monticello Reservoir operating limits for Unit 1 also apply to the operation of Units 2 and 3.

2.4.11.6 Heat Sink Dependability Requirements

The ultimate heat sink for the AP1000 system is the atmosphere. Heat dissipation is aided by evaporative cooling from the passive system of water flow over the containment structure sourced from the primary emergency tank located in the structure above the containment. No water from the Parr or Monticello Reservoirs or from other outside sources is required for safe emergency shutdown.

VCS COL 2.4-4

2.4.12 GROUNDWATER

2.4.12.1 Regional and Local Hydrogeology

2.4.12.1.1 Regional Hydrogeology

The region within 200 miles around the Units 2 and 3 site encompasses parts of four physiographic provinces. These include, from west to east, the Valley and Ridge, Blue Ridge, Piedmont, and Coastal Plain physiographic provinces. These provinces are defined on the basis of physical geography and geology. The provinces are described in more detail in Subsection 2.5.1.1 and are illustrated in Figure 2.5.1-201. Figure 2.4-223 shows the aquifer systems associated with these provinces. Figure 2.4-224 is a cross section view of these provinces. Although Figure 2.4-224 includes the Appalachian Plateau province, groundwater conditions in this province will not be addressed because of its distance from, and lack of influence on, the site. This figure shows a sharp change in topographic slope that defines the boundary between the Blue Ridge and Piedmont Provinces. These provinces, however, exhibit essentially the same aquifer system characteristics and are considered together in the description provided below. Groundwater occurrence is of significance to the Units 2 and 3 site only within the Piedmont physiographic province. However, brief discussions of groundwater within the other provinces within 200 miles of the site are presented below to provide a more complete picture of regional hydrogeologic conditions.

2.4.12.1.1.1 The Valley and Ridge Aquifer System

The Valley and Ridge aquifer system lies within the Valley and Ridge physiographic province about 190 miles west of the site (Figure 2.4-223). The aquifer is composed of Paleozoic age folded and faulted sedimentary rock.

Carbonate and sandstone layers form the principal aquifers in the system. The carbonate rocks, mainly limestone, generally form most of the more productive aquifers and underlie valleys within the province. Most of the groundwater flow is in the fractures and dissolution features in the folded and faulted strata. Typical well yields are from 10 gpm in sandstone formations to 10 to 50 gpm within the limestone units. Locally high yields are possible within highly fractured strata or solution cavities (Reference 219).

2.4.12.1.1.2 Piedmont and Blue Ridge Aquifer System

The Piedmont and Blue Ridge physiographic provinces exhibit essentially the same aquifer system characteristics. The aquifer system associated with these provinces is combined and referred to as the Piedmont and Blue Ridge aquifer system. This system lies beneath the site and to the north and west of the site.

Both provinces are composed of metamorphic rocks with igneous intrusions and with alluvial deposits along stream valleys. The Piedmont Province is characterized by saprolite or residual soil that overlies the crystalline bedrock. The crystalline rock of the Blue Ridge Province is generally overlain by thinner deposits of residual soil. In general, groundwater occurs in the fractured portions of the bedrock, within the saprolite and alluvium, and at the saprolite/crystalline rock contact. Well yields are generally low within this aquifer system (6 to 28 gpm) and mainly depend on the local fracture density of the bedrock. Localized large yielding wells are possible and are dependent upon the geologic unit present and the surrounding geologic structure. Locally, where the crystalline rocks consist of marble, and where the dissolution action of acidic groundwater has resulted in solution openings, the rocks yield relatively large volumes of water (Reference 219).

2.4.12.1.1.3 Southeastern Coastal Plain Aquifer System

The Southeastern Coastal Plain aquifer system is the aquifer system associated with the Coastal Plain physiographic province (sometimes referred to as the Atlantic Coastal Plain physiographic province). This province lies approximately 15 miles south and east of the site. The divide between the Piedmont and Coastal Plain physiographic provinces is defined as the Fall Line. The Coastal Plain province is further divided into an Upper and Lower Coastal Plain, as shown on Figure 2.4-223. The geology of the Coastal Plain province is characterized by aquifers developed in layers of sands, silts, or high-permeability limestone confined by units of clay and silts or low-permeability limestone (Reference 207).

Most of South Carolina’s groundwater resources are from the Coastal Plain. In general, reliance on groundwater for irrigation, industrial uses, and public water supply increases dramatically east of the Fall Line (Figure 2.4-223) (Reference 207).

Within South Carolina, the aquifers that make up the Southeastern Coastal Plain aquifer system include: the Surficial Aquifer, Tertiary Sand/Limestone Aquifer, the Black Mingo Aquifer, the Black Creek Aquifer, the Middendorf Aquifer, and the Cape Fear Aquifer as indicated in Figure 2.4-225 (Reference 223).

2.4.12.1.2 Local Hydrogeology

The area within 6 miles of the site lies within the Piedmont and Blue Ridge Aquifer system within the Piedmont physiographic province as shown in Figure 2.4-226. The bedrock underlying the site area principally consists of Paleozoic crystalline metamorphic and igneous intrusives of the Carolina Zone as discussed in Subsection 2.5.1.2.2 and shown in Figure 2.5.1-220.

The metamorphic and igneous rocks weather to regolith consisting of residual (saprolitic) soils of clayey, silty, and sandy composition. The character of the overburden is related to the type of bedrock and degree of weathering. The overburden can be as thick as 100 feet or more, but this thickness varies considerably from place to place (Reference 219).

Groundwater in the site area occurs in two types of formations: (1) jointed and fractured crystalline bedrock, and (2) lower zones in the residual soil overburden as shown in Figure 2.4-227. Recharge to these formations is principally by infiltration of precipitation falling on the upland areas as shown in Figure 2.4-227. Some of the water infiltrating the surface soil evaporates, transpires from plants, or reemerges at the surface downslope at short distances from points of infiltration. A small portion of the water percolates to perched water zones or deeper into the water table in the lower soils and the underlying jointed bedrock. The groundwater table, in general, is parallel to the land surface, but with more subdued relief. Groundwater discharges as visible seeps and springs and/or percolates through the ground into creeks and streams. Some groundwater is discharged via wells, but the amount pumped is very small because the formations generally are not transmissive enough to sustain well yields greater than a few gallons per minute.

2.4.12.2 Groundwater Sources and Use

2.4.12.2.1 Groundwater Sources

Groundwater sources of water supply are found throughout South Carolina in varying quantities, qualities, and depths that reflect the nature of the geologic materials that host the respective aquifers. Within 6 miles of the Units 2 and 3 site, these sources reflect a small portion of the Piedmont aquifer. Figure 2.4-225 presents a hydrogeologic cross section of South Carolina indicating the primary sources of groundwater within the state.

2.4.12.2.1.1 Piedmont Aquifer

The geology of the Piedmont province is typically characterized by a weathering profile of metamorphic crystalline bedrock. The weathering profile generally consists of clayey saprolite, ranging in depth from several feet to several tens of feet, overlying bedrock. The saprolite grades downward through a highly permeable transition zone of weathered rock to unaltered parent bedrock.

Groundwater conditions in the bedrock are dependent on the number of fractures and degree of interconnection of the fracture systems. Groundwater moves slowly through the saprolite and discharges to surface water bodies, wells, or is released from storage to the underlying bedrock. In general, wells in the Piedmont region yield little water when compared to wells drilled in the Coastal Plain owing to the inherently low porosity and hydraulic conductivity of the crystalline rock (Reference 207).

2.4.12.2.1.2 Variations in Groundwater Availability

Groundwater supplies are subject to seasonal variation in water levels and decline due to prolonged drought, but usually to a lesser degree than surface water supplies. Groundwater levels lowered during the summer and fall because of both increased pumping and reduced recharge, usually recover during the winter and spring because of increased aquifer recharge and reduced pumping.

Figure 2.4-228 is a hydrograph that shows typical seasonal variations in groundwater level within the Piedmont formation (Reference 204). Multiyear droughts lower aquifer water levels by limiting the recharge that normally occurs during the wet winter and spring months. Figure 2.4-229 is a hydrograph of a Piedmont Aquifer well in Greenville County. This hydrograph illustrates the effect of prolonged drought on groundwater levels within Piedmont Aquifer wells.

Units 2 and 3 are located within the Piedmont aquifer system. Water levels within the Piedmont aquifer wells on the site can be expected to exhibit similar seasonal variations in water levels.

In addition to seasonal variations in the water supply, long-term variations in the climate can, over time, affect the water supply. Climate changes affect precipitation, temperature, and evapotranspiration, gradually changing the "normal" values. Normal amount of precipitation is defined in Reference 204 as the average annual precipitation for the previous 30 years. Using this definition, the "normal" value will change as the climate changes. Figure 2.4-230 illustrates how the normal rainfall amounts in South Carolina have changed during the 20th century. Over the past 50 years, there has been a trend toward increasing precipitation; a normal amount of rain in the 1990s, for example, would have been a greater-than-normal amount in the 1950s (Reference 204).

2.4.12.2.2 Groundwater Use

Total water use for South Carolina reported for 2005 was about 20.5 trillion gallons from 862 reporting facilities. Surface water withdrawal from 481 facilities accounted for approximately 20.4 trillion gallons, approximately 99.6% of total water use. Groundwater withdrawal from 541 reporting facilities accounted for approximately 72.1 billion gallons or approximately 0.4% of total use (Reference 207). However, over 90% of the reported water use includes nonconsumptive use. When discounting water use for hydroelectric and thermoelectric water use categories, surface water accounts for approximately 82.7% and groundwater accounts for approximately 17.3% of consumptive use. Table 2.4-212 summarizes water use for South Carolina for 2005.

A 6-mile radius around the Units 2 and 3 site includes portions of Fairfield and Newberry Counties. The Units 2 and 3 site is located near the western edge of Fairfield County. The Newberry County line is approximately 1 mile to the west on the western side of the Broad River (Figure 2.4-226). Tables 2.4-213 and 2.4-214 summarize water use in each of these counties, respectively. Surface water is the primary water source in both of these counties. Groundwater accounts for 0.002% of the total water use in Fairfield County; however, when discounting use for nuclear power and hydroelectric, groundwater accounts for approximately 10.4% of consumptive water use within the county. Groundwater in Fairfield County is used for public water supply and provides 10.4% of the public water supply (See Table 2.4-213). In Newberry County, groundwater accounts for 4.1% of the total water use in the county of which 66.7% is used at golf courses, 31.1% of water used for irrigation, and 1.1% of water used for public water supply (See Table 2.4-214).

Table 2.4-215 lists public groundwater supply wells within 6 miles of the Units 2 and 3 site. Of the 16 listed public groundwater supply wells, nine are located within Newberry County, which is on the other side of the Parr Reservoir from the site. Of the remaining seven wells that are located within Fairfield County, three wells are Jenkinsville water supply wells, which are located on the other side of the Monticello Reservoir from the Units 2 and 3 site, three wells are located within recreation areas of the Monticello Reservoir on the opposite side of the reservoir from the Units 2 and 3 site, and one is located in the vicinity of Parr Hydro. There are no SCDHEC records of agricultural or industrial wells within 6 miles of the site.

Future regional groundwater use in the area surrounding the VCSNS site is expected to increase moderately, corresponding with the population predictions for Fairfield County. The Fairfield County population is predicted to rise to 27,280 by 2025, an approximate 12% increase over 2005 levels (Reference 253). This moderate increase in population suggests only a moderate increase in water demand over this period. Table 2.4-213 reports the 2005 groundwater use for Fairfield County as approximately 68 million gallons.

There are no plans to use local groundwater for construction or operation of VCSNS Units 2 and 3. Construction of Units 2 and 3 power blocks requires temporary dewatering of the power block area. The low hydraulic conductivity of the local formations and limited spatial extent of the power block suggests the amount of groundwater derived from construction dewatering will be small. The small scale and temporary nature of the construction dewatering activities indicate a small impact to the groundwater flow system.

Water for construction purposes will be obtained from the Monticello Reservoir and the Jenkinsville Water District. The Jenkinsville Water District can meet the projected VCSNS water demand via purchase agreements with the Midcounty Water District which has significant excess capacity. SCE&G plans to construct a water treatment facility, which draws from the Monticello Reservoir, to provide the plant with potable water in the future.

The nearest a water supply well could be located to the proposed facility is approximately 0.75 miles to the southeast. This relatively long distance coupled with the low well yields typical of the area (less than 30 gallons per minute) (see Subsection 2.4.12.1.1.2), suggests any impacts to the groundwater flow system would be negligible.

2.4.12.2.3 Groundwater Quality of the Broad River Basin

An ambient groundwater quality monitoring network has been established by SCDHEC for the purpose of obtaining statewide and aquifer-specific baseline values of groundwater quality. The Units 2 and 3 site is located at the southern end of the Broad River Basin (Figure 2.4-231). Ten wells are monitored by SCDHEC in the Broad River Basin within the Piedmont bedrock and saprolite. Figure 2.4-231 shows the locations of the ten wells. Wells AMB-57 and AMB-60 are the closest to the VCSNS site and are approximately 10 miles to the northeast at the northern end of the Monticello Reservoir in the town of Jenkinsville. These wells are also referred to as Jenkinsville #11 and Jenkinsville #4, respectively. Both of these wells are completed in fractured Piedmont bedrock. (Reference 206).

The Broad River ambient groundwater quality report (Reference 206) describes the water quality testing that was performed on samples from these wells. The groundwater quality is affected by the lithology of the bedrock, residence time of the groundwater, and influences from man-made sources of alteration or contamination. Because of the variable nature of the bedrock within the Piedmont, the groundwater composition is also variable; however, in general, groundwater in the Piedmont can be described as calcium carbonate-type water. The following paragraph from the Broad River Ambient Groundwater Quality Report (Reference 206) summarizes the results of the water quality testing:

"Analyses indicate that water samples from the 2004 ambient monitoring display similarity in composition and are suitable for most purposes with minor exceptions. Ambient wells AMB-109 and AMB-110 displayed levels of iron in excess of National Secondary Drinking Water Standard (0.3 ppm), though at levels that do not cause health concerns. The Secondary Standard was established for public water systems for aesthetic purposes, and is a guideline for water quality. Wells AMB-110 and AMB-67 exceed the Secondary Standard for manganese (0.05 ppm), and may cause staining of plumbing fixtures. Based on the total dissolved solids (TDS), sodium, calcium and magnesium concentrations, the water is suitable for most irrigation purposes and has a low-to-medium salinity hazard. Of all samples processed, AMB-110 from Chester County returned the highest TDS, alkalinity, hardness, and electrical conductivity of all 2004 samples. In addition, chloride, sulfate, total organic carbon (TOC), and some common metals were detected in greater abundance in this well than in any other well in the network. The water obtained from AMB-110 may be influenced by longer residence time in the host rocks, host rocks with a lithology distinctive from that of other wells in the network, or from man-made contaminants."

Based on these analyses, well pairs that penetrate the saprolite and bedrock indicate that the groundwater is similar in composition within both hydrostratigraphic zones.

In portions of the Broad River Basin, naturally occurring radionuclides have been detected in some private and public water supplies, particularly in southeast Greenville and southern Spartanburg Counties, which are located northwest of the site (Figure 2.4-231). The highest concentration of dissolved uranium in groundwater occurs in the Simpsonville area near the hydrologic divide between the Broad and Saluda River watersheds, about 60 miles northwest of the site (Figure 2.4-232). The U.S. EPA funded a study conducted jointly by SCDHEC, the South Carolina Department of Natural Resources, and Clemson University (Reference 251). A 700-foot-deep well was drilled and core samples revealed that radionuclides were concentrated in uranium-carbonate minerals in bedrock fractures along the northern boundary of the Reedy River Fault System (Figure 2.4-232).

2.4.12.3 Site Hydrogeology

The hydrogeology of the Units 2 and 3 site is consistent with the hydrogeology of the Piedmont province. The hydrogeologic profile consists of two hydrogeologic zones. These zones are the saprolite/shallow bedrock hydrostratigraphic zone, which is primarily a water table aquifer, and the deep bedrock hydrostratigraphic zone, where groundwater occurs within fractures in the bedrock. Recharge to the saprolite/shallow bedrock zone occurs locally from surface infiltration. The aquifer then discharges locally into the drainage swales to the northwest and southwest of the site. The deep bedrock zone is recharged by infiltration from the saprolite/shallow bedrock zone. The deep bedrock zone flows westward from the site toward the Broad River. The Monticello Reservoir is located approximately 1 mile to the north of the Units 2 and 3 site.

2.4.12.3.1 Observation Well Installation and Testing Program

Thirty-one observation wells were installed at the Units 2 and 3 site as part of a geotechnical subsurface investigation program (Subsection 2.5.4) (Figures 2.4-233 and 2.4-234). The site investigation wells are screened in the saprolite/shallow bedrock zone or the deep bedrock zone. The wells are located to provide adequate distribution with which to determine site groundwater levels and subsurface flow directions and gradients beneath the site. Well pairs were installed to determine if the saprolite/shallow bedrock, and deep bedrock zones are hydraulically connected. Table 2.4-216 contains the well construction details for each well, including the material type in which each well was screened.

Field hydraulic conductivity testing was conducted in each observation well following the slug test procedures in ASTM D4044 (Reference 202). In addition, field hydraulic conductivities were determined in selected deep bedrock zone boreholes based on the packer test method, as described in ASTM D4630 (Reference 203).

Groundwater-level measurements in the observation wells were taken monthly for one year from June 2006 through June 2007 (Table 2.4-217). Figure 2.4-235 shows hydrographs for all of the saprolite/shallow bedrock zone wells over the monitoring period. Figure 2.4-236 shows hydrographs for all of the deep bedrock zone wells over the monitoring period. In general, the hydrographs indicate that the piezometric levels remained relatively constant during the monitoring period. The exceptions to this include OW-624 in the saprolite/shallow bedrock zone and OW-233 and OW-627a within the deep bedrock zone. For both OW-624 and OW-233, the groundwater level rose quickly over the first four or five readings and then stabilized. These data are interpreted to indicate that low permeability within the screened material caused a slow recovery to original piezometric levels within the aquifer. This would indicate these wells have completed their recovery of groundwater levels due to well installation and that there is minimal seasonal variation in piezometric levels at the site. However, for OW-627a, the hydrograph indicates that piezometric levels rose between the June 2006 and July 2006 readings and then dropped quickly at the time of the August 2006 reading. This rapid drop between July and August appears to be an anomaly because of groundwater sampling of this well. Since August 2006, the piezometric level in OW-627a has been steadily rising, indicating that the well is still recovering to its original levels.

Details of the observation well installation and testing program are provided in Reference 218.

2.4.12.3.2 Groundwater Levels and Flow Directions

2.4.12.3.2.1 Horizontal Groundwater Flow

The groundwater level data for the Units 2 and 3 site was used to determine groundwater flow patterns across the site. Piezometric level contour maps were created for the saprolite/shallow bedrock zone and the deep bedrock zone. One contour map for each zone was created for each quarter using a representative month of piezometric levels. Of the 31 observation wells installed on the site, 22 are completed in the saprolite/shallow bedrock zone and 9 are completed in the deep bedrock zone. This includes five well pairs that consist of two adjacent wells; one completed within the saprolite/shallow bedrock zone and one completed in the deep bedrock zone.

Figures 2.4-237 through 2.4-240 show piezometric level contours for the saprolite/shallow bedrock zone that are representative of each of the four quarters during which levels were recorded. Figures 2.4-241 through 2.4-244 show the piezometric level contours for the deep bedrock zone that are representative of each of the four quarters during which levels were recorded. Groundwater data collected in June 2006, September 2006, December 2006, and March 2007 was used to create the contour maps.

The piezometric contour maps of the saprolite/shallow bedrock zone are very similar for all four quarters. No seasonal changes were observed within the saprolite/shallow bedrock zone. The piezometric contour maps of the deep bedrock zone changed during the monitoring period; however, as stated above, this reflects slow recovery in OW-233.

The piezometric level elevation contour map of the saprolite/shallow bedrock zone indicates that groundwater flows from ridgetops toward drainage swales, with the piezometric surface approximately parallel to the topography. The drainage swales at the site all lead eventually to the west toward the Broad River. The ridge to the north of the power block area (PBA) (in the vicinity of OW-622) appears to be hydraulically connected to the area of Unit 1 which is again connected to the Monticello Reservoir. Contour maps of the deep bedrock zone indicate groundwater flow westward within the bedrock from the PBA off the site toward the Broad River.

The groundwater gradient in the saprolite/shallow bedrock zone ranges from 0.001 to 0.003 ft/ft on top of the ridge, and it is steeper (0.037 to 0.05 ft/ft) on the ridge flanks.

The groundwater gradient in the deep bedrock zone ranges from 0.011 to 0.012 ft/ft on top of the ridge, and it is steeper (0.06 to 0.08 ft/ft) on the ridge flanks.

This groundwater flow regime is consistent with the regional conditions described in Subsection 2.4.12.1.2 and illustrated in Figure 2.4-227.

2.4.12.3.2.2 Vertical Groundwater Flow

Five well pairs were installed as part of the subsurface investigation to assess if the saprolite/shallow bedrock and the deep bedrock zones are hydraulically connected. The well pairs are OW-205a&b, OW-305a&b, OW-401a&b, OW-621a&b, and OW-627a&b. The data from these well pairs indicates that the saprolite/shallow bedrock and the deep bedrock zones are hydraulically connected.

At ridgetops, the water levels within the two aquifers are very nearly the same (OW-305[b-a] & OW-401[b-a]), indicating that the two are directly connected. The average vertical gradient calculated at each of these locations is –0.011 and -0.007, respectively, indicating a slight upward vertical gradient. This is likely because of the slight inaccuracies in measurements of the water levels—there is assumed to be no vertical gradient at the ridge tops. Moving away from the ridgetop toward the ridge flanks the water levels within the two aquifers begin to diverge indicating a downward gradient. The average vertical gradient calculated at OW-205(b-a) is 0.17, indicating a downward gradient. Closer to drainage swales, the difference between the water levels within the two aquifers becomes even greater OW-621(b-a) & OW-627(b-a) (Figure 2.4-245). The average vertical gradient calculated at each of these locations is 1.58 and 2.07, respectively indicating a larger downward vertical gradient. These observations are illustrated in Figure 2.4-245.

2.4.12.3.3 Hydraulic Conductivity

As described in Subsection 2.4.12.3.1, hydraulic conductivities of the site subsurface materials were determined in the observation wells using the slug test method and in selected geotechnical borings using the packer test method. The results of the slug tests are presented in Table 2.4-218 and the results of the packer tests are presented in Table 2.4-219.

Slug tests were conducted in 29 of the 31 observation wells; two wells, OW-312 and OW-501, were not tested. OW-312 was dry, and OW-501 was screened in fill and residual soil.

Of the 29 wells that were tested, 8 were assessed as providing invalid or unreliable test results for the following reasons:

- Large ratio of theoretical head change over the submerged screen length
- Failure to approach asymptote

- Erratic data.

The remaining 21 slug test results were analyzed and low, high, and geometric mean values were calculated for each of the hydrostratigraphic zones. The saprolite/shallow bedrock hydrostratigraphic zone tests were completed in saprolite, partially weathered rock, or a combination of both. Based on 16 slug tests, hydraulic conductivity values for this zone vary from 0.0017 feet/day to 18 feet/day with a geometric mean for this zone of 0.62 feet/day. The deep bedrock hydrostratigraphic zone tests were completed in sound rock. Based on five slug tests, the hydraulic conductivity values for the deep bedrock zone vary from 0.0088 feet/day to 0.38 feet/day with a geometric mean for this zone of 0.07 feet/day. Figure 2.4-246 is a graph of hydraulic conductivity versus depth and hydrostratigraphic zone. This plot indicates that within the saprolite/shallow bedrock zone the hydraulic conductivities do not vary much with depth; however, in the deep bedrock zone, hydraulic conductivities decrease with depth.

Table 2.4-219 gives the results of packer tests conducted in selected geotechnical borings. These tests were conducted in the deep bedrock hydrostratigraphic zone. The hydraulic conductivity values for the deep bedrock zone from the packer tests vary from 0 to 1.14 feet/day with the non-zero packer tests having a geometric mean value of 0.17 feet/day. Some hydraulic conductivity values are listed as zero. This is a result of a test conducted in a zone that did not take any water. This geometric mean hydraulic conductivity value of the packer tests is higher than the 0.07 feet/day geometric mean hydraulic conductivity value indicated by the slug test results for the deep bedrock zone. The differences in values measured by the two tests are interpreted as a result of the depths at which the tests were conducted. The packer tests were generally conducted at shallower depths than the slug tests. The hydraulic conductivity values of the deep bedrock zone increase at shallower depths. When compared with just the shallow slug test results, the packer test values and the slug test values are in much closer agreement (Figure 2.4-246).

Table 2.4-220 presents porosity values derived from laboratory test results for grain size, moisture content, and specific gravity on residual soil and saprolite. The porosity values calculated for the residual soil vary from 0.465 to 0.631, with a geometric mean porosity value of 0.524. The porosity values calculated for the saprolite material vary from 0.401 to 0.632 with a geometric mean porosity value of 0.492. This is based on seven samples of residual soil and 23 samples of saprolite. The saprolite value is considered to be representative of the porosity value for the saprolite/shallow bedrock zone.

2.4.12.3.4 Groundwater Quality

Naturally occurring radionuclides have been documented within the deep bedrock zone in other parts of the state, as discussed in Subsection 2.4.12.2.3. Naturally occurring radionuclides have also been found in groundwater supply wells of the Jenkinsville Water Company closer to the site. The SCDHEC provided copies of communications dating back to 1974 between the SCDHEC and the Jenkinsville Water Company (Reference 228). These communications indicate that several water supply wells used by the Jenkinsville Water Company were found to contain naturally occurring radionuclides and needed to be removed from the water system and abandoned. The radionuclides present in the Jenkinsville Water Company wells consisted of Radium-226 and its daughter products.

2.4.12.3.5 Subsurface Pathways

Units 2 and 3 are located on a ridgetop. Piezometric contour maps developed from piezometric levels measured for one year from June 2006 through June 2007 indicate that groundwater flows in all directions from the ridgetop. Drainage swales are present to the northwest, southwest, and east of the site, as can be seen from the topographic map in Figure 2.4-201. All of these drainage swales are tributaries that eventually lead to the Broad River. The Broad River is located approximately 1 mile to the west of the site. Groundwater levels and flow directions discussed in Subsection 2.4.12.3.2 indicate that the shallow subsurface groundwater flow regime is approximately parallel to the topography and flows through the saprolite/shallow bedrock hydrostratigraphic zone. This data also indicates that groundwater from the saprolite/shallow bedrock zone recharges the deep bedrock hydrostratigraphic zone. Piezometric level contour maps developed for the deep bedrock zone indicate a flow path that leads directly toward the Broad River. Subsection 2.4.13 discusses subsurface pathways in more detail relative to an accidental release of radioactive liquid effluent.

2.4.12.3.6 Plant Groundwater Use and Effects

Groundwater is not used by Unit 1 and is not used to supply water to Units 2 and 3.

2.4.12.4 Monitoring or Safeguard Requirements

Long-term monitoring of groundwater quality at the VCSNS site takes place through programs implemented for Unit 1.

As part of detailed engineering for Units 2 and 3, the Unit 1 groundwater monitoring programs will be evaluated with respect to the addition of Units 2 and 3 to determine if any modification of the programs is required to adequately monitor plant effects on the groundwater.

Administrative controls will be used to minimize the potential for adverse impacts to the groundwater by construction and operation of Units 2 and 3. These controls consider the use of lined containment structures around storage tanks and hazardous materials storage areas, emergency cleanup procedures to capture and remove surface contaminants, and other measures deemed necessary to prevent or minimize adverse impacts to the groundwater beneath the VCSNS site and surrounding groundwater users.

Groundwater level measurements were made monthly in the installed observation wells from June 2006 through June 2007 as part of a geotechnical subsurface investigation program.

2.4.12.5 Design Basis for Dewatering and Subsurface Hydrostatic Loading

The need for a permanent groundwater dewatering system is not anticipated for the Units 2 and 3 site because the maximum expected groundwater level is 20 feet below site grade. However, localized temporary dewatering is expected to be required during construction.

A sump and pump temporary dewatering system is expected to be sufficient during plant foundation excavation and construction. The final design of this system will be prepared after further development of the excavation design. Modifications to the system will be made as warranted during construction.

For Units 2 and 3, the maximum expected groundwater level for both aquifer zones is at El. 380 feet NAVD88. This number is based on groundwater level observations between June 2006 and June 2007 at the VCSNS site.

The maximum groundwater level encountered in Unit 2 during this period was at El. 367.4 feet NAVD88 and the maximum groundwater level encountered in the Unit 3 location was 375.1 feet NAVD88 (Table 2.4-221). Because the period of observation is limited, the maximum groundwater elevation in the saprolite/upper bedrock zone could be higher. Over the 13 months of readings, the minimum change in water level readings was 0.9 feet and the maximum was 2.3 feet (a change of 44.4 feet was noted in OW-233 and a change of 5.7 feet was noted for OW-333, but this is attributed to slow recovery of water levels within the well after construction as a result of low permeability of the screened interval of the bedrock). Based on this observed fluctuation in the monitoring wells to date, a value of approximately 5 feet (which is approximately 2 times the observed seasonal fluctuation) was added to the maximum observed water level of 375.1 feet NAVD88 to determine the maximum expected groundwater level (380 feet NAVD88).

Table 2.4-221 provides the groundwater data available for the Units 2 and 3 site. Groundwater data from Unit 1 locations was used to confirm that these Unit 2 and 3 numbers were reasonable. Table 2.4-222 provides a listing of groundwater wells installed at Unit 1 and at the Units 2 and 3 site. The Unit 1 groundwater data includes water levels recorded in the piezometry program, auxiliary boiler fuel oil storage tank, and National Pollutant Discharge Elimination System (NPDES) wells. The maximum range of values from the auxiliary boiler fuel oil storage tank and NPDES wells over the period between 1998 and 2006 was 4.36 feet. GW-12 indicated a range of 14 feet; however, because the shape of this hydrograph is not consistent with the other four wells and all wells are completed in the same aquifer zone, the data from this well has not been used in this analysis. The piezometry program wells indicated a much larger range of values; however, these wells were installed to determine the change in water level associated with the impoundment of the Monticello Reservoir and are not considered directly applicable to Units 2 and 3. Additionally, the auxiliary boiler fuel oil storage tank and NPDES hydrographs (Figures 2.4-247 and 2.4-248) indicate that 2006–2007 water levels are not significantly higher or lower than the water levels measured in these wells over the previous 10 years.

Groundwater data from the auxiliary boiler fuel oil storage tank and NPDES wells was compared with precipitation data obtained from the Parr Climate Station. Table 2.4-223 summarizes the precipitation data from Parr Climate Station by water year. Figures 2.4-249 and 2.4-250 shows groundwater depth at the auxiliary boiler fuel oil storage tank and NPDES wells with precipitation annual departure from the mean and groundwater depth in the auxiliary boiler fuel oil storage tank and NPDES wells with precipitation cumulative annual departure from the mean, respectively. Each well was compared individually to determine if there was a significant correlation between depth to groundwater and precipitation data. Table 2.4-224 summarizes the results of these correlations. It was found that there was no significant correlation between annual maximum groundwater levels observed in the wells and annual precipitation values. This may be due to the proximity of these wells to the Monticello Reservoir.

A hydrograph from the South Carolina Water Plan (Reference 204) showing typical seasonal variations in groundwater level within the Piedmont Aquifer (Figure 2.4-228) indicates depth-to-water values ranging approximately 6 feet between 1991 and 1996. This is further evidence that the addition of 5 feet to the maximum recorded groundwater level for determination of the maximum expected groundwater level is reasonable.

For Units 2 and 3, the deepest foundation in the PBA is at El. 360.5 feet MSL (Plot Plan, 25242-0-P1-0010-00002) for the nuclear island structures. This elevation is approximately 19.5 feet below the design water level (El. 380 feet NAVD88) resulting in 19.5 feet of hydrostatic head beneath the containment structures. The excavation for the nuclear island is likely to reach an average elevation of 355 feet NAVD88 (Subsection 2.5.4). This would result in 24.5 feet of hydrostatic head at the base of the nuclear island excavation. The design plant grade elevation is 400 feet NAVD88. The maximum expected groundwater level of 380 feet NAVD88 lies 20 feet below the plant grade elevation.

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2.4.13 ACCIDENTAL RELEASES OF RADIOACTIVE LIQUID EFFLUENTS IN GROUND AND SURFACE WATERS

2.4.13.1 Accidental Releases to Groundwater

The assumed accidental release scenario has been selected based on information developed by Westinghouse to assist in evaluating the accidental liquid release of effluents (Reference 252). The scenario assumes an instantaneous release from one of the two effluent holdup tanks located in the lowest level of the AP1000 auxiliary building.

There are two effluent holdup tanks for each unit, each with a capacity of 28,000 gallons. These tanks have both the highest potential radionuclide concentrations and the largest volume. Therefore, they have been selected as the limiting tanks for evaluating an accidental release of liquid effluents that could lead to the most adverse contamination of groundwater or surface water, via the groundwater pathway.

The estimated radionuclide concentration of the effluent holdup tanks is assumed to be equal to 101% of the reactor coolant concentration. Westinghouse determined that the radionuclide concentrations in the reactor coolant itself should be estimated as follows:

- For tritium (H-3), a coolant concentration of 1.0 μCi/g should be used.
- Corrosion products (Cr-51, Mn-54, Mn-56, Fe-55, Fe-59, Co-58 and Co-60) should be taken directly from the AP1000 DCD, Table 11.1-2, *Design Basis Reactor Coolant Activity*.
- Other radionuclides should be based on the AP1000 DCD, Table 11.1-2 multiplied by 0.12/0.25 to adjust the failed fuel rate from the design basis to a conservatively bounding value for this analysis.

Based on these recommendations, the expected radionuclide concentrations in the effluent holdup tanks have been calculated, and the results are summarized in Table 2.4-225.

The assessment of a potential release of radioactive liquids following the postulated failure of a tank and its components, located outside of containment, and the impacts of the release of radioactive materials at the nearest potable water supply, located in an unrestricted area, presented in this subsection follows the guidance provided in Branch Technical Position 11-6 (Reference 247).

2.4.13.1.1 Conceptual Model

Figure 2.4-251 illustrates the conceptual model used to evaluate an accidental release of liquid effluent to groundwater, or to surface water via the groundwater pathway. The key elements and assumptions embodied in the conceptual model are described and discussed below.

As indicated above, the effluent holdup tanks are assumed to be the source of the release, with each tank having a capacity of 28,000 gallons and radionuclide concentrations as summarized in Table 2.4-225. These tanks are located at the lowest level of the auxiliary building of each unit, which has a building slab elevation of 366.5 feet NAVD88 and a base of concrete elevation of 360.5 feet NAVD88, and is about 5 to 7 feet below the 2007 groundwater levels and approximately 17 feet below the maximum water table level. One of these tanks is postulated to rupture, and 80% of the liquid volume (22,400 gallons is assumed to be released in accordance with NUREG-0800, BTP 11-6). Flow from a tank rupture would initially flood the tank room, and begin to flow to the auxiliary building radiologically controlled area sump via floor drains, as described in Subsection 3.4.1.2.2.2 of the DCD. It is assumed that the sump pumps are inoperable. According to the DCD, this would result in the 22,400-gallon release flooding the balance of Level 1 of the auxiliary building via the interconnecting floor drains. Once Level 1 is flooded, it is assumed that a pathway is created that would allow the entire 22,400 gallons to enter the groundwater (unconfined aquifer) instantaneously. This assumption is very conservative because it requires failure of the floor drain system, plus it ignores the barriers presented by the 6-foot-thick basemat and the sealed, 3-foot-thick exterior walls of the auxiliary building.

As already mentioned, the slab of the floor where effluent holdup tanks are located is well below the water table (about 5 to 7 feet below the 2007 groundwater levels and 17 feet below the maximum water level). Therefore, any cracks in the foundation would result in an influx of uncontaminated groundwater to the auxiliary building as opposed to an efflux of radioactive water out of the building and to the subsurface. Considering the release of nuclides, despite this flow condition that would tend to prevent it, adds conservatism to the present analysis.

With the postulated instantaneous release of the contents of an effluent holdup tank to groundwater, radionuclides would enter the saprolite/shallow bedrock zone and migrate with the groundwater in the direction of decreasing the hydraulic head. The hydraulic head contour map for the saprolite/shallow bedrock zone presented on Figure 2.4-252 indicates that the groundwater pathway from a point of release in either of the auxiliary buildings would be either towards the unnamed creek north-northwest of Unit 2, or south-southwest of Unit 3 towards an unnamed creek as illustrated conceptually in Figure 2.4-251. Cross sections were developed roughly along these groundwater pathways as indicated in Figure 2.4-253. These cross sections, shown in Figures 2.4-254 and 2.4-255, represent the profile along the pathways from Unit 2 and Unit 3, respectively, to the nearest groundwater discharge point. These cross sections indicate that flow from the source to the release point would be primarily through the saprolite material. During saturated zone transport, radionuclide concentrations of the liquid released to the water table would be reduced by the processes of adsorption, hydrodynamic dispersion, and radioactive decay. Upon discharge to either of the two unnamed creeks, radionuclides would mix with uncontaminated surface water in the creeks and eventually discharge into the Broad River, leading to further reduction of concentrations.

An alternative conceptual model for nuclide transport is for flow through the bedrock. This would be consistent with the downward hydraulic gradient at the site. However, as discussed in Subsection 2.4.12.3.3, on the properties of the different subsurface materials at the site, the saprolite material has a much higher hydraulic conductivity, and therefore provides the fastest groundwater pathway. Therefore, the alternative conceptual model through the bedrock would provide lower nuclide concentrations at the groundwater discharge points. Similarly, other alternative conceptual models, for example groundwater discharge to other nearby creeks, would result in longer pathways and longer travel times. The pathways selected for analysis bound the set of plausible alternatives because they combine the shortest distance from the point of release to the point of groundwater discharge to the surface with the material—saprolite, which has the highest hydraulic conductivity of all other materials found at the site. This combination provides the shortest travel time, therefore bounding all plausible alternatives.

There are no existing water supply wells between the postulated release points and the points where groundwater discharges. Surface water users downstream of VCSNS on the Broad River are discussed in Subsection 2.4.1.2.3 and are listed in Table 2.4-205. Figure 2.4-209 shows the rivers and lakes where the downstream water users listed in Table 2.4-205 are located.

Potable water resources on SCE&G property and under the control of SCE&G have not been analyzed. In the event of an accidental tank rupture event, SCE&G will monitor and control these sources in accordance with regulatory limits.

2.4.13.1.2 Radionuclide Transport Analysis

2.4.13.1.2.1 Modeling Approach

A radionuclide transport analysis was conducted to estimate the radionuclide concentrations that might impact existing and future water users in the vicinity of Units 2 and 3 based on an instantaneous release of the radioactive material contents of an effluent holdup tank.

The analysis of nuclide transport commences with the simplest of models, using radioactive decay only, and ignoring adsorption and dispersion. Radionuclide concentrations resulting from the preliminary analysis are then compared against the maximum permissible concentrations identified in 10 CFR Part 20, Appendix B, Table 2, Column 2 to determine acceptability. If required, further analysis using adsorption in the subsurface will be completed. If the concentrations still exceed an acceptable level after evaluation using radioactive decay and adsorption, further analysis will be completed accounting for dilution in surface water.

This analysis accounts for the parent radionuclides expected to be present in the radwaste tank plus progeny radionuclides that would be generated subsequently during transport. The analysis considered all progeny in the decay chain sequences that are important for dosimetric purposes. The International Commission on Radiation Protection Publication 38 (Reference 210) was used to identify the member for which the decay chain sequence can be truncated. For some of the radionuclides expected to be present in an effluent holdup tank, consideration of up to three members of the decay chain sequence was required. The derivation of the equations governing the transport of the parent and progeny radionuclides follows.

Radionuclide transport along a groundwater pathline is governed by the advection-dispersion-reaction equation (Reference 215):

$$R\frac{\partial C}{\partial t} = D\frac{\partial^2 C}{\partial x^2} - v\frac{\partial C}{\partial x} - \lambda RC \qquad \text{(Equation 2.4-1)}$$

where: *C* = radionuclide concentration; *R* = retardation factor; *D* = coefficient of longitudinal hydrodynamic dispersion; *v* = average linear groundwater velocity; and λ = radioactive decay constant. The retardation factor is defined from the relationship:

$$R = 1 + \frac{\rho_b K_d}{n_e}$$

(Equation 2.4-2)

where: ρ_b is the bulk density; K_d is the distribution coefficient; n_e is the effective porosity. The average linear groundwater velocity is determined using Darcy's law, i.e.:

$$v = -\frac{K}{n_e}\frac{dh}{dx}$$

(Equation 2.4-3)

where: *K* is the hydraulic conductivity; and *dh/dx* is the hydraulic gradient. The radioactive decay constant can be written as:

$$\lambda = \frac{\ln 2}{t_{1/2}}$$

(Equation 2.4-4)

where $t_{1/2}$ is the radionuclide half-life.

Using the method of characteristics approach, the material derivative of concentration can be written as:

$$\frac{dC}{dt} = \frac{\partial C}{\partial t} + \frac{dx}{dt}\frac{\partial C}{\partial x}$$

(Equation 2.4-5)

Conservatively neglecting hydrodynamic dispersion, the characteristic equations for Equation 2.4-1 can be expressed as follows:

$$\frac{dC}{dt} = -\lambda C$$

(Equation 2.4-6)

$$\frac{dx}{dt} = \frac{v}{R}$$ (Equation 2.4-7)

The solutions of the system of equations comprising Equations 2.4-6 and 2.4-7 can be obtained by integration to yield the characteristic curves of Equation 2.4-1. For the parent radionuclide, the equations representing the characteristic curves can be obtained as:

$$C_1 = C_{10} \exp(-\lambda_1 t)$$ (Equation 2.4-8)

$$t = R_1 L / v$$ (Equation 2.4-9)

where: C_1 is the concentration of the parent radionuclide; C_{10} is the initial concentration of the parent radionuclide; λ_1 is the radioactive decay constant for the parent radionuclide; R_1 is the retardation factor for the parent radionuclide; and L is the groundwater pathline length.

Similar equations can be written for the progeny radionuclides. For the first progeny in the decay chain, the advection-dispersion-reaction equation is written as:

$$R_2 \frac{\partial C_2}{\partial t} = D \frac{\partial^2 C_2}{\partial x^2} - v \frac{\partial C_2}{\partial x} + d_{12} \lambda_1 R_1 C_1 - \lambda_2 R_2 C_2$$ (Equation 2.4-10)

where the subscript 2 denotes the first progeny radionuclide; and d_{12} is the fraction of parent radionuclide transitions that result in production of progeny radionuclides. The characteristic equations for Equation 2.4-10, again conservatively neglecting hydrodynamic dispersion, can be derived as:

$$\frac{dC_2}{dt} = d_{12} \lambda_1' C_1 - \lambda_2 C_2$$ (Equation 2.4-11)

$$\frac{dx}{dt} = \frac{v}{R_2}$$ (Equation 2.4-12)

where: $\lambda_1' = \lambda_1 R_1 / R_2$. Recognizing that Equation 2.4-11 is formally similar to Equation B.43 of Reference 216, these equations can be integrated to yield:

$$C_2 = K_1 \exp(-\lambda_1' t) + K_2 \exp(-\lambda_2 t)$$ (Equation 2.4-13)

$$t = R_2 L / v$$ (Equation 2.4-14)

for which:

$$K_1 = \frac{d_{12}\lambda_2 C_{10}}{\lambda_2 - \lambda_1'}$$

$$K_2 = C_{20} - \frac{d_{12}\lambda_2 C_{10}}{\lambda_2 - \lambda_1'}$$

The advection-dispersion-reaction equation for the second progeny in the decay chain is:

$$R_3 \frac{\partial C_3}{\partial t} = D\frac{\partial^2 C_3}{\partial x^2} - v\frac{\partial C_3}{\partial x} + d_{13}\lambda_1 R_1 C_1 + d_{23}\lambda_2 R_2 C_2 - \lambda_3 R_3 C_3$$ (Equation 2.4-15)

where: subscript 3 denotes the second progeny radionuclide; d_{13} is the fraction of parent radionuclide transitions that result in production of second progeny radionuclide; and d_{23} is the fraction of first progeny radionuclide transitions that result in production of second progeny radionuclide. The characteristic equations for Equation 2.4-15, again conservatively neglecting hydrodynamic dispersion, can be derived as:

$$\frac{dC_3}{dt} = d_{13}\lambda_1' C_1 + d_{23}\lambda_2' C_2 - \lambda_3 C_3$$ (Equation 2.4-16)

$$\frac{dx}{dt} = \frac{v}{R_3}$$ (Equation 2.4-17)

where: $\lambda_1' = \lambda_1 R_1 / R_3$; and $\lambda_2' = \lambda_2 R_2 / R_3$. Considering the formal similarity of Equation 2.4-16 to Equation B.54 of Reference 216, Equations 2.4-16 and 2.4-17 can be integrated to yield:

$$C_3 = K_1 \exp(-\lambda_1' t) + K_2 \exp(-\lambda_2' t) + K_3 \exp(-\lambda_3 t) \qquad \text{(Equation 2.4-18)}$$

$$t = R_3 L / v \qquad \text{(Equation 2.4-19)}$$

for which:

$$K_1 = \frac{d_{13}\lambda_3 C_{10}}{\lambda_3 - \lambda_1'} + \frac{d_{23}\lambda_2' d_{12}\lambda_3 C_{10}}{(\lambda_3 - \lambda_1')(\lambda_2' - \lambda_1')}$$

$$K_2 = \frac{d_{23}\lambda_3 C_{20}}{\lambda_3 - \lambda_2'} - \frac{d_{23}\lambda_2' d_{12}\lambda_3 C_{10}}{(\lambda_3 - \lambda_2')(\lambda_2' - \lambda_1')}$$

$$K_3 = C_{30} - \frac{d_{13}\lambda_3 C_{10}}{\lambda_3 - \lambda_1'} - \frac{d_{23}\lambda_3 C_{20}}{\lambda_3 - \lambda_2'} + \frac{d_{23}\lambda_2' d_{12}\lambda_3 C_{10}}{(\lambda_3 - \lambda_1')(\lambda_3 - \lambda_2')}$$

To estimate the radionuclide concentrations in groundwater discharging to the surface, Equations 2.4-8, 2.4-13, and 2.4-18 were applied as appropriate along the groundwater pathline that would originate at either of the liquid effluent release points beneath the auxiliary buildings and terminate at the points of groundwater discharge in the two nearby creeks. First, a screening analysis was completed considering radioactive decay only. Then, those radionuclides that exceeded 1% of their maximum permissible concentrations were included in the next step of the analysis which accounted for radioactive decay as well as retardation because of adsorption. These two steps are described in more detail in the next two subsections.

2.4.13.1.2.2 Screening Analysis: Transport Considering Radioactive Decay Only

In the initial screening analysis, the concentrations of the radionuclides appearing in Table 2.4-225 were decayed for a period equal to the groundwater travel time from the point of release to each of the two nearby creeks using Equations 2.4-8, 2.4-13, or 2.4-18, as appropriate, with $R_1 = R_2 = R_3 = 1$. The travel time in the saprolite/shallow bedrock zone between each of the two auxiliary buildings and the nearest creek where groundwater discharges were conservatively determined based on site-specific data that is summarized in Subsection 2.4.12.3.3. The horizontal hydraulic gradients were calculated as shown in Table 2.4-226.

For the unnamed creek to the north-northwest of the site, the average advective velocity is calculated using the following parameters:

> hydraulic conductivity K = 1.7 feet/day (75th percentile hydraulic conductivity value from the slug test data in the saprolite material).
>
> effective porosity n_e = 0.39 (The effective porosity of the saprolite was estimated using Figure 2.17 of Reference 211.)
>
> horizontal hydraulic gradient $\frac{dh}{dx}$ = - 0.0307 ft/ft.

Substituting these values in Equation 2.4-3 yields:

$$v = -\frac{K}{n_e}\frac{dh}{dx} = -\frac{1.7\, ft/day}{0.39}\left(-0.0307\, ft/ft\right) = 0.134\, ft/day \approx 48.9\, ft/yr$$

The straight-line distance from the auxiliary building of Unit 2 to the unnamed creek to the north-northwest of the site is about L = 850 feet, which results in a conservatively estimated groundwater travel time of:

$$t = \frac{LR}{v} = \frac{850\, ft \times 1}{48.9\, ft/yr} \approx 17.4\, yrs$$

For the unnamed creek to the south-southwest of the site, the average advective velocity is calculated using the following parameters:

> hydraulic conductivity K = 1.7 feet/day (75th percentile hydraulic conductivity value from the slug test data in the saprolite material)
>
> effective porosity n_e = 0.39
>
> horizontal hydraulic gradient $\frac{dh}{dx}$ = - 0.0369 ft/ft.

Substituting these values in Equation 2.4-3 yields:

$$v = -\frac{K}{n_e}\frac{dh}{dx} = -\frac{1.7\,ft/day}{0.39}\left(-0.0369\,ft/ft\right) = 0.161\,ft/day \approx 58.8\,ft/yr$$

The straight-line distance from the auxiliary building of Unit 3 to the unnamed creek south-southwest of the site is about L = 1727 feet, which results in a conservatively estimated groundwater travel time of:

$$t = \frac{LR}{v} = \frac{1727\,ft \times 1}{58.8\,ft/yr} \approx 29.4\,yrs$$

The estimated concentrations at the end of the pathways originating from each of the two potential release points are given in Table 2.4-227 and Table 2.4-228. Most of the estimated radionuclide concentrations shown in these tables have concentrations less than 1% of their respective maximum permissible concentrations and are eliminated from further consideration because their concentrations would be well below their regulatory limits. Table 2.4-227 and Table 2.4-228 identify the radionuclides that exceed 1% of their maximum permissible concentration. These are H-3, Fe-55, Co 60, Sr-90, Y-90, I-129, Cs-134, and Cs-137 for a travel time of 17.4 years and H-3, Co 60, Sr-90, Y-90, I-129, Cs-134, and Cs-137 for a travel time of 29.4 years. These radionuclides are retained for further evaluation.

2.4.13.1.2.3 Transport Considering Radioactive Decay and Adsorption

The radionuclides retained from the screening analysis (H-3, Fe-55, Co-60, Sr-90, Y-90, I-129, Cs-134, and Cs-137) were further evaluated considering adsorption and retardation in addition to radioactive decay. Laboratory measurements of onsite materials were used to establish K_d values for Cobalt (Co-60), Strontium (Sr-90), and Cesium (Cs-134 and Cs-137) as reported in Attachment H of Reference 217. The lowest reported value within the saprolite material for each radionuclide was used in this analysis (Table 2.4-229). The distribution coefficients for H-3 and I-129 were taken equal to zero because of their chemical characteristics. Because no site-specific data for the distribution coefficients of Fe-55 and Y-90 was available, distribution coefficients for these two nuclides were conservatively taken as equal to zero. Retardation factors were then calculated using Equation 2.4-2 with an effective porosity of n_e = 0.39 and bulk density of ρ_b = 1.41 g/cm^3, based on information provided in Subsection 2.4.12.3.3.

Concentrations were then determined at the point of groundwater discharge in the two unnamed creeks, north-northwest of Unit 2 and south-southwest of Unit 3, using Equations 2.4-8, 2.4-13, or 2.4-18 with the appropriate retardation factors. Table 2.4-230 and Table 2.4-231 provide the calculations and results for Units 2 and 3, respectively. The radionuclides with concentrations greater than 1% of their respective maximum permissible concentrations are H-3, Fe-55 and I-129 for Unit

2. For Unit 3 the radionuclides with concentrations greater than 1% of their respective maximum permissible concentrations are H-3 and I-129. The transport of H-3, Fe-55, and I-129 is analyzed further for Unit 2 and the transport of H-3 and I-129 is analyzed further for Unit 3. Subsection 2.4.13.1.2.4 follows the transport of these nuclides through the respective unnamed creeks downstream of the point of the groundwater discharge and into the Broad River.

2.4.13.1.2.4 Transport Considering Radioactive Decay, Adsorption, and Dilution

The H-3, Fe-55, and I-129 discharging with the groundwater to the two unnamed creeks near the site of Units 2 and 3 would mix with uncontaminated groundwater discharging into the creeks and into the Broad River, leading to further reduction of concentrations. As determined in Subsection 2.4.13.1.2.3, H-3, Fe-55, and I-129 would be discharged from the local groundwater in the unnamed creek north-northwest of Unit 2, and H-3 and I-129 would be discharged from the local groundwater in the unnamed creek south-southwest of Unit 3, as shown on Figure 2.4-252.

For the purpose of evaluating the effects of an accidental release of radioactive liquid effluents on the surface water systems downstream of the discharge point, the average concentration and discharge of the highly diluted liquid effluent discharged from the aquifer was determined. The present analysis is based on the conservative assumption that there is no transverse or longitudinal dispersion in the subsurface.

It is assumed that in the event of an accidental release of the contents of the effluent holdup tank, the liquid will directly enter the saprolite, the most permeable material under Units 2 and 3. The rate at which a release from an effluent holdup tank discharges to surface water is determined by the transport characteristics of the saprolite. A release from an effluent holdup tank would travel through the saturated saprolite to the release points in the two unnamed creeks as shown on Figure 2.4-252.

The discharge rate itself is a function of the Darcy velocity, and the assumed volume and dimensions of the resulting contaminant slug. For Unit 2, the Darcy velocity was calculated to be 0.0522 feet/day, using a hydraulic conductivity of 1.7 feet/day and a hydraulic gradient of 0.0307 ft/ft. For Unit 3, the Darcy velocity was calculated to be 0.0627 feet/day, using a hydraulic conductivity of 1.7 feet/day and a hydraulic gradient of 0.0369 ft/ft. These values are based on the hydrogeologic characteristics of the saprolite as described previously. The hydraulic gradients are the most conservative based on groundwater levels measured at the site between June 2006 and June 2007. The volume of the liquid release has been assumed to be 22,400 gallons (approximately 2,995 ft^3), which represents 80% of the 28,000-gallon capacity of one effluent holdup tank (NUREG-0800, BTP 11-6 recommends that 80% of the liquid volume be considered in this analysis). Considering the effective porosity of the saprolite (0.39), the volume of the saturated saprolite that would be occupied by the release is:

$$V_{saprolite} = \frac{V_{release}}{n_e} = \frac{2995 \text{ ft}^3}{0.39} = 7679 \text{ ft}^3$$

The shape of the resulting contaminant slug is assumed to be square in plan view and extend vertically throughout the entire saturated thickness of the saprolite. Using 10 feet as a representative saturated thickness (water table to base of saprolite), the slug would have an area of about 767.9 square feet in plan view and a width of about 27.7 feet. The cross-sectional area of the contaminant slug normal to the groundwater flow direction would therefore be:

$A = 10 \times 27.7 = 277 \text{ ft}^2$

The total flow through this area from Unit 2 toward the nearest unnamed creek is estimated to discharge into the unnamed creek to the north-northwest of Unit 2 at a rate, Q_2, of about:

$Q_2 = AU_2 = 277 \times 0.0522 \cong 14.46 \text{ ft}^3\text{/day} \cong 1.67 \times 10^{-4}$ cfs

The total flow through this area from Unit 3 toward the nearest unnamed creek is estimated to discharge into the unnamed creek to the south-southwest of Unit 3 at a rate, Q_3, of about:

$Q_3 = AU_3 = 277 \times 0.0627 \cong 17.37 \text{ ft}^3\text{/day} \cong 2.01 \times 10^{-4}$ cfs

These are the flow rates of groundwater contaminated with H-3, Fe-55, and I-129 that would flow to the unnamed creek at each location. Before reaching the Broad River, the spill would be further diluted by water originating from uncontaminated portions of the local groundwater regime that is flowing into the unnamed creeks downstream of discharge points in the two unnamed creeks.

Upon reaching the Broad River, further dilution of an accidental release of radioactive liquid effluents would result from the natural stream flow into the Parr Reservoir. To add conservatism to the analysis, the 100-year daily mean low flow in the Broad River is used to estimate the dilution factor. This is very conservative because the time scale for groundwater transport is much greater than a day. Therefore, the associated flow rate in the river that would mix with the groundwater discharge would also be greater over a longer duration. The 100-year daily mean low flow in the Broad River at Parr Shoals Dam is 125 cfs, as described in Subsection 2.4.11. Assuming that the accidental liquid effluent release occurs during the 100-year low flow in the Broad River, the corresponding dilution factor would be equal to $1.67 \times 10^{-4}/125 \cong 1.34 \times 10^{-6}$ for Unit 2 and $2.01 \times 10^{-4}/125 \cong 1.61 \times 10^{-6}$ for Unit 3. Calculation of the dilution factor is illustrated in Table 2.4-232 and Table 2.4-233 for Units 2 and 3, respectively.

This dilution factor is applied to the H-3, Fe-55, and I-129 concentrations reported in Table 2.4-230 and Table 2.4-231 to account for dilution in addition to radioactive decay and adsorption. Table 2.4-234 summarizes the resulting concentrations, which would represent the concentrations in the surface water at Parr Shoals Dam.

The above analysis of an accidental release of radioactive liquid effluents reaching the surface water system has conservatively neglected the initial dilution provided by the water impounded in the Parr Reservoir. Even at its minimum pool elevation of 256 feet the Parr Reservoir has a storage volume of 2,500 acre-feet available for dilution. Credit for the additional dilution provided by tributary inflow at downstream locations along the Broad River has also not been considered.

2.4.13.2 Accidental Releases to Surface Waters

2.4.13.2.1 Direct Releases to Surface Waters

No outdoor tanks contain radioactivity in the AP1000 design (Reference 252). In particular, the AP1000 design does not require boron changes for load follow and does not recycle boric acid or reactor coolant water, so the boric acid tank is not radioactive.

2.4.13.3 Compliance with 10 CFR Part 20

Compliance with 10 CFR Part 20 is required for the nearest potable water supply in an unrestricted area. There are no potable water supplies between Units 2 and 3 and the Broad River. The nearest downstream potable water supply is the Columbia Canal Water Plant, located approximately 28 miles downstream. Therefore, compliance with 10 CFR Part 20 is evaluated in the Broad River downstream of the site.

The radionuclide transport analysis presented above demonstrates that each of the radionuclides that could be accidentally released to the groundwater is individually below its maximum permissible concentration. 10 CFR Part 20, Appendix B, Table 2 imposes additional requirements when the identity and concentration of each radionuclide in a mixture are known. In this case, the ratio present in the mixture and the concentration otherwise established in Appendix B for the specific radionuclide not in a mixture must be determined. The sum of such ratios for all of the radionuclides in the mixture may not exceed “1” (*i.e.,* “unity”).

This sum-of-fractions approach has been applied to the radionuclide concentrations conservatively estimated above. Results are summarized in Table 2.4-235 and Table 2.4-236. The sum of the ratios of the concentration of each nuclide over the corresponding maximum permissible concentration for Units 2 and 3 are 5.32×10^{-4} and 3.01×10^{-3} respectively, which is below unity. Therefore, it is concluded that an accidental liquid release of effluents in groundwater would not exceed 10 CFR Part 20 limits.

No other hazards exist close to the site that could affect the radioactive concentration from the postulated tank failure related to accidental release of radioactive liquid effluents to ground and surface waters for the proposed plant site. This includes both seismic and non-seismic events.

VCS COL 2.4-6

2.4.14 TECHNICAL SPECIFICATIONS AND EMERGENCY OPERATION REQUIREMENTS

The hydrologic design bases developed in preceding sections do not indicate that technical specifications or emergency procedures are necessary to ensure safety-related plant functions are protected against flooding. The Unit 2 and 3 location is a dry site as described in Subsection 2.4.3 and the site parameter for flooding elevation (reference El. 100 feet in DCD Table 2-1) is met (see Table 2.0-201). In addition, the effects of seismic, wind, surge, seiche, tsunami, ice, and local intense precipitation on flooding have been considered in Subsections 2.4.3 through 2.4.7 in meeting the site parameter for flooding.

Units 2 and 3 do not use the Monticello Reservoir as a safety-related water source. Units 2 and 3 are AP1000 designs that use a passive cooling design as described in DCD Subsection 6.2.2. No external sources of cooling makeup water are required for at least 7 days following an accident.

The safety-related systems, structures, and components for Units 2 and 3 are protected against flooding as discussed in DCD Section 3.4.

2.4.15 REFERENCES

201. American Nuclear Society, ANSI/ANS 2.8-1992, *Determining Design Basis Flooding at Power Reactor Sites*, July 1992.

202. ASTM, *Standard Test Method (Field Procedure) for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers*, ASTM D-4044-96 (Reapproved 2002), ASTM International.

203. ASTM, *Standard Test Method for Determining Transmissivity and Storage Coefficient of Low Permeability Rocks by In Situ Measurements Using the Constant Head Injection Test*, ASTM D-4630-96 (Reapproved 2002), ASTM International.

204. Badr, A. W., Wachob, A. and Gellici, J. A, *South Carolina Water Plan,* Second Edition, South Carolina Department of Natural Resources, Land, Water and Conservation Division, January 2004.

205. Bradford, Thomas G., South Carolina Atlas Map. 1:267,200-scale Engraved by, Boynton, George W, Published by Boston: Weeks, Jordan & Co.; New York: Wiley and Putnam, 1838.

206. Childress, J. M, *South Carolina Ambient Groundwater Quality Report 2005 Summary*, South Carolina Department of Environmental Control, Bureau of Water, February 2006.

207. Childress, J. M. and Butler, A., *South Carolina Water Use Report 2005 Summary*, South Carolina, Department of Environmental Control, Bureau of Water, April 2006.

208. Cook, James 1773. *A Map of the Province of South Carolina*. Cummings 443 (illustrated plate 67) and p 59; Sellers and Van Ee 1521; J.D. Black, Map Collector 34, p. 10-15; Cumming, British Maps of Colonial America p.47-48; Tooley Dictionary p. 130.

209. Cooney, T. W., Drewes, P. A., Ellisor, S. W., Lanier, T. H. and Melendez, F., 2006: *Water Resources Data South Carolina: Water Year 2005*, Water-Data Report SC-05-1, U.S. Department of the Interior, U.S. Geological Survey, Columbia, South Carolina, 621 pages. Available at http://pubs.er.usgs.gov/usgspubs/wdr/wdrSC051.

210. ICRP, *Radionuclide Transformations – Energy and Intensity of Emissions*, ICRP Publication 38, Volumes 11-13, Pergamon Press, New York, 1983.

211. de Marsily, G., *Quantitative Hydrogeology: Groundwater Hydrology for Engineers*, Academic Press, Inc., Orlando, Florida 1986.

212. Not used.

213. Not used.

214. Henderson, F. M., *Open Channel Flow*, Macmillan, 1966.

215. Javandel, I., Doughty, C. and Tsang, C-F, 1984: *Groundwater Transport: Handbook of Mathematical Models*, Water Resources Monograph 10, American Geophysical Union, 1984.

216. Kennedy, W. E., and Strenge, D. L., *Residual Radioactive Contamination From Decommissioning*, NUREG/CR-5512, Volume 1, Pacific Northwest Laboratory, October 1992.

217. Mactec Data Report Attachment H – Kd Distribution Coefficient Test Results consists of: Report by Savannah River National Laboratory, WSRC-STI-2007-00244, May 2007.

218. Mactec, Engineering and Consulting, Results of Geotechnical Exploration and Testing, SCE&G COL Project, V. C. Summer Nuclear Station, submitted to Bechtel Power Corporation, August 2006.

219. Miller, James A., *Ground Water Atlas of the United States*, Segment 6, Alabama, Florida, Georgia, and South Carolina, U.S. Geological Survey, Hydrologic Investigations Atlas 730-G.

220. National Climate Data Center (NCDC), *Cooperative Summary of the Day, TD3200, Period of Record through 2001*, (Includes Daily Weather Data

from the Eastern United States, Puerto Rico, and the Virgin Islands), Version 1.0 (CD-ROM), November 2002.

221. Craig, R. F., *Soil Mechanics*, 6th Edition, Spon Press, May 1997.

222. SCDHEC: Data sent on September 11, 2006 by Erica Johnson of Water Quality Modeling, SC DHEC [johnsoea@dhec.sc.gov, (803) 898-4166] to Steve Summer in response to the August 29, 2006 FOI request #0608292.

223. SCE&G, *Dam Safety Analysis for Fairfield Pumped Storage Facility*, FERC Project No, 1894, prepared by Post, Buckley, Schuh and Jernigan, Inc., 1990.

224. SCE&G, *Preliminary Safety Analysis Report: Virgil C. Summer Nuclear Station, Unit 1*, 1971.

225. SCE&G: *Amended Application of South Carolina Electric & Gas Company*, Project No. 1894, July 1972.

226. SCE&G, *Updated Final Safety Analysis Report: Virgil C. Summer Nuclear Station*, Unit 1, 2005.

227. SCE&G, V. C. Summer Nuclear Station Relicensing Environmental Report, August 2002.

228. South Carolina Department of Environmental Control (SCDHEC), South Carolina, Correspondence Letter *RE: VC SUMMER PLANT*, August 10, 2006.

229. USGS, *Surface-Water Data for the Surface-Water Data for the Nation*. Available at http://waterdata.usgs.gov/usa/nwis/sw. Accessed April 24, 2006.

230. USGS, Digital elevation maps (DEM) 1:24,000 for South Carolina purchased in 2007 from GeoCommunity (http://data.geocomm.com), an official USGS distributor of SDTS format DEMs.

231. U.S. Army Corps of Engineers Bulletin 52-8, EM 1110-2-1411.

232. U.S. Army Corps of Engineers, Coastal Hydraulics Laboratory, EM1110-2-1100, Coastal Engineering Manual, October 2005.

233. U.S. Army Corps of Engineers, Cold Region Research and Engineering Laboratory (CRREL), *Ice Jam Database*. Available at http://www.crrel.usace.army.mil/ierd/ijdb/, accessed July 9, 2007.

234. U.S. Army Corps of Engineers, *Dam Free Board Requirements, Engineering and Design - Hydrologic Engineering Requirements for Reservoirs*, EM 1110-2-1420, October 1997.

235. U.S. Army Corps of Engineers, *Flood-Runoff Analysis*, Engineering Manual 1110-2-1417, August 1994.

236. U.S. Army Corps of Engineers: HEC-RAS, *River Analysis System Hydraulic Reference Manual*, Hydrologic Engineering Center (HEC), Davis, California, November 2002.

237. U.S. Army Corps of Engineers, *HEC-1 Flood Hydrograph Package*, User's Manual, Version 4.0, September 1990.

238. U.S. Army Corps of Engineers, *Hydrologic Modeling System, HEC-HMS Model*, Version 3.0.1, User's Manual, April 2006.

239. U.S. Army Corps of Engineers: *National Inventory of Dams*, updated in February 2005. Available at http://crunch.tec.army.mil/nid/webpages/nid.cfm, accessed on August 6, 2006.

240. U.S. Army Engineer Division, Ohio River, Main Report Development of Water Resources in Appalachia, Part III, Project Analyses, Chapter 5, Cincinnati, Ohio, 1969.

241. Not used.

242. U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA), *Probable Maximum Precipitation Estimates – United States East of the 105th Meridian*, Hydrometeorological Report No. 51, August 1982.

243. U.S. Department of Commerce, NOAA, *Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian*, Hydrometeorological Report No. 52, August 1982.

244. U.S. Department of Commerce, NOAA, Hydrometeorological Report No. 53, *Seasonal Variation of 10-Square-Mile Maximum Precipitation Estimates – United States East of the 105th Meridian*, August 1982.

245. Not used.

246. U.S. Environmental Protection Agency, *Safe Drinking Water Information System (SDWIS)*. Available at http://www.epa.gov/enviro/html/sdwis/sdwis_query.html. Accessed August 16, 2006.

247. U.S. NRC, *Postulated Radioactive Releases Due To Liquid-Containing Tank Failures*, Standard Review Plan, NUREG 0800, Branch Technical Position 11-6, March 2007.

248. U.S. NRC, *Historical Extreme Winds for the United States – Atlantic and Gulf of Mexico Coastlines*, NUREG/CR-2639, May 1982.

249. Not used.

250. U.S. Weather Bureau, *Seasonal Variation of the Probable Maximum Precipitation, East of the 105th Meridian for Area from 10 to 100 Square Miles and Durations of 6, 12, 24, and 48 hours*, Hydrometeorological Report No. 33, 1956.

251. Warner, R., Meadows, J., Sojda, S., and Fleisher, C., 2004 - *Petrography and Uranium Mineralogy of a Deep Well Core at Jenkins Bridge Road near Simpsonville*, SC Proceedings of the twelfth annual David S. Snipes/Clemson Hydrogeology Symposium, April 18, 2004.

252. Westinghouse: *Liquid Tank Rupture Evaluation (COL Item 15.7.6)*, AP1000 Calculation Note Number APP-WLS-M3C-021, 2006.

253. Butler, A., S*outh Carolina Water Use Report 2006 Summary*, South Carolina, Department of Environmental Control, Bureau of Water, July 2007.

Table 2.4-201
Stream Flow Gauging Stations

Station name		Alston [Reference 209]	Richtex [Reference 229]	Carlisle [Reference 209]
USGS station number		02161000	02161500	02156500
Latitude		34°14'35"	34°11'05"	34°35'46"
Longitude		81°19'11"	81°11'48"	81°25'20"
Distance from Parr Shoals Dam	mi	1.2 downstream	14 downstream	21 upstream
Period of record		October 1896 to December 1907	October 1925 to July 1928	October 1938 to current year
		October 1980 to current year	October 1929 to September 1983	
Remarks		Records good except for estimated daily discharges, which are poor. Records for the 1897–1908 water years are poor. Regulation at low and medium flow by power plants above station.	Discontinued in 1983.	Records good except for estimated daily discharges, which are poor. Some regulation at low and medium flow by power plants above station. Capacity of reservoirs insufficient to affect monthly figures of runoff.
Drainage area	sq mi	4,790	4,850	2,790
Water years		1897–1906 & 1980–2005	1925-83	1939–2005
Annual mean	cfs	6,302	6,155	3,880
Highest annual mean	cfs	11,750	—	5,977
Lowest annual mean	cfs	2,153	—	1,255
Highest daily mean	cfs	130,000	211,000	114,000
Lowest daily mean	cfs	49	149	44
Annual 7-day minimum	cfs	200	—	220
Maximum peak flow	cfs	~140,000 (on 6-7-1903)	228,000 (on 10-3-1929)	~123,000 (on 10-10-1976)
Annual runoff	in	17.67	—	18.89

Source: **Reference 209** and **Reference 229**.

Table 2.4-202
Monticello Reservoir Area and Storage Capacity Curves Data

Elevation Feet NGVD29	Area Acres	Incremental Volume Acre-Feet	Cumulative Volume Acre-Feet
270	37		
		870	
280	137		870
		2,080	
290	279		2,950
		3,650	
300	451		6,600
		5,550	
310	649		12,150
		7,960	
320	943		20,110
		10,920	
330	1,242		31,030
		14,620	
340	1,682		45,650
		19,160	
350	2,150		64,810
		24,440	
360	2,730		89,250
		30,250	
370	3,320		119,500
		36,200	
380	3,920		155,700
		42,200	
390	4,520		197,900
		48,400	
400	5,160		246,300
		55,200	
410	5,880		301,500
		61,550	
420	6,430		363,050
		68,000	
430	7,170		431,050

Source: Reference 225

Table 2.4-203
Parr Reservoir Area and Storage Capacity Curves Data

Elevation Feet NGVD29	Area Acres	Incremental Volume Acre-Feet	Cumulative Volume Acre-Feet
253	0		
		800	
255	800		800
		2,733	
257	1,850		3,533
		6,638	
260	2,727		10,171
		17,150	
265	4,116		27,321
		23,795	
270	5,402		51,116

Source: Reference 225

Table 2.4-204 (Sheet 1 of 4)
Reservoirs Located in the Broad River Watershed

Rank	Dam Name	State	County	River	Owner		Agency	Dam Data					Reservoir		Drainage Area	Longitude	Latitude
					Name	Type[a]		Year	Type[b]	Length	Height	Hydr. Height	Storage	Area			
										feet	feet	feet	acre-feet	acres	sq. mi.	deg.	deg.
1	Dam A, B, C and D (Frees Creek Dams)[c]	SC	Fairfield	Frees Cr. Broad River	South Carolina Electric & Gas Company	U	FERC	1977	RE	11,130		169	400,000	6,650	17	–81.33	34.32
2	Moss Lake Dam	NC	Cleveland	Buffalo Creek	City Of Kings Mountain	U	NC	1973	RE	840		85	53,280	1,329		–81.46	35.28
3	Lake Robinson Dam	SC	Greenville	South Tyger River	Commission of Pub Works	L	SC	1984	REPG	986	77		45,000	802	49	–82.30	34.99
4	Lake Lure Dam	NC	Rutherford	Rocky Broad River	Town Of Lake Lure	U	NC	1927	CNVA	480		122	44,914	740	95	–82.18	35.43
5	Parr Shoals Dam	SC	Newberry/Fairfiel d	Broad River	South Carolina Electric & Gas Company	U	FERC	1914	PGRE	2,715	52	45	32,000	3,550	4,750	–81.33	34.27
6	H Taylor Blalock Res Dam	SC	Spartanburg	Pacolet River	Comm Pub Wks-Spartanburg	L	SC	1983	REPG	1,000	72		23,000	1,050	276	–81.87	35.05
7	Lake Summit Dam (Duke FERC)	NC	Henderson	Green River	Duke Power Company	P	NC		CNCB				15,840	0		–82.40	35.23
8	N Tyger R Wcd Dam No2	SC	Spartanburg	Jordan Creek	SJWD Water District	L	SC	1976	RE	827	50		6,800	330	9	–82.10	34.99
9	South Pacolet Riverres 1	SC	Spartanburg	South Pacolet	Spartanburg Water System	L	SC	1926/ 1955	CB	449	74		6,242	182	93	–81.97	a35.11
10	Bvrdam Warrier Crk Wcd 1m	SC	Laurens	Beaverdam Creek	Bvrdam-Warrior Ck WCD	L	SC	1976	RE	1,460	40		5,800	105	9	–82.08	34.64
11	Chesterres Dam	SC	Chester	Sandy River	Lake Mcgregorinc	L	SC	1928	RE	574	42		4,130	111	17	–81.26	34.71
12	Second Broad Watershed Structure #2	NC	Rutherford	Cathey's Creek	Rutherford Co.Watershed Comm.	L	NC	1995	RE	585		26	3,360	41	7	–81.98	35.50
13	Gaston Shoals Upper	SC	Chreokee	Broad River	Duke Power Company	U	FERC	1908	CNPG	707	45		2,500	251	1,250	–81.60	35.14
14	Thicketty Creek Wcd #26	SC	Cherokee	Thicketty Creek	SC Dept Of Natural Resources	S	SC	1967/ 1982	RE	901	50		2,431	100	6	–81.78	35.08
15	Lockhart Dam	SC	Chester/Union	Broad River	Lockhart Power Company	P	FERC	1921	PG	1,299	16	16	2,400	300	2,600	–81.46	34.80
16	Sjwd Waterdist Rcc Dam	SC	Spartanburg	North Tyger River	SJWD Water District	O	SC	1997	PG	650	44		2,400	137	25	–82.06	34.94
17	Lockhart West Canal Embankment	SC	Union	Broad River	Lockhart Power Company	P	FERC	1920	RE	7,350	20	18	2,400	300		–81.46	34.79
18	Ninety Nine Islands	SC	Cherokee	Broad River	Duke Power Company	U	FERC	1910	CNPG	1,568	62		2,300	433	1,550	–81.49	35.03
19	Bvrdam-Warrior Ck Wcd Dm5	SC	Laurens	Warrior Creek	Byrds Lawn & Lands Inc	L	SC	1980	RE	1,300	43		2,255	15	10	–82.04	34.61
20	Browns Creek Wcd Dam #2	SC	Union	Browns Creek	Browns Creek Wcd Dam #2	L	SC	1973	RE	1,039	44		2,229	35	5	–81.56	34.77
21	Lake John D Long	SC	Union	Hughes Creek	SC Dept Of Natural Resources	S	SC	1978	RE	838	45		2,109	81	4	–81.51	34.77

Table 2.4-204 (Sheet 2 of 4)
Reservoirs Located in the Broad River Watershed

Rank	Dam Name	State	County	River	Owner Name	Owner Type[a]	Agency	Dam Data Year	Dam Data Type[b]	Dam Data Length feet	Dam Data Height feet	Dam Data Hydr. Height feet	Reservoir Storage acre-feet	Reservoir Area acres	Drainage Area sq. mi.	Longitude deg.	Latitude deg.
22	Summer Cat I Emergency Cooling Water	SC	Fairfield	Frees Creek - Os	South Carolina Electric & Gas Company	U	US NRC	1979	BLANK	1,500	129		1,600	36		–81.32	34.30
23	Neal Shoals	SC	Chester/Union	Broad River	South Carolina Electric & Gas Company	U	FERC	1905	CN	1,087	25	25	1,492	550	2,730	–81.45	34.67
24	Dam No. 19 D-3406	SC	Cherokee	Thicketty Creek	Thicketty Creek Wcd	L	USDA NRCS	1970	RE	734	45		1,446	19	4	–81.74	35.09
25	Second Broad Water Shed #13	NC	Rutherford	Mill Creek	Rutherford Co W Shed Commission	L	NC	1990	RE	935		18	1,269	22	3	–82.01	35.48
26	Berry Shoals Pond Dam	SC	Spartanburg	South Tyger River	Bluestone Enerdesign Inc	P	SC	1920	OTPG	300	29	0	1,128	60	106	–82.10	34.89
27	Lake Sheila Dam	NC	Henderson	Pacolet River-Tributary	Grover Haynes Lake Sheila Poa	P	NC	1964	RE	400		49	1,024	40		–82.37	35.19
28	Second Broad River W/S, Site 23	NC	Rutherford	Stoney Mine Creek	Rutherford Co. Watershed Commission	L	NC	1980	RE	945	57		998	0	2	–81.98	35.52
29	W R Grace Dam 1	SC	Laurens	Tr-Warrier Creek	W R Grace & Co	P	SC	1989	RE	1,800	93		930	116		–81.99	34.61
30	Bvrdam-Warrior Ck Wcd Dm2	SC	Laurens	Wallace Branch	Bvrdam-Warrior Ck Wcd	L	SC	1974	RE	1,360	44		808	15	2	–82.06	34.65
31	Bverdm Warrior Wcd Dam 33	SC	Laurens	Tr-Strouds Branch	Beaverdam Warrior Crk Wcd	L	SC	1977	RE	670	34		800	20	2	–81.96	34.59
32	Duncan Creek Wcd Dam 7	SC	Laurens	Sand Creek	Duncan Creek Wcd Dam 7	L	SC	1963	RE	695	37		773	25	3	–81.83	34.49
33	2nd Broad River Watershed #23	NC	Rutherford	Stoney Creek	Rutherford County Watershed	L	NC	1979	RE	949		12	770	12	2	–81.98	35.52
34	Pat Hartness Dam	SC	Spartanburg	Tr-Enoree River	Hartness International	P	SC	1998	RE	1,135	37		750	40	1	–82.00	34.67
35	Startex Mill Dam #1	SC	Spartanburg	Middle Tyger River	Spartan Mills-Startex Div	P	SC	1890	OT	280	30		720	50	57	–82.10	34.93
36	Kings Mountain City Lake #2	NC	Cleveland	Kings Creek-Tr	City Of Kings Mountain	U	NC	1954	RE	409		54	714	30	1	–81.35	35.19
37	2nd Broad River Watershed #22	NC	Rutherford	Hox Creek	Rutherford County Watershed Di	L	NC	1978	RE	512		26	710	10		–81.96	35.51
38	Nabors Pond	SC	Laurens		W R Grace	P	DOL MSHA				25		687	0		–81.99	34.61
39	Bvrdam Warrior Crk Wcd #4	SC	Laurens	Tr-Warrior Creek	Bvrdam-Warrior Ck Wcd	L	SC	1974	RE	888	35	0	671	21	3	–82.01	34.59
40	Houser Lake Dam	NC	Cleveland	Broad River-Os	Yates Houser	P	NC	1970	RE	340		45	660	30		–81.73	35.22
41	South Tyger River Wcd 4c	SC	Greenville	Tr-Mush Creek	S Tyger River Wcd	L	SC	1974	RE	665	28		619	22	3	–82.38	35.05
42	2nd Broad River Watershed #16	NC	Rutherford	Mountain Creek	Rutherford County Watershed	L	NC	1979	RE	600		23	593	6		–81.85	35.49

Table 2.4-204 (Sheet 3 of 4) Reservoirs Located in the Broad River Watershed

Rank	Dam Name	State	County	River	Owner		Agency	Dam Data					Reservoir		Drainage Area	Longitude	Latitude
					Name	Type(a)		Year	Type(b)	Length	Height	Hydr. Height	Storage	Area			
										feet	feet	feet	acre-feet	acres	sq. mi.	deg.	deg.
43	Clinton Mill Pond Dam D-2989	SC	Laurens	Tr-Beards Fork Creek	Clinton Cotton Mill	P	USDA NRCS	1968	RE	425	25		591	0	3	–81.91	34.49
44	Styger Ruver Wcd Dam #2c	SC	Greenville	Meadow Fork Creek	North Greenville Jr Co	P	SC	1969	RE	424	26		583	12	3	–82.38	35.07
45	BSA Piedmont Council “A” Dam	NC	Rutherford	Second Broad River	David Allen	P	NC	1981	RE	450		45	540	20		–81.92	35.45
46	Lake Wineemoko Dam	SC	Union	Mcclure Creek	Lake Winemoko Owners Assn	P	SC		RE	500	35		500	35	2	–81.70	34.79
47	Second Broad Watershed #14	NC	Rutherford	Fork Creek	Rutherford Co. Watershed Comm.	L	NC	1982	RE	400		30	480	13	2	–81.86	35.51
48	Muddy Creek #4	NC	Mcdowell	Goose Creek	Mcdowell S&Wcd	L	NC	1976	RE	352		24	456	9		–81.99	35.61
49	Kings Mountain Lake Dam #1	NC	Cleveland	King Creek	Mr. Jimmy Maney, City Of Kings Mountain	U	NC	1929	CB	195		29	450	30		–81.35	35.20
50	Duncan Creek Wcd Dam 8	SC	Laurens	Tr-Sand Creek	Duncan Creek Wcd Dam 8	L	SC	1963	RE	778	41		438	10	2	–81.84	34.49
51	Hooper Creek Dam	NC	Polk	Hooper Creek	Red Fox Country Club.	P	NC	1965	RE	465		17	400	25		–82.14	35.20
52	Foote Mineral Tailings Dam (Breached)	NC	Cleveland	Mill Creek	Cyprus-Foote Mineral Co	P	NC	1958	RE	4,400		50	384	12		–81.35	35.21
53	2nd Broad River Ws #4	NC	Rutherford	Cherry Creek	Rutherford Co Ws Commission	L	NC	1987	RE	760		16	375	24		–81.95	35.45
54	Sandy Plains Dam	NC	Polk	Hughes Creek	Barbara C. Martin	P	NC		RE	522		19	350	20		–82.10	35.24
55	City Of Jonesville Dam	SC	Union	Eison Branch	City Of Jonesville	L	SC	1969	RE	610	29		345	26	1	–81.68	34.86
56	Una S Johnson Dam	SC	Cherokee	Tr-Broadriver	Johnson, Una S	P	SC	1958	RE	300	35		331	18		–81.48	34.88
57	Lake Saranac Dam	SC	Spartanburg	Mineral Spring Branch	Robertson, Sarah Lucille	P	SC	1962	RE	400	54		304	15		–81.87	34.99
58	Lake Saranac Dam	SC	Spartanburg	Mineral Springs Br.	Cecil Osmith Est	P	SC	1959	RE	340	54		304	17	1	–82.02	34.90
59	Bogan Dam	SC	Union	Swink Creek	Bogan, Robert	P	SC	1971	RE	1,500	30		300	25	1	–81.68	34.82
60	Daves Pond Dam	SC	York	Tr-Broadriver	Daves, Gene & Nancy C	P	SC	1998	RE	650	64		300	10		–81.46	34.93
61	Cedar Lake Dam	NC	Cleveland	Little Buffalo Creek	Ron Chidester	P	NC	1965	RE	851		32	300	20		–81.46	35.40
62	Bald Mountain Lake	NC	Rutherford	Buffalo Creek	Fairfield Mountains Poa (Tim Gordon)	P	NC	1960	RE	500		50	288	50	5	–82.19	35.46
63	Murray Hilton Lake	NC	Rutherford	South Creek	Pioneer Girl Scout Council	P	NC	1963	RE	534		38	274	10		–81.80	35.51
64	Shagreen Nursery Dam	NC	Cleveland	First Broad River-Tr	Emile Gebel, Md	P	NC	1993	RE	310		30	266	18	3	–81.55	35.33
65	Bailey Dam	SC	Laurens	Tr-Beards Fork Creek	Dixon, Bailey	P	SC		RE	420	26		260	17		–81.91	34.49
66	Lake Emory Dam	SC	Spartanburg	Greene Creek	Brock Realty	P	SC	1997	RE	900	20		256	20	3	–82.06	35.04
67	Clifton No. 3	SC	Spartanburg	Pacolet R	Bluestone Energy Design, Inc.	P	FERC	1899	PGMS	375	33	28	250	20	318	–81.83	35.00

Table 2.4-204 (Sheet 4 of 4)
Reservoirs Located in the Broad River Watershed

Rank	Dam Name	State	County	River	Owner		Agency	Dam Data					Reservoir		Drainage Area	Longitude	Latitude
					Name	Type[a]		Year	Type[b]	Length	Height	Hydr. Height	Storage	Area			
										feet	feet	feet	acre-feet	acres	sq. mi.	deg.	deg.
68	Duncan Creek Wcd Dam 10	SC	Laurens	Saxton Branch	Duncan Creek Wcd	L	SC	1969	RE	554	26		240	13	1	–81.87	34.55
69	Hickory Nut Lower	NC	Rutherford	Broad River	Ecological Development Corp	P	NC	1974	RE	250		40	240	12		–82.17	35.43
70	Sunny Slope Farms Dam	SC	Cherokee	Tr-L. Thick Creek	Sunny Slope Farms	P	SC	1988	RE	700	32		225	18		–81.78	35.04
71	Cecils Pond Dam	SC	Laurens	Tr-Duncan Creek	Loblolly Timberlandscorp	P	SC	1955	RE	580	33		224	16		–81.82	34.54
72	Chestnut Lake Dam	SC	Spartanburg	Tr-Lawsons Fork	Reed, Cullen	P	SC	1975	RE	460	24		224	15		–82.05	35.01
73	Eptings Pond	SC	Newberry	Tr-Crimscreek	Epting, Dale	P	SC	1900	RE	330	31		221	15	1	–81.46	34.22
74	Clifton Mills Pond 2 Dam	SC	Spartanburg	Pacolet River	Clifton Hydro-Pwr Lim	P	SC	1888	OT	340	16		220	25	320	–81.82	34.98
75	Pacolet Mills Pond #2 Dam	SC	Spartanburg	Pacolet River	Milliken & Company	P	SC	1890	OT	370	22		220	25	350	–81.74	34.92
76	Dysart Lake Dam	SC	Greenville	Meadow Fork Creek	Dysart, John O	P	SC	1948	RE	212	32		204	14	1	–82.41	35.08
77	Park Lake Dam	SC	Spartanburg	Tr-Lawsons Fork Ck	Park Lake Inc	P	SC	1962	RE	447	37		202	11		–81.89	34.94
78	BMW Dam 1	SC	Spartanburg	Abner Creek	BMW	P	SC	1994	RE	700	26		200	7	1	–82.17	34.89
79	Cherokee Falls	SC	Cherokee	Broad River	Broad River Electric Coop Inc	P	FERC	1826	MSPG CN	1,850	16	16	200	35	1,490	–81.55	35.06
80	Forest Lake Dam	NC	Rutherford	2nd Broad River-Tr	Joe Robbins	P	NC	1963	RE	475		34	200	10		–81.88	35.36
81	Brooks Lake	NC	Rutherford	Mountain Creek	Lake Brooks Poa	P	NC	1950	RE	465		28	200	16		–82.01	35.44
82	Shumont Estates	NC	Rutherford	Bill S Creek-Tr	Fairfield Mountains Poa,Inc.	P	NC	1990	RE	480		35	200	12		–82.18	35.45

(a) Owner Type
(b) Dam Type
(c) All data in Table 2.4-204 sourced from Reference 239. The data from Reference 239 corrected as per Reference 226.

F = Federal
S = State
L = Local Government
U = Public Utility
P = Private
RE = Earth
ER = Rockfill
PG = Gravity
CB = Buttress

VA = Arch
MV = Multi-Arch
CN = Concrete
MS = Masonry
ST = Stone
TC = Timber Crib
OT = Other

Table 2.4-205
Significant Surface Water Users

User	Water Body	Withdrawal Rate Mg/yr	Withdrawal Rate Mgd
Consumptive Users			
Downstream Water Users:			
Columbia Canal Water Plant	Broad River-Columbia Canal	12,587.46	34.5
W. Columbia Saluda Intake	Saluda River[(a)]	1,208.00	3.3
Martin Marietta Cayce Plant	Congaree River	415.64	1.1
City Cayce Intake #2	Congaree River	1,128.60	3.1
Eastman Chemical Voridian Div.	Congaree River	26,392.68	72.3
Santee Cooper Resort C.C.	Lake Marion	39.54	0.1
St. Julian Plantation	Lake Marion	7.06[(b)]	0.058
Santee Cooper Cross Station	Lake Moultrie	21,794.14	59.7
Ga. Pacific Russellville Plywood	Lake Moultrie (rediversion canal)	112.78	0.3
Santee Cooper Reg. Water	Lake Moultrie	5,071.40	13.9
Amoco Chemical Cooper River Plant	Back River Reservoir	1,983.41	5.4
Bayer Corp. Bushy Park (Sun Chemical)	Back River Reservoir.	876.4	2.4
Charleston CPW Bushy Park	Back River Reservoir	16,871.60	46.2
Chargeurs Wool Prouvost	Santee River	49.8	0.1
SCPSA Winyah Steam Station	N. Santee River	289.7	0.8
Upstream Water Users:			
Spartanburg	S. Pacolet River, Lk Blalock	12,053	33.0
Gaffney	Broad River, Lake Whelchel	2,812	7.7
Nonconsumptive Users			
Columbia Canal Hydro	Broad River-Columbia Canal	469,660.89	1,286.7
Santee Cooper L. Marion Hydro	L. Marion (spillway)	142,890.28	391.5
US Army/St Stephen	L. Moultrie (rediversion canal)	2,079,847[(c)]	5,698.2
Santee Cooper Jeffries Hydro	L. Moultrie	1,108,728.73	3,037.6
SCE&G A.M. Williams Station	Back River Reservoir	191,813.00	525.5

(a) Intake is in the confluence of the Saluda and Broad and at times does receive water from the Broad River
(b) For 4 months only
(c) Flow computed from daily mean discharge at USGS 02171645

Table 2.4-206
Major Historic Floods and Peak Flows in the Broad River near the Site

Date	Observed at Richtex[(a)] or Alston[(b)] Station		Estimated at Parr Shoals Dam[(c)]	
	Maximum Discharge (cfs)	Water Elevation (feet, NGVD29)	Peak Discharge (cfs)	Water Elevation[(d)] (feet NGVD29)
10/3/1929	228,000 [(a)]	215.54[(e)]	223,299	266.2
8/17/1928	222,000 [(a)]	214.94[(e)]	217,423	266.1
4/8/1936	157,000 [(a)]	209.80[(e)]	153,763	264.2
10/11/1976	146,000 [(a)]	208.54	142,990	263.9
8/16/1940	120,000 [(a)]	205.94	117,526	263.0
10/18/1964	102,000 [(a)]	204.14	99,897	262.4
10/18/1932	101,000 [(a)]	204.04	98,918	262.4
10/14/1990	119,000 [(b)]	238.81	118,006	263.0
3/3/1987	108,000 [(b)]	237.51	107,098	262.7
8/30/1995	99,100 [(b)]	236.82	98,272	262.4

(a) Recorded in Broad River at Richtex USGS gauging station No. 02161500 (drainage area: 4,850 square miles).
(b) Recorded in Broad River at Alston USGS gauging station No. 02161000 (drainage area: 4,790 square miles).
(c) Peak values at Parr Shoals Dam (drainage area: 4750 square miles) are estimated based on drainage area ratios.
(d) Based on ogee spillway crest elevation at 257 feet (NGVD29) and gates opened.
(e) Data obtained from the PSAR for the Unit 1 [Reference 224].

Table 2.4-207
Six-Hour Local PMP

Duration	PMP Depth (inches)
5 minutes	6.2
15 minutes	9.7
30 minutes	14.1
1 hour	19.0
6 hours	30.4

Table 2.4-208
PMP for Broad River Watershed at Richtex

Time (hours)	Depth (inches)
6	9.1
12	12.7
24	15.9
48	19.6
72	22.1

Table 2.4-209
Distribution of 72-Hour PMP for Broad River Watershed at Richtex

Time (Hour)	Depth (inches)
0 to 6	0.52
6 to 12	0.57
12 to 18	0.62
18 to 24	0.71
24 to 30	1.33
30 to 36	3.60
36 to 42	9.08
42 to 48	1.90
48 to 54	1.14
54 to 60	0.95
60 to 66	0.85
60 to 72	0.81

Table 2.4-210
PMP for Frees Creek Watershed

Duration (hour)	PMP Depth (inches)
6 hours	30.0
12 hours	35.6
24 hours	40.6
48 hours	45.0
72 hours	48.6

Table 2.4-211
Storage Volumes of Existing and Proposed Reservoirs Upstream of Parr Shoals Dam on Broad River

	Storage (acre-feet)		
Reservoir	**From Reference 226**	**From Reference 239**	**Assumed Maximum Storage (acre-feet)**
Clinchfield	1,275,000[a]	—	1,275,000
Houser Lake	—	660	660
Gaston Shoals	1,150	2,500	2,500
Cherokee Fall	—	200	200
Ninety-Nine Islands	4,130	2,300	4,130
Daves Pond	—	300	300
Una S Johnson	—	331	331
Lockhart	—	2,400	2,400
Neal Shoals	—	1,492	1,492
Parr	31,500[b]	—	31,500
		Total	**1,318,513**

(a) Maximum storage at El. 830 feet NGVD (Reference 240)
(b) Maximum storage at El. 266 feet NGVD (Reference 226)

Table 2.4-212
Reported Water Use in South Carolina, 2005 (in Millions of Gallons)

Water Use Category	Groundwater	Surface Water	Total	Groundwater Percentage	Surface Water Percentage	Percentage of Total Water Use
Consumptive and Nonconsumptive Use						
Aquaculture	182.93	227.37	410.30	44.6%	55.4%	0.0020%
Golf Courses	3,099.41	8,808.68	11,908.09	26.0%	74.0%	0.0583%
Industrial	11,830.92	14,0255.88	152,086.80	7.8%	92.2%	0.7445%
Irrigation	14,065.22	7,858.81	21,924.03	64.2%	35.8%	0.1073%
Mining	2,709.77	595.40	3,305.17	82.0%	18.0%	0.0162%
Other	105.63		105.63	100.0%	0.0%	0.0005%
Hydroelectric	0.33	15,766,866.75	15,766,867.08	0.0%	100.0%	77.1793%
Thermoelectric	2,043.32	4,254,461.12	4,256,504.44	0.0%	100.0%	20.8357%
Public Water Supply	38,113.35	177,657.70	215,771.05	17.7%	82.3%	1.0562%
Totals	**72,150.88**	**20,356,731.71**	**20,428,882.59**	**0.4%**	**99.6%**	**100.0%**
Consumptive Use Only						
Aquaculture	182.93	227.37	410.30	44.58%	55.42%	0.1012%
Golf Courses	3,099.41	8,808.68	11,908.09	26.03%	73.97%	2.9366%
Industrial	11,830.92	140,255.88	152,086.80	7.78%	92.22%	37.5050%
Irrigation	14,065.22	7,858.81	21,924.03	64.15%	35.85%	5.4065%
Mining	2,709.77	595.40	3,305.17	81.99%	18.01%	0.8151%
Other	105.63		105.63	100.00%	0%	0.0260%
Public Water Supply	38,113.35	177,657.70	215,771.05	17.66%	82.34%	53.2097%
Totals	**70,107.23**	**335,403.84**	**405,511.07**	**17.30%**	**82.7%**	**100.0%**

Source: Modified from South Carolina Water Use Report – 2005 (Reference 207)

Table 2.4-213
Reported Water Use in Fairfield County, 2005

Water Use in Millions of Gallons				Percent of Total Water Use	
Groundwater Use		Surface Water Use		Groundwater	Surface Water
Consumptive and Nonconsumptive Use					
Aquaculture	—	Aquaculture	—	—	—
Golf Course	—	Golf Course	—	—	—
Hydroelectric	*	Hydroelectric	2,944,701.12	—	100%
Industrial	—	Industrial	—	—	—
Irrigation	—	Irrigation	—	—	—
Mining	—	Mining	—	—	—
Nuclear Power	*	Nuclear Power	561,096.95	—	100%
Public Water Supply	67.82	Public Water Supply	586.29	10.4%	89.6%
Other	—	Other	—	—	—
Total	67.82		3,506,384.36	0.002%	99.998%
Consumptive Use Only					
Aquaculture	—	Aquaculture	—	—	—
Golf Course	—	Golf Course	—	—	—
Industrial	—	Industrial	—	—	—
Irrigation	—	Irrigation	—	—	—
Mining	—	Mining	—	—	—
Public Water Supply	67.82	Public Water Supply	586.29	10.4%	89.6%
Other	—	Other	—	—	—
Total	67.82		586.29	10.4%	89.6%

Source: Modified From South Carolina Water Use Report – 2005 (Reference 207)
— = Not Reported
(*) = Not listed in source table

Table 2.4-214
Reported Water Use in Newberry County, 2005

Water Use in Millions of Gallons				Percent of Total Water Use	
Groundwater Use		Surface Water Use		Groundwater	Surface Water
Consumptive Use Only Reported					
Aquaculture	—	Aquaculture	—	—	—
Golf Course	12	Golf Course	6	66.7%	33.3%
Hydroelectric	*	Hydroelectric	—	—	—
Industrial	—	Industrial	—	—	—
Irrigation	55.23	Irrigation	122.5	31.1%	68.9%
Mining	—	Mining	—	—	—
Thermal Power	*	Thermal Power	—	—	—
Public Water Supply	21.63	Public Water Supply	1,928.53	1.1%	98.9%
Other	—	Other	—	—	—
Total	88.86		2,057.03	4.14%	95.86%

Source: Modified From South Carolina Water Use Report – 2005 [Reference 207].
— = Not Reported
(*) = Not listed in source table.

Table 2.4-215
Public Water Supply Wells within 6 Miles of Units 2 and 3, South Carolina

Water System Name	Water System ID	Population Served[a]	Well ID	Status	Depth of Well (feet)	Design Yield (gpm)	Aquifer	County
JENKINSVILLE WATER DIST	2020001	1969	G20109	Active	Unknown	unknown	Piedmont Bedrock	Fairfield
			G20105	Active	246	20	Piedmont Bedrock	Fairfield
			G20104	Active	360	9	Piedmont Bedrock	Fairfield
SCE&G HWY 99 BOAT RAMP	2070913	26	G20167	Active	92	7	Piedmont Bedrock	Fairfield
SCE&G MONTICELLO REC	2070676	26	G20114	Active	259	20	Piedmont Bedrock	Fairfield
SCE&G MONTICELLO REC	2070677	26	G20115	Active	246	20	Piedmont Bedrock	Fairfield
SCE&G PARR STEAM PLANT	2030005	unknown	G20159	Active	365	29	Piedmont Bedrock	Fairfield
EDCON WAREHOUSE	3670910	unknown	G36178	Active	305	5	Piedmont Bedrock	Newberry
GATEWAY MHP	3660006	25	G36128	Active	350	unknown	Piedmont Bedrock	Newberry
			G36127	Active	180	unknown	Piedmont Bedrock	Newberry
			G36129	Active	350	unknown	Piedmont Bedrock	Newberry
			G36130	Inactive	350	unknown	Piedmont Bedrock	Newberry
H.J. SMITH PROPERTIES	3670903	25	G36172	Active	246	20	Piedmont Bedrock	Newberry
SHEALY MHP	3660001	25	G36124	Inactive	125	12	Piedmont Bedrock	Newberry
THE QUE STICK	3670211	40	G36155	Active	705	5	Piedmont Bedrock	Newberry
WEBER MHP	3660010	26	G36134	Active	unknown	unknown	Piedmont Bedrock	Newberry

(a) Reference 246.

Table 2.4-216 (Sheet 1 of 2)
Observation Well Details

Well ID	Northing[(a)]	Easting[(a)]	GS Elevation (feet)	Top of Casing Elevation (feet)	Total Well Depth (feet bgs)	Screen Interval Depth (feet)	Screen Interval Elevation (feet)	Top of Filter Pack (feet bgs)	Top of Filter Pack Elevation (feet)	Formation of Screen Interval
OW-205a	892829.3	1903189.8	423.3	425.9	110.0	98.5–108.5	324.8–314.8	80.0	343.3	Sound Rock
OW-205b	892842.4	1903192.5	422.9	425.0	60.0	54.9–59.9	368.0–363.0	49.9	373.0	Partially Weathered Rock
OW-212	893105.1	1903036.8	396.2	399.3	68.0	56.0–66	340.2–330.2	53.0	343.2	Saprolite/Partially Weathered Rock
OW-213	892975.6	1903457.3	402.1	404.5	55.25	44.75–54.75	357.3–347.3	41.5	360.6	Saprolite
OW-227	892494.0	1903408.0	422.7	425.1	84.25	71.25–81.25	351.4–341.4	67.0	355.7	Bedrock
OW-233	892786.5	1902693.4	426.2	428.3	120.0	99.0–119	327.2–307.2	74.0	352.2	Bedrock
OW-305a	892008.7	1902841.2	424.9	427.8	141.0	119.5–139.5	305.4–285.4	95.0	329.9	Sound Rock
OW-305b	891996.7	1902857.5	423.7	426.3	66.5	54.5–64.5	369.2–359.2	51.0	372.7	Partially Weathered Rock/Sound Rock
OW-312	892256.5	1902709.6	425.1	427.1	36.5	30.5–35.5	394.6–389.6	26.4	398.7	Saprolite/Partially Weathered Rock
OW-313	892167.6	1903132.5	420.9	423.8	59.0	48.0–58	372.9–362.9	44.1	376.8	Saprolite/Partially Weathered Rock
OW-327	891669.2	1903084.1	410.7	413.4	66.0	55.0–65	355.7–345.7	51.5	359.2	Partially Weathered Rock
OW-333	891954.4	1902319.6	394.5	397.1	71.0	60.0–70	334.5–324.5	52.0	342.5	Sound Rock
OW-401a	891017.8	1903595.5	404.1	406.3	92.5	80.0–90	324.1–314.1	76.0	328.1	Sound Rock
OW-401b	891013.1	1903585.0	404.1	406.8	66.0	60.0–65	344.1–339.1	57.0	347.1	Saprolite/Partially Weathered Rock
OW-405	890180.4	1903650.2	392.6	395.4	58.5	44.0–54	348.6–338.6	41.0	351.6	Partially Weathered Rock
OW-501	897817.4	1903702.3	429.5	431.9	32.0	20.0–30	409.5–399.5	17.5	412.0	Fill/Residual Soil
OW-612	892415.5	1904227.3	406.8	409.4	62.0	47.5–57.5	359.3–349.3	44.5	362.3	Saprolite
OW-614	891671.1	1903536.1	376.1	379.1	33.0	21.5–31.5	354.6–344.6	18.5	357.6	Saprolite
OW-617	889886.3	1902373.7	447.2	450.1	108.0	98.0–108	349.2–339.2	93.0	354.2	Partially Weathered Rock
OW-618	890955.6	1901480.1	307.4	310.5	32.5	18.5–28.5	288.9–278.9	13.8	293.6	Saprolite
OW-619	892594.0	1901843.9	405.7	407.7	104.0	83.0–103	322.7–302.7	77.5	328.2	Bedrock

Table 2.4-216 (Sheet 2 of 2)
Observation Well Details

Well ID	Northing[a]	Easting[a]	GS Elevation (feet)	Top of Casing Elevation (feet)	Total Well Depth (feet bgs)	Screen Interval Depth (feet)	Screen Interval Elevation (feet)	Top of Filter Pack (feet bgs)	Top of Filter Pack Elevation (feet)	Formation of Screen Interval
OW-620	893593.8	1903017.2	382.8	385.0	91.0	76.5–86.5	306.3–296.3	74.0	308.8	Partially Weathered Rock
OW-621a	893732.7	1903676.2	420.9	423.5	97.3	84.5–94.5	336.4–326.4	80.0	340.9	Sound Rock
OW-621b	893742.6	1903677.8	421.2	423.6	71.0	60.0–70	361.2–351.2	55.0	366.2	Saprolite/Partially Weathered Rock
OW-622	894292.2	1904118.1	438.1	440.7	62.0	48.5–58.5	389.6–379.6	44.5	393.6	Bedrock
OW-623	893819.9	1904946.1	439.6	441.8	90.0	76.5–86.5	363.1–353.1	72	367.6	Bedrock
OW-624	891595.7	1904623.8	359.3	361.6	62.0	48.5–58.5	310.8–300.8	45	314.3	Bedrock
OW-625	889895.0	1904957.3	403.2	405.9	108.0	84.5–104.5	318.7–298.7	80.5	322.7	Saprolite
OW-626	893202.4	1904129.9	416.4	418.8	85.0	71–81	345.4–335.4	63	353.4	Saprolite
OW-627a	891239.9	1902130.4	327.6	330.3	86.0	66–86	261.6–241.6	64	263.6	Sound Rock
OW-627b	891231.6	1902129.7	326.9	329.5	56.0	43–53	283.9–273.9	37	289.9	Saprolite/Partially Weathered Rock

(a) South Carolina State Plane NAD 83
(b) bgs = below ground surface

Table 2.4-217 (Sheet 1 of 2)
Monthly Groundwater Level Elevations

Well ID	Formation	Hydrostratigraphic Zone	Water Level Elevation												
			2006							2007					
			6-23	7-25	8-30	9-19	10-24	11-29	12-20	1-26	2-20	3-20	4-19	5-23	6-27
OW-205a	Sound Rock	Deep Bedrock	357.3	357.3	357.1	357.2	357.1	357.4	357.5	358.4	358.6	358.9	359.07	359.01	358.97
OW-205b	Partially Weathered Rock	Saprolite/Shallow Bedrock	364.9	365.0	365.2	366.1	366.1	365.3	365.4	365.5	365.7	365.9	366.30	366.85	367.15
OW-212	Saprolite/Partially Weathered Rock	Saprolite/Shallow Bedrock	351.4	351.0	351.2	351.1	350.8	351.6	351.3	352.5	352.8	353.1	352.91	352.75	352.59
OW-213	Saprolite	Saprolite/Shallow Bedrock	359.1	359.1	359.1	359.1	359.0	359.1	359.2	360.3	360.6	361.0	361.10	361.00	360.82
OW-227	Bedrock	Deep Bedrock	361.5	361.3	361.3	361.3	361.3	361.3	361.3	361.4	361.7	362.0	362.29	362.60	362.84
OW-233	Bedrock	Deep Bedrock	322.5	339.9	358.6	362.4	365.2	366.2	366.4	366.9	367.1	367.1	367.30	367.22	367.43
OW-305a	Sound Rock	Deep Bedrock	368.2	368.3	368.1	368.2	368.2	368.3	368.3	368.4	368.5	368.6	368.80	368.99	369.18
OW-305b	Partially Weathered Rock/Sound Rock	Saprolite/Shallow Bedrock	367.4	367.5	367.4	367.4	367.5	367.6	367.5	367.6	367.7	367.8	367.97	368.15	368.35
OW-312	Saprolite/Partially Weathered Rock	Saprolite/Shallow Bedrock	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry
OW-313	Saprolite/Partially Weathered Rock	Saprolite/Shallow Bedrock	372.8	372.7	372.9	373.0	373.2	373.3	373.3	373.1	373.8	374.1	374.51	374.90	375.05
OW-327	Partially Weathered Rock	Saprolite/Shallow Bedrock	359.2	359.1	359.2	359.3	359.4	359.6	359.7	360.0	360.2	360.4	360.75	361.10	361.39
OW-333	Sound Rock	Deep Bedrock	333.8	334.7	335.1	335.1	335.1	334.6	335.0	336.2	337.6	338.5	339.54	339.54	339.05
OW-401a	Sound Rock	Deep Bedrock	351.2	351.0	351.1	351.2	351.5	351.4	351.3	351.7	352.3	352.6	352.92	353.00	352.87
OW-401b	Saprolite/Partially Weathered Rock	Saprolite/Shallow Bedrock	351.0	350.9	351.0	351.0	351.4	351.2	351.1	351.5	352.1	352.4	352.71	352.85	352.72
OW-405	Partially Weathered Rock	Saprolite/Shallow Bedrock	353.8	353.7	353.8	353.9	354.0	353.9	353.8	354.3	354.8	355.2	355.74	355.95	355.95
OW-501	Fill/Residual Soil	Saprolite/Shallow Bedrock	NA	NA	419.1	419.3	418.9	418.1	419.0	418.9	418.6	418.5	418.50	418.90	418.70
OW-612	Saprolite	Saprolite/Shallow Bedrock	357.3	357.2	357.3	357.3	357.4	357.3	357.3	357.6	357.9	358.2	358.55	358.74	358.75
OW-614	Saprolite	Saprolite/Shallow Bedrock	349.9	349.1	349.4	349.2	348.4	350.2	349.4	351.9	351.4	351.7	351.10	350.52	350.00
OW-617	Partially Weathered Rock	Saprolite/Shallow Bedrock	349.3	349.2	349.2	349.1	349.0	348.9	348.9	348.9	348.8	348.7	348.72	348.67	348.67
OW-618	Saprolite	Saprolite/Shallow Bedrock	303.5	303.3	303.6	303.6	303.3	303.8	303.7	304.2	304.2	304.1	304.08	303.54	303.55
OW-619	Bedrock	Deep Bedrock	303.1	303.9	305.6	306.7	308.5	310.3	311.4	313.1	314.4	315.7	317.09	318.63	320.15
OW-620	Partially Weathered Rock	Saprolite/Shallow Bedrock	348.1	347.8	348.0	345.1	347.7	348.2	348.0	348.8	349.0	349.0	348.96	348.70	348.59
OW-621a	Sound Rock	Deep Bedrock	325.9	327.5	328.5	329.0	330.0	330.8	331.2	331.8	332.5	333.1	333.72	334.41	335.09

Table 2.4-217 (Sheet 2 of 2)
Monthly Groundwater Level Elevations

Well ID	Formation	Hydrostratigraphic Zone	Water Level Elevation												
			2006							2007					
			6-23	7-25	8-30	9-19	10-24	11-29	12-20	1-26	2-20	3-20	4-19	5-23	6-27
OW-621b	Saprolite/Partially Weathered Rock	Saprolite/Shallow Bedrock	368.6	368.5	368.7	368.7	368.7	368.8	368.8	369.0	369.4	369.7	370.35	370.83	371.26
OW-622	Bedrock	Saprolite/Shallow Bedrock	394.0	393.9	394.1	394.2	394.2	394.2	394.2	394.2	394.4	394.6	394.82	394.85	394.79
OW-623	Bedrock	Saprolite/Shallow Bedrock	369.7	369.6	369.6	369.7	369.6	369.7	369.7	369.9	370.3	370.7	371.11	371.23	371.22
OW-624	Bedrock	Saprolite/Shallow Bedrock	302.5	307.6	313.5	315.9	317.9	318.8	319.1	319.9	320.2	320.5	320.75	320.68	320.52
OW-625	Saprolite	Saprolite/Shallow Bedrock	316.9	317.1	317.6	318.0	318.4	318.3	318.2	318.7	319.1	319.1	319.34	319.22	319.15
OW-626	Saprolite	Saprolite/Shallow Bedrock	368.9	368.8	368.9	368.9	369.0	369.0	369.0	369.3	369.7	370.1	370.00	370.96	371.15
OW-627a	Sound Rock	Deep Bedrock	258.5	267.5	249.5	249.3	254.8	259.7	262.3	270.7	276.8	282.6	288.24	293.35	297.85
OW-627b	Saprolite/Partially Weathered Rock	Saprolite/Shallow Bedrock	317.4	317.2	317.4	317.3	316.6	317.6	317.3	318.6	318.5	318.4	318.00	317.20	317.20

Table 2.4-218
Slug Test Results

Well Number	Test Interval: Screened Interval (feet bgs)	Test Interval: Hydrostratigraphic Zone	Test Interval: Submerged Screen	Hydraulic Conductivity: Falling Head Test (cm/s)	Hydraulic Conductivity: Rising Head Test (cm/s)	Hydraulic Conductivity: Maximum Test Result (feet/day)
OW-205A	98.5–108.5	Deep bedrock	Fully submerged screen	3.1E-6	Discard	0.0088
OW-212	56–66	Saprolite/Shallow bedrock	Fully submerged screen	8.7E-4	3.6E-4	2.5
OW-213	44.75–54.75	Saprolite/Shallow bedrock	Fully submerged screen	No test	5.9E-4	1.7
OW-227	71.25–81.25	Deep bedrock	Fully submerged screen	4.5E-5	4.4E-5	0.13
OW-305A	119.5–139.5	Deep bedrock	Fully-submerged screen	7.3E-6	6.2E-6	0.021
OW-313	48–58	Saprolite/Shallow bedrock	Partially submerged screen	No test	3.4E-3	9.6
OW-327	55–65	Saprolite/Shallow bedrock	Fully submerged screen	No test	7.1E-5	0.20
OW-333	60–70	Deep bedrock	Partially submerged screen	No test	1.3E-4	0.38
OW-401A	80–90	Deep bedrock	Fully submerged screen	8.2E-5	6.9E-5	0.23
OW-401B	60–65	Saprolite/Shallow bedrock	Fully submerged screen	1.7E-5	1.5E-5	0.047
OW-405	44–54	Saprolite/Shallow bedrock	Fully submerged screen	6.4E-3	4.9E-3	18
OW-612	47.5–57.5	Saprolite/Shallow bedrock	Partially submerged screen	No test	5.0E-4	1.4
OW-617	98–108	Saprolite/Shallow bedrock	Fully submerged screen	No test	5.9E-7	0.0017
OW-618	18.5–28.5	Saprolite/Shallow bedrock	Fully submerged screen	2.2E-4	4.3E-4	1.2
OW-620	76.6–86.5	Saprolite/Shallow bedrock	Fully submerged screen	1.1E-3	1.3E-3	3.6
OW-621B	60–70	Saprolite/Shallow bedrock	Fully submerged screen	2.2E-4	2.2E-4	0.61
OW-622	48.5–58.5	Saprolite/Shallow bedrock	Fully submerged screen	4.8E-4	4.8E-4	1.4
OW-623	76.5–86.5	Saprolite/Shallow bedrock	Fully submerged screen	1.8E-4	1.1E-4	0.52
OW-625	84.5–104.5	Saprolite/Shallow bedrock	Partially submerged screen	No test	4.2E-4	1.2
OW-626	71–81	Saprolite/Shallow bedrock	Fully submerged screen	3.1E-5	1.3E-5	0.087
OW-627B	43–53	Saprolite/Shallow bedrock	Fully submerged screen	5.6E-5	1.6E-5	0.16

Hydrostratigraphic Zone	Maximum Test Result Range: Low (feet/day)	Maximum Test Result Range: High (feet/day)	Maximum Test Result Range: Geometric Mean (feet/day)
Saprolite/Shallow Bedrock Zone	0.0017	18	0.62
Deep Bedrock Zone	0.0088	0.38	0.07
All	0.0017	18	0.37

Slug test results for eight wells are not included because of invalid test conditions or questionable data.
Statistics are calculated using maximum result from either falling head test or rising head test (if both performed).

Table 2.4-219
Packer Test Results

Boring Number	Test Section Depth (feet bgs)	Material	Feet/Year	Feet/Day
	Test Interval		Hydraulic Conductivity	
B-201	65–75	Sound Rock	0	0.00
	86–96	Sound Rock	49	0.13
B-205	59–69	Rock/Sound Rock	417	1.14
	96–106	Sound Rock	0	0.00
B-305	62–72	Sound Rock	86	0.24
	72–82	Sound Rock	0	0.00
B-330	57–67	Sound Rock	5	0.014
	67–77	Sound Rock	92	0.25

Hydraulic Conductivity (feet/day)		
Minimum	Maximum	Geometric Mean
0	1.14	0.17

Table 2.4-220 (Sheet 1 of 2)
Summary of Laboratory Test Results for Grain Size, Moisture Content, and Specific Gravity and Derived Porosity Values

Source of Sample	Sample Number	Sample Depth	USCS	Unit[(a)]	Gs	Dry Density (pcf)	Void Ratio[(b)]	Porosity[(b)]	Wet Density	Water Content
B-204	UD-2	18.5	ML	Residual Soil	2.87	95.07	0.884	0.469	112	17.8%
B-204	UD-3	28.5	ML	Saprolite	2.95	87.44	1.105	0.525	109	24.1%
B-209	UD-1	8.5	MH	Residual Soil	2.81	70.59	1.484	0.597	101	42.9%
B-209[(c)]	UD-2	18.5	SM	Residual Soil	2.795	64.38	1.709	0.631	96	48.7%
B-209	UD-4	38.5	ML	Saprolite	2.86	87.32	1.044	0.511	114	30.2%
B-210	UD-1	8.5	ML	Residual Soil	2.75	88.56	0.938	0.484	108	22.3%
B-210	UD-3	28.5	ML	Saprolite	2.73	95.85	0.777	0.437	118	23.4%
B-210	UD-4	38.5	ML	Saprolite	2.78	84.91	1.043	0.511	108	27.1%
B-215	UD-1	8.5	SM	Saprolite	2.78	85.97	1.018	0.504	112	30.5%
B-215[(c)]	UD-2	18.5	SM	Saprolite	2.82	91.17	0.930	0.482	113	24.2%
B-215[(c)]	UD-3	28.5	SM	Saprolite	2.791	86.7	1.009	0.502	108	24.2%
B-216[(c)]	UD-1	6.5	ML	Saprolite	2.791	64.05	1.719	0.632	87	35.8%
B-216[(c)]	UD-2	13.5	ML	Saprolite	2.791	81.19	1.145	0.534	108	32.6%
B-216[(c)]	UD-3	23.8	ML	Saprolite	2.791	81.55	1.136	0.532	110	35.4%
B-217[(c)]	UD-1	8.5	SM	Saprolite	2.791	87.93	0.981	0.495	112	27.8%
B-222	UD-1	8.5	ML	Residual Soil	2.71	90.49	0.869	0.465	115	26.7%
B-222	UD-2	18.5	ML	Residual Soil	2.84	89.78	0.974	0.493	110	22.3%
B-222[(c)]	UD-3	28.5	SM	Saprolite	2.791	87.1	1.000	0.500	105	20.3%
B-309[(c)]	UD-1	8.5	SM	Saprolite	2.791	87.19	0.997	0.499	107	22.4%

Table 2.4-220 (Sheet 2 of 2)
Summary of Laboratory Test Results for Grain Size, Moisture Content, and Specific Gravity and Derived Porosity Values

Source of Sample	Sample Number	Sample Depth	USCS	Unit[a]	Gs	Dry Density (pcf)	Void Ratio[b]	Porosity[b]	Wet Density	Water Content
B-309[c]	UD-3	28.5	ML	Saprolite	2.791	81.45	1.138	0.532	104	27.7%
B-309[c]	UD-4	38.5	SM	Saprolite	2.791	88.6	0.966	0.491	108	21.7%
B-319[c]	UD-2	18.5	SM	Saprolite	2.791	91.6	0.901	0.474	109	19.5%
B-319	UD-3	28.5	ML	Saprolite	2.75	91.85	0.868	0.465	115	24.9%
B-319	UD-4	38.5	ML	Saprolite	2.75	102.8	0.669	0.401	123	19.6%
B-321[c]	UD-2	18.5	SM	Saprolite	2.791	90.79	0.918	0.479	109	19.7%
B-321	UD-3	28.5	SM	Saprolite	2.83	102.6	0.721	0.419	120	16.7%
B-322[c]	UD-2	18.5	SM	Saprolite	2.791	88.28	0.973	0.493	102	15.2%
B-325(c)	UD-1	3.5	ML	Residual Soil	2.795	78.2	1.230	0.552	108	38.0%
B-325	UD-3	13.5	SM	Saprolite	2.77	82.91	1.085	0.520	104	25.8%
B-325	UD-8	38.5	SM	Saprolite	2.69	97.39	0.724	0.420	118	21.0%
		MIN VALUES:		Residual Soil	2.71	64.38	0.869	0.465	96	17.8%
				Saprolite	2.69	64.05	0.669	0.401	87	15.2%
		MAX VALUES:		Residual Soil	2.87	95.07	1.709	0.631	115	48.7%
				Saprolite	2.95	102.80	1.719	0.632	123	35.8%
		GEOMEAN VALUES:		Residual Soil	2.80	81.71	1.118	0.524	107.0	29.4%
				Saprolite	2.79	87.75	0.976	0.492	109.5	24.2%

(a) Unit from Table 2A of Reference 218

(b) Calculated values using Equation 1.20 of Reference 221, Page 26

Equation 1.20

$\rho_d=((Gs)/(1+e))*\rho_w$

This can be rearranged to show:

$e=(Gs*\rho_w/\rho_d)-1$

Porosity can be derived from the void ratio by:

$n=e/(1+e)$

Where:

ρ_d = Dry Density
ρ_w = Density of Water
e = void ratio
n = porosity
Gs = Specific Gravity

(c) No Gs value was obtained for these samples. For these samples, the average value was used to calculate the void ratio and porosity values

Data summarized from Table F-1 - Summary of Soil Tests (Reference 218)

Table 2.4-221
Groundwater Levels at Unit 2 and Unit 3

Well ID	Formation	Hydro. Zone	June Water Level El. 6/23/06	July Water Level El. 7/25/06	Aug. Water Level El. 8/30/06	Sept. Water Level El. 9/19/06	Oct. Water Level El. 10/24/06	Nov. Water Level El. 11/29/06	Dec. Water Level El. 12/20/06	Jan. Water Level El. 01/26/07	Feb. Water Level El. 02/20/07	March Water Level El. 03/20/07	April Water Level El. 04/19/07	May Water Level El. 05/23/07	June Water Level El. 06/27/07	MIN	MAX	RANGE
UNIT 2																		
OW-205 a	Sound Rock	Deep Bedrock	357.3	357.3	357.1	357.2	357.1	357.4	357.5	358.4	358.6	358.9	359.1	359.0	359.0	**357.1**	**359.1**	**2.0**
OW-205 b	Partially Weathered Rock	Saprolite/Shallow Bedrock	364.9	365.0	365.2	366.1	366.1	365.3	365.4	365.5	365.7	365.9	366.3	366.9	367.2	**364.9**	**367.2**	**2.2**
OW-212	Saprolite/Partially Weathered Rock	Saprolite/Shallow Bedrock	351.4	351.0	351.2	351.1	350.8	351.6	351.3	352.5	352.8	353.1	352.9	352.8	352.6	**350.8**	**353.1**	**2.3**
OW-213	Saprolite	Saprolite/Shallow Bedrock	359.1	359.1	359.1	359.1	359.0	359.1	359.2	360.3	360.6	361.0	361.1	361.0	360.8	**359.0**	**361.1**	**2.1**
OW-227	Bedrock	Deep Bedrock	361.5	361.3	361.3	361.3	361.3	361.3	361.3	361.4	361.7	362.0	362.3	362.6	362.8	**361.3**	**362.8**	**1.6**
OW-233	Bedrock	Deep Bedrock	322.5	339.9	358.6	362.4	365.2	366.2	366.4	366.9	367.1	367.1	367.3	367.2	367.4	**322.5**	**367.4**	**45.0**
UNIT 3																		
OW-305 a	Sound Rock	Deep Bedrock	368.2	368.3	368.1	368.2	368.2	368.3	368.3	368.4	368.5	368.6	368.8	369.0	369.2	**368.1**	**369.2**	**1.1**
OW-305 b	Partially Weathered Rock/Sound Rock	Saprolite/Shallow Bedrock	367.4	367.5	367.4	367.4	367.5	367.6	367.5	367.6	367.7	367.8	368.0	368.2	368.4	**367.4**	**368.4**	**0.9**
OW-312	Saprolite/Partially Weathered Rock	Saprolite/Shallow Bedrock	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry	**0.0**	**0.0**	**0.0**
OW-313	Saprolite/Partially Weathered Rock	Saprolite/Shallow Bedrock	372.8	372.7	372.9	373.0	373.2	373.3	373.3	373.1	373.8	374.1	374.6	374.9	375.1	**372.7**	**375.1**	**2.3**
OW-327	Partially Weathered Rock	Saprolite/Shallow Bedrock	359.2	359.1	359.2	359.3	359.4	359.6	359.7	360.0	360.2	360.4	360.7	361.1	361.4	**359.1**	**361.4**	**2.3**
OW-333	Sound Rock	Deep Bedrock	333.8	334.7	335.1	335.1	335.1	334.6	335.0	336.2	337.6	338.5	339.5	339.5	339.1	**333.8**	**339.5**	**5.7**

Notes:
Hydro. = Hydrostratigraphic
El. = Elevation

Table 2.4-222
Groundwater Wells at Unit 1 Locations and Unit 2 & Unit 3 Site

Period	Saprolite/Shallow Bedrock	Assumed Saprolite/Shallow Bedrock	Deep Bedrock
July 1977 to October 1984		P-1, P-2, P-3, P-4, P-5, P-6, P-7	
May 1998 to May 2006		GW-1, GW-2, GW-3, GW-4, GW-5, GW-6, GW-7	
3rd Quarter 2000 to 3rd Quarter 2006		GW-8, GW-9, GW-12, GW-13A, GW-15	
June 2006 to June 2007	OW-205b, OW-212, OW-213, OW-305b, OW-312, OW-313, OW-327, OW-401b, OW-405, OW-501, OW-612, OW-614, OW-617, OW-618, OW-620, OW-621b, OW-622, OW-623, OW-624, OW-625, OW-626, OW-627b		OW-205a, OW-227, OW-233, OW-305a, OW-333, OW-401a, OW-619, OW-621a, OW-627a

Table 2.4-223 (Sheet 1 of 2)
Monthly Rainfall Data from Parr Climate Station by Water Year

Monthly Rainfall Statistics (inches)																
Water Year	Sum	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep			
Average	43.77	2.83	2.73	3.26	4.04	3.80	4.45	3.12	3.40	3.96	4.33	4.11	3.76			
Stdev	8.42	2.38	1.81	1.90	1.92	1.71	2.42	1.74	1.52	2.54	2.33	2.34	2.58			
Rec Max	59.91	10.25	8.19	7.78	8.33	7.77	10.61	8.07	8.47	12.20	11.47	11.44	12.17			
Rec Min	27.86	0.00	0.18	0.00	0.00	0.68	0.56	0.40	0.60	0.48	1.17	0.00	0.00			
Monthly Rainfall Amounts (inches)														Departure From Mean		
Water Year	Sum	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual (inches)	Cumulative (inches)	Cumulative (%)
1949	50.60	1.52	8.19	4.41	1.18	4.67	1.20	3.56	3.94	1.90	6.78	11.44	1.81	6.83	6.83	13.5%
1950	30.93	1.62	2.19	1.77	1.6	1.03	3.40	1.09	3.85	2.74	5.51	2.88	3.25	–12.84	–6.02	–11.9%
1951	35.26	2.60	0.75	3.51	1.73	1.12	4.54	3.59	0.60	4.53	3.22	3.66	5.41	–8.51	–14.53	–28.7%
1952	47.77	0.39	3.35	4.38	2.71	4.48	8.16	2.98	3.91	1.68	3.33	9.39	3.01	4.00	–10.54	–20.8%
1953	32.34	0.63	1.21	3.54	2.26	4.97	3.67	2.03	4.01	2.11	1.62	2.60	3.69	–11.43	–21.97	–43.4%
1954	27.86	0.21	0.72	7.39	4.32	2.95	2.02	2.90	2.10	0.49	3.00	1.41	0.35	–15.91	–37.88	–74.9%
1955	35.26	0.66	2.68	2.35	4.52	2.64	1.67	4.35	4.70	1.46	4.10	5.04	1.09	–8.51	–46.40	–91.7%
1956	39.39	2.74	2.90	0.24	1.69	6.11	4.00	5.06	3.47	1.38	3.54	3.44	4.82	–4.38	–50.78	–100.4%
1957	37.32	2.73	0.55	2.03	2.52	1.35	4.81	1.41	8.47	1.61	2.38	3.18	6.28	–6.45	–57.24	–113.1%
1958	46.65	2.56	6.63	2.3	4.48	3.65	4.11	5.45	3.52	5.15	5.69	2.38	0.73	2.88	–54.36	–107.4%
1959	53.71	2.71	0.18	3.61	2.58	4.12	5.78	3.55	2.96	2.78	9.04	4.23	12.17	9.94	–44.43	–87.8%
1960	48.33	6.71	0.53	1.41	8.29	7.19	5.18	3.01	2.76	3.05	4.27	2.12	3.81	4.56	–39.87	–78.8%
1961	40.45	2.58	1.27	1.97	3.06	6.37	4.23	5.93	3.05	4.48	4.11	2.94	0.46	–3.32	–43.19	–85.4%
1962	39.95	0.13	1.51	4.62	6.7	3.54	5.30	2.73	1.21	3.65	2.45	2.59	5.52	–3.82	–47.02	–92.9%
1963	43.69	0.23	3.11	2.33	4.84	4.75	4.74	3.80	3.89	4.26	4.74	2.24	4.76	–0.08	–47.10	–93.1%
1964	59.91	0.02	5.07	3.1	6.22	4.81	8.57	4.71	2.56	5.79	8.40	7.84	2.82	16.14	–30.97	–61.2%
1965	59.91	10.25	1.58	6.55	1.51	3.70	8.26	4.50	3.13	9.92	5.33	2.80	2.38	16.14	–14.83	–29.3%
1966	34.13	1.93	2.63	0.66	5.06	3.92	3.07	1.89	4.69	2.87	1.29	3.23	2.89	–9.64	–24.47	–48.4%
1967	35.72	3.06	0.95	2.61	2.74	3.47	1.12	2.55	4.16	3.86	3.37	5.83	2.00	–8.05	–32.53	–64.3%
1968	41.59	1.51	3.20	3.03	6.18	0.68	3.32	3.03	3.44	8.95	5.29	1.86	1.10	–2.18	–34.71	–68.6%
1969	48.28	1.45	5.40	2.57	3.46	3.70	4.55	8.07	2.60	3.25	1.82	4.36	7.05	4.51	–30.21	–59.7%
1970	32.83	0.45	1.08	4.8	3.27	2.20	6.49	1.32	3.23	0.66	3.51	3.94	1.88	–10.94	–41.15	–81.3%
1971	57.28	9.50	1.10	3.71	5.63	4.32	7.78	2.48	4.35	2.83	6.60	6.62	2.36	13.51	–27.65	–54.6%
1972	49.67	3.81	1.98	2.36	5.44	5.07	3.57	0.78	7.13	4.74	8.13	4.72	1.94	5.90	–21.75	–43.0%
1973	53.95	1.56	3.49	5.87	4.44	3.92	4.41	4.38	5.98	8.38	2.53	4.10	4.89	10.18	–11.57	–22.9%
1974	45.83	1.38	0.60	6.7	5.8	3.51	2.11	2.75	5.14	3.55	5.13	4.75	4.41	2.06	–9.52	–18.8%
1975	53.57	0	1.92	5.74	7.13	5.11	6.83	2.57	5.08	1.64	11.47	1.06	5.02	9.80	0.28	0.5%
1976	42.10	1.95	3.62	4.28	2.94	1.05	4.89	1.23	5.16	3.61	4.38	3.83	5.16	–1.67	–1.40	–2.8%

Table 2.4-223 (Sheet 2 of 2)
Monthly Rainfall Data from Parr Climate Station by Water Year

	Monthly Rainfall Amounts (inches)													Departure From Mean		
Water Year	Sum	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual (inches)	Cumulative (inches)	Cumulative (%)
1977	50.14	5.42	4.66	4.63	3.06	0.88	10.24	0.82	2.43	5.07	5.08	3.73	4.12	6.37	4.97	9.8%
1978	47.63	6.45	2.35	2.63	6.97	0.99	5.88	3.88	5.38	4.67	1.36	5.22	1.85	3.86	8.83	17.4%
1979	42.39	1.50	2.22	1.74	5.47	6.13	2.36	7.36	3.46	3.20	3.84	0.41	4.70	–1.38	7.44	14.7%
1980	44.86	1.84	3.54	1.49	6.01	2.05	10.61	1.51	1.19	5.81	1.17	1.67	7.97	1.09	8.53	16.9%
1981	32.46	3.46	2.11	1.27	0.80	3.83	2.10	1.95	2.08	3.47	2.83	4.70	3.86	–11.31	–2.79	–5.5%
1982	50.78	3.15	0.82	7.78	6.37	6.89	1.52	6.37	4.52	4.40	2.74	4.33	1.89	7.01	4.22	8.3%
1983	42.77	2.38	2.69	5.12	3.96	4.81	8.47	4.00	1.31	0.48	2.78	2.86	3.91	–1.00	3.22	6.4%
1984	54.76	2.99	3.93	6.69	4.40	5.41	3.91	4.62	6.13	2.41	8.43	5.43	0.41	10.99	14.20	28.1%
1985	34.75	1.58	0.88	1.67	4.74	6.12	0.65	1.18	3.56	1.62	6.55	5.58	0.62	–9.02	5.18	10.2%
1986	37.97	3.52	6.74	0.66	1.64	1.59	2.84	0.40	4.08	2.03	4.28	9.05	1.14	–5.80	–0.63	–1.2%
1987	49.58	4.18	3.50	4.08	7.28	5.70	5.11	0.48	1.29	5.92	1.74	2.72	7.58	5.81	5.18	10.2%
1988	34.82	0.79	3.23	1.50	2.98	1.99	3.45	2.52	2.01	1.91	1.89	5.09	7.46	–8.95	–3.78	–7.5%
1989	53.28	2.93	3.62	0.30	2.37	3.66	6.74	4.26	3.91	12.2	4.28	3.43	5.58	9.51	5.73	11.3%
1990	35.79	3.55	3.15	4.02	3.62	4.36	2.43	1.71	2.40	1.39	2.54	5.02	1.60	–7.98	–2.25	–4.5%
1991	54.55	10.01	3.01	2.60	5.70	2.65	5.13	4.93	1.54	5.45	8.20	4.25	1.08	10.78	8.52	16.8%
1992	39.76	0.14	0.46	3.68	2.48	4.04	4.68	2.89	2.25	5.45	3.98	6.73	2.98	–4.01	4.51	8.9%
1993	49.28	5.91	7.75	2.97	5.74	4.41	6.38	2.25	1.48	2.26	3.73	1.56	4.84	5.51	10.01	19.8%
1994	47.93	4.63	2.44	2.65	4.15	3.87	5.99	1.34	3.07	7.06	2.44	7.00	3.29	4.16	14.17	28.0%
1995	48.92	4.23	3.73	4.94	5.17	5.69	2.07	0.99	4.35	5.01	4.09	7.11	1.54	5.15	19.32	38.2%
1996	47.22	5.21	3.62	1.75	4.04	2.09	4.66	3.82	3.00	3.76	2.09	7.62	5.56	3.45	22.76	45.0%
1997	47.93	5.30	2.04	2.67	4.60	4.88	5.18	3.41	2.54	4.24	7.64	1.77	3.66	4.16	26.92	53.2%
1998	54.25	4.22	4.97	4.34	8.33	7.77	0.56	5.81	2.17	1.91	1.35	4.33	8.49	10.48	37.39	73.9%
1999	30.66	1.69	1.78	2.92	4.32	2.80	2.35	2.71	2.01	3.55	1.20	0.74	4.59	–13.11	24.28	48.0%
2000	41.11	2.18	1.58	1.46	4.86	2.43	3.40	2.53	2.55	3.00	7.51	1.84	7.77	-2.66	21.61	42.7%
2001	31.84	0	3.30	1.27	0	2.02	6.05	1.47	3.37	3.90	7.83	0.97	1.66	–11.93	9.68	19.1%
2002	29.30	1.18	0.97	2.33	3.71	2.41	3.25	3.71	4.01	1.05	3.33	0	3.35	–14.47	–4.79	–9.5%
2003	52.10	3.71	3.54	4.64	2.18	4.35	8.75	5.85	4.17	3.85	5.77	3.11	2.18	8.33	3.53	7.0%
2004	40.42	2.87	1.01	2.15	1.57	4.72	1.09	1.05	1.25	7.10	2.18	5.08	10.35	–3.35	0.18	0.4%
2005	35.60	0.82	4.22	0	2.37	4.67	3.28	3.46	3.20	4.32	4.11	5.15	0	–8.17	–8.00	–15.8%
2006	51.77	3.44	1.92	7.23	2.86	2.69	1.06	2.02	3.19	11.74	5.08	7.46	3.08	8.00	0.00	0.0%

Table 2.4-224
Summary of Depth to Groundwater Correlation with Precipitation Data

	Well ID	Installed Date	Elevation	Total Depth	Depth to Groundwater Correlation with Precipitation Data			
					Annual Deviation from the Mean		Cumulative Annual Deviation from the Mean	
					R^2 Value	Linear Regression Formula	R^2 Value	Linear Regression Formula
Piezometry Program	P1	1977	436.8	Unknown	Not calculated. Data was not measured after 1984 and these wells were installed to show the impact on groundwater levels of the impoundment of the Monticello Reservoir.			
	P2	1977	438.22	Unknown				
	P3	1977	Unknown	Unknown				
	P4	1977	436.35	Unknown				
	P5	1977	436.7	Unknown				
	P6	1977	Unknown	Unknown				
	P7	1977	437.1	Unknown				
Auxiliary Building Fuel Oil Storage Tank Wells	GW-1	1998	Unknown	25.6	0.1379	y = 0.4687x + 1.4019	0.0493	y = -0.1802x + 1.6799
	GW-2	1998	Unknown	25.4	0.2194	y = -0.2071x - 0.6195	0.2257	y = -0.1351x + 1.2593
	GW-3	1998	Unknown	24.2	0.0497	y = 0.3632x + 1.0861	0.3756	y = -0.6422x + 5.9858
	GW-4	1998	Unknown	22.9	0.0494	y = -0.1692x - 0.5061	0.6596	y = -0.3974x + 3.7041
	GW-5	1998	Unknown	23.4	8E-05	y = 0.0085x + 0.0253	0.556	y = -0.4522x + 4.2151
	GW-6	1998	Unknown	23.92	0.1798	y = -0.5224x - 1.5625	0.1614	y = -0.3182x + 2.9663
	GW-7	1998	Unknown	25.36	0.0708	y = -0.1114x - 0.3331	0.5842	y = -0.2058x + 1.9179
NPDES Wells	GW-8	2000	Unknown	26.66	0.2384	y = -0.4849x - 1.6819	0.1302	y = -0.3254x + 1.0326
	GW-9	2000	Unknown	37.84	0.0152	y = -0.1386x - 0.4806	0.3804	y = -0.6287x + 1.9953
	GW-12	2000	Unknown	37.56	0.4402	y = 4.134x + 14.339	0.0027	y = 0.2937x - 0.9321
	GW-13a	2000	Unknown	47.78	0.0142	y = -0.1259x - 0.4368	0.2273	y = -0.4569x + 1.4502
	GW-15	2000	Unknown	35.34	0.0029	y = 0.0593x + 0.2058	0.583	y = -0.7619x + 2.418

Table 2.4-225 (Sheet 1 of 2)
Radionuclide Inventory for Tank Rupture

Radionuclide	Design Basis Reactor Coolant Activity[(a)] (μCi/g)	Reactor Coolant Concentration[(b)] ($\mu Ci/cm^3$)	Effluent Holdup Tank Concentration[(c)] ($\mu Ci/cm^3$)
H-3	1.00E+00	1.00E+00	1.01E+00
Cr-51	1.30E-03	1.30E-03	1.31E-03
Mn-54	6.70E-04	6.70E-04	6.77E-04
Mn-56	1.70E-01	1.70E-01	1.72E-01
Fe-55	5.00E-04	5.00E-04	5.05E-04
Fe-59	1.30E-04	1.30E-04	1.31E-04
Co-58	1.90E-03	1.90E-03	1.92E-03
Co-60	2.20E-04	2.20E-04	2.22E-04
Br-83	3.20E-02	1.54E-02	1.55E-02
Br-84	1.70E-02	8.16E-03	8.24E-03
Br-85	2.00E-03	9.60E-04	9.70E-04
Rb-88	1.50E+00	7.20E-01	7.27E-01
Rb-89	6.90E-02	3.31E-02	3.35E-02
Sr-89	1.10E-03	5.28E-04	5.33E-04
Sr-90	4.90E-05	2.35E-05	2.38E-05
Sr-91	1.70E-03	8.16E-04	8.24E-04
Sr-92	4.10E-04	1.97E-04	1.99E-04
Y-90	1.30E-05	6.24E-06	6.30E-06
Y-91m	9.20E-04	4.42E-04	4.46E-04
Y-91	1.40E-04	6.72E-05	6.79E-05
Y-92	3.40E-04	1.63E-04	1.65E-04
Y-93	1.10E-04	5.28E-05	5.33E-05
Nb-95	1.60E-04	7.68E-05	7.76E-05
Zr-95	1.60E-04	7.68E-05	7.76E-05
Mo-99	2.10E-01	1.01E-01	1.02E-01
Tc-99m	2.00E-01	9.60E-02	9.70E-02
Ru-103	1.40E-04	6.72E-05	6.79E-05
Rh-103m	1.40E-04	6.72E-05	6.79E-05
Rh-106	4.50E-05	2.16E-05	2.18E-05
Ag-110m	4.00E-04	1.92E-04	1.94E-04
Te-127m	7.60E-04	3.65E-04	3.68E-04
Te-129m	2.60E-03	1.25E-03	1.26E-03
Te-129	3.80E-03	1.82E-03	1.84E-03
Te-131m	6.70E-03	3.22E-03	3.25E-03
Te-131	4.30E-03	2.06E-03	2.08E-03
Te-132	7.90E-02	3.79E-02	3.83E-02
Te-134	1.10E-02	5.28E-03	5.33E-03
I-129	1.50E-08	7.20E-09	7.27E-09
I-130	1.10E-02	5.28E-03	5.33E-03
I-131	7.10E-01	3.41E-01	3.44E-01
I-132	9.40E-01	4.51E-01	4.56E-01
I-133	1.30E+00	6.24E-01	6.30E-01

Table 2.4-225 (Sheet 2 of 2)
Radionuclide Inventory for Tank Rupture

Radionuclide	Design Basis Reactor Coolant Activity[(a)] (μCi/g)	Reactor Coolant Concentration[(b)] ($\mu Ci/cm^3$)	Effluent Holdup Tank Concentration[(c)] ($\mu Ci/cm^3$)
I-134	2.20E-01	1.06E-01	1.07E-01
I-135	7.80E-01	3.74E-01	3.78E-01
Cs-134	6.90E-01	3.31E-01	3.35E-01
Cs-136	1.00E+00	4.80E-01	4.85E-01
Cs-137	5.00E-01	2.40E-01	2.42E-01
Cs-138	3.70E-01	1.78E-01	1.79E-01
Ba-137m	4.70E-01	2.26E-01	2.28E-01
Ba-140	1.00E-03	4.80E-04	4.85E-04
La-140	3.10E-04	1.49E-04	1.50E-04
Ce-141	1.60E-04	7.68E-05	7.76E-05
Ce-143	1.40E-04	6.72E-05	6.79E-05
Pr-143	1.50E-04	7.20E-05	7.27E-05
Ce-144	1.20E-04	5.76E-05	5.82E-05
Pr-144	1.20E-04	5.76E-05	5.82E-05

(a) Values from AP1000 DCD, Table 11.1-2.
(b) For tritium (H-3) a coolant concentration of 1.0 µCi/g is used; corrosion products (Cr-51, Mn-54, Mn-56, Fe-55, Fe-59, Co-58 and Co-60) are taken directly from the AP1000 DCD, Table 11.1-2; and other radionuclides are based on the AP1000 DCD, Table 11.1-2 multiplied by 0.12/0.25. The density of all liquids is assumed to be 1 g/cm3.
(c) Values are 101% of the reactor coolant concentrations.

Table 2.4-226
Hydraulic Gradient Calculation for Unit 2 and Unit 3

Unit 2 Hydraulic Gradient

*using September 2006 Groundwater Levels

Boring	**Northing**	**Easting**	**Distance**		**WL Elevation**
			(feet)		(feet)
OW-205b	892842	1903193			366.1
Groundwater discharge point at unnamed creek to the north-northwest of Unit 2 (Point A)			850		340
				Change in Head =	26.1
				Hydraulic Gradient =	**0.0307**

Unit 3 Hydraulic Gradient

*using September 2006 Groundwater Levels

Boring	**Northing**		**Distance**		**WL Elevation**
			(feet)		(feet)
OW-305	891997	1902858			367.4
OW-618 (Point B)	890956	1901480	1,727		303.6
				Change in Head =	63.8
				Hydraulic Gradient =	**0.0369**

Table 2.4-227 (Sheet 1 of 3)
Results of Transport Analysis with Radioactive Decay Only - Discharge to Unnamed Creek to North-Northwest from Unit 2 (Travel Time of 17.39 Years)

Parent Nuclide	Progeny in Chain	Radionuclide Characteristics						C_0[d] (μCi/cm^3)	Radioactive Decay				
		$t_{1/2}$[a] (days)	d_{12}	d_{13}	d_{23}	λ[b] (days^{-1})	MPC[c] (μCi/cm^3)		K_1	K_2	K_3	C[e] (μCi/cm^3)	C/MPC
H-3		4.51E+03				1.54E-04	1.00E-03	1.01E+00				3.81E-01	3.81E+02
Cr-51		2.77E+01				2.50E-02	5.00E-04	1.31E-03				1.27E-72	2.53E-69
Mn-54		3.13E+02				2.21E-03	3.00E-05	6.77E-04				5.28E-10	1.76E-05
Mn-56		1.07E-01				6.48E+00	7.00E-05	1.72E-01				0.00E+00	0.00E+00
Fe-55		9.86E+02				7.03E-04	1.00E-04	5.05E-04				5.81E-06	5.81E-02
Fe-59		4.45E+01				1.56E-02	1.00E-05	1.31E-04				1.44E-47	1.44E-42
Co-58		7.08E+01				9.79E-03	2.00E-05	1.92E-03				1.91E-30	9.56E-26
Co-60		1.93E+03				3.59E-04	3.00E-06	2.22E-04				2.27E-05	7.56E+00
Br-83		9.96E-02				6.96E+00	9.00E-04	1.55E-02				0.00E+00	0.00E+00
	Kr-83m	7.63E-02	0.9998			9.09E+00	NA[f]					0.00E+00	
Br-84		2.21E-02				3.14E+01	4.00E-04	8.24E-03				0.00E+00	0.00E+00
Br-85		2.01E-03				3.44E+02	NA[f]	9.70E-04				0.00E+00	
Rb-88		1.24E-02				5.61E+01	4.00E-04	7.27E-01				0.00E+00	0.00E+00
Rb-89		1.06E-02				6.54E+01	9.00E-04	3.35E-02				0.00E+00	0.00E+00
	Sr-89	5.05E+01	1.0000			1.37E-02	8.00E-06	5.33E-04	–7.03E-06	5.40E-04		7.53E-42	9.41E-37
Sr-90		1.06E+04				6.54E-05	5.00E-07	2.38E-05				1.57E-05	3.14E+01
	Y-90	2.67E+00	1.0000			2.60E-01	7.00E-06	6.30E-06	2.38E-05	–1.75E-05		1.57E-05	2.25E+00
Sr-91		3.96E-01				1.75E+00	2.00E-05	8.24E-04				0.00E+00	0.00E+00
	Y-91m	3.45E-02	0.5780			2.01E+01	2.00E-03	4.46E-04	5.22E-04	–7.57E-05		0.00E+00	0.00E+00
	Y-91	5.85E+01		0.4220	1.0000	1.18E-02	8.00E-06	6.79E-05	–5.93E-06	4.47E-08	7.38E-05	1.55E-37	1.93E-32
Sr-92		1.13E-01				6.14E+00	4.00E-05	1.99E-04				0.00E+00	0.00E+00
	Y-92	1.48E-01	1.0000			4.68E+00	4.00E-05	1.65E-04	–6.40E-04	8.05E-04		0.00E+00	0.00E+00
Y-93		4.21E-01				1.65E+00	2.00E-05	5.33E-05				0.00E+00	0.00E+00
Zr-95		6.40E+01				1.08E-02	2.00E-05	7.76E-05				1.05E-34	5.23E-30
	Nb-95m	3.61E+00	0.0070			1.92E-01	3.00E-05		5.76E-07	-5.76E-07		7.76E-37	2.59E-32
	Nb-95	3.52E+01		0.9930	1.0000	1.97E-02	3.00E-05	7.76E-05	1.73E-04	6.58E-08	-9.50E-05	2.32E-34	7.75E-30
Mo-99		2.75E+00				2.52E-01	2.00E-05	1.02E-01				0.00E+00	0.00E+00
	Tc-99m	2.51E-01	0.8760			2.76E+00	1.00E-03	9.70E-02	9.83E-02	–1.33E-03		0.00E+00	0.00E+00
Ru-103		3.93E+01				1.76E-02	3.00E-05	6.79E-05				1.54E-53	5.14E-49
	Rh-103m	3.90E-02	0.9970			1.78E+01	6.00E-03	6.79E-05	6.78E-05	1.36E-07		1.54E-53	2.56E-51
Rh-106		3.45E-04				2.01E+03	NA[f]	2.18E-05				0.00E+00	

Table 2.4-227 (Sheet 2 of 3)
Results of Transport Analysis with Radioactive Decay Only - Discharge to Unnamed Creek to North-Northwest from Unit 2 (Travel Time of 17.39 Years)

Parent Nuclide	Progeny in Chain	Radionuclide Characteristics						C_0(d) (µCi/cm^3)	Radioactive Decay				
		$t_{1/2}$(a) (days)	d_{12}	d_{13}	d_{23}	λ(b) (days^{-1})	MPC(c) (µCi/cm^3)		K_1	K_2	K_3	C(e) (µCi/cm^3)	C/MPC
Ag-110m		2.50E+02				2.77E-03	6.00E-06	1.94E-04				4.37E-12	7.29E-07
	Ag-110	2.85E-04	0.0133			2.43E+03	NA(f)		2.58E-06	-2.58E-06		5.82E-14	
Te-127m		1.09E+02				6.36E-03	9.00E-06	3.68E-04				1.06E-21	1.18E-16
	Te-127	3.90E-01	0.9760			1.78E+00	1.00E-04		3.60E-04	-3.60E-04		1.04E-21	1.04E-17
Te-129m		3.36E+01				2.06E-02	7.00E-06	1.26E-03				1.60E-60	2.29E-55
	Te-129	4.83E-02	0.6500			1.44E+01	4.00E-04	1.84E-03	8.20E-04	1.02E-03		1.04E-60	2.60E-57
	I-129	5.73E+09		0.3500	1.0000	1.21E-10	2.00E-07	7.27E-09	–7.39E-12	–8.59E-15	7.28E-09	7.28E-09	3.64E-02
Te-131m		1.25E+00				5.55E-01	8.00E-06	3.25E-03				0.00E+00	0.00E+00
	Te-131	1.74E-02	0.2220			3.98E+01	8.00E-05	2.08E-03	7.32E-04	1.35E-03		0.00E+00	0.00E+00
	I-131	8.04E+00		0.7780	1.0000	8.62E-02	1.00E-06	3.44E-01	–6.00E-04	–2.92E-06	3.45E-01	5.78E-239	5.78E-233
Te-132		3.26E+00				2.13E-01	9.00E-06	3.83E-02				0.00E+00	0.00E+00
	I-132	9.58E-02	1.0000			7.24E+00	1.00E-04	4.56E-01	3.95E-02	4.17E-01		0.00E+00	0.00E+00
Te-134		2.90E-02				2.39E+01	3.00E-04	5.33E-03				0.00E+00	0.00E+00
	I-134	3.65E-02	1.0000			1.90E+01	4.00E-04	1.07E-01	–2.07E-02	1.28E-01		0.00E+00	0.00E+00
I-130		5.15E-01				1.35E+00	2.00E-05	5.33E-03				0.00E+00	0.00E+00
I-133		8.67E-01				7.99E-01	7.00E-06	6.30E-01				0.00E+00	0.00E+00
	Xe-133m	2.19E+00	0.0290			3.17E-01	NA(f)		–1.20E-02	1.20E-02		0.00E+00	
	Xe-133	5.25E+00		0.9710	1.0000	1.32E-01	NA(f)		–1.19E-01	–8.58E-03	1.27E-01	0.00E+00	
I-135		2.75E-01				2.52E+00	3.00E-05	3.78E-01				0.00E+00	0.00E+00
	Xe-135m	1.06E-02	0.1540			6.53E+01	NA(f)		6.05E-02	-6.05E-02		0.00E+00	
	Xe-135	3.79E-01		0.8460	1.0000	1.83E+00	NA(f)		–1.01E+00	1.75E-03	1.00E+00	0.00E+00	
Cs-134		7.53E+02				9.21E-04	9.00E-07	3.35E-01				9.69E-04	1.08E+03
Cs-136		1.31E+01				5.29E-02	6.00E-06	4.85E-01				5.67E-147	9.45E-142
Cs-137		1.10E+04				6.30E-05	1.00E-06	2.42E-01				1.62E-01	1.62E+05
	Ba-137m	1.77E-03	0.9460			3.91E+02	NA(f)	2.28E-01	2.29E-01	–9.32E-04		1.53E-01	
Cs-138		2.24E-02				3.09E+01	4.00E-04	1.79E-01				0.00E+00	0.00E+00
Ba-140		1.27E+01				5.46E-02	8.00E-06	4.85E-04				1.44E-154	1.79E-149
	La-140	1.68E+00	1.0000			4.13E-01	9.00E-06	1.50E-04	5.59E-04	–4.09E-04		1.65E-154	1.84E-149
Ce-141		3.25E+01				2.13E-02	3.00E-05	7.76E-05				1.17E-63	3.90E-59
Ce-143		1.38E+00				5.04E-01	2.00E-05	6.79E-05				0.00E+00	0.00E+00
	Pr-143	1.36E+01	1.0000			5.11E-02	2.00E-05	7.27E-05	–7.66E-06	8.04E-05		8.38E-146	4.19E-141
Ce-144		2.84E+02				2.44E-03	3.00E-06	5.82E-05				1.08E-11	3.60E-06

Table 2.4-227 (Sheet 3 of 3)
Results of Transport Analysis with Radioactive Decay Only - Discharge to Unnamed Creek to North-Northwest from Unit 2 (Travel Time of 17.39 Years)

Parent Nuclide	Progeny in Chain	Radionuclide Characteristics						C_0[d] (μCi/cm^3)	Radioactive Decay				
		$t_{1/2}$[a] (days)	d_{12}	d_{13}	d_{23}	λ[b] (days^{-1})	MPC[c] (μCi/cm^3)		K_1	K_2	K_3	C[e] (μCi/cm^3)	C/MPC
	Pr-144m	5.07E-03	0.0178			1.37E+02	NA[f]		1.04E-06	–1.04E-06		1.92E-13	
	Pr-144	1.20E-02		0.9822	0.9990	5.78E+01	6.00E-04	5.82E-05	5.82E-05	7.57E-07	-7.58E-07	1.08E-11	1.80E-08

(a) Values are taken from Table E.1 of NUREG/CR-5512 (Reference 216), with the exception of Sr-92, Rh-106, Ba-137m, Ag-110, Xe-133, Xe-133m, Xe-135m, and PR-144m, which are taken from ICRP Publication 38 (Reference 210)
(b) Values calculated from Equation 2.4-4.
(c) Values from 10 CFR Part 20, Appendix B, Table 2, Column 2.
(d) Values from Table 2.4-225.
(e) Values calculated from Equation 2.4-8, 2.4-13, or 2.4-18 depending on position in decay chain.
(f) Maximum Permissible Concentration (MPC) is not available.

Table 2.4-228 (Sheet 1 of 3)
Results of Transport Analysis with Radioactive Decay Only - Discharge to Unnamed Creek to South-Southwest from Unit 3 (Travel Time of 29.35 Years)

Parent Nuclide	Progeny in Chain	Radionuclide Characteristics						C_0[d] ($\mu Ci/cm^3$)	Radioactive Decay				
		$t_{1/2}$[a] (days)	d_{12}	d_{13}	d_{23}	λ[b] ($days^{-1}$)	MPC[c] ($\mu Ci/cm^3$)		K_1	K_2	K_3	C[e] ($\mu Ci/cm^3$)	C/MPC
H-3		4.51E+03				1.54E-04	1.00E-03	1.01E+00				1.94E-01	1.94E+02
Cr-51		2.77E+01				2.50E-02	5.00E-04	1.31E-03				4.20E-120	8.40E-117
Mn-54		3.13E+02				2.21E-03	3.00E-05	6.77E-04				3.32E-14	1.11E-09
Mn-56		1.07E-01				6.48E+00	7.00E-05	1.72E-01				0.00E+00	0.00E+00
Fe-55		9.86E+02				7.03E-04	1.00E-04	5.05E-04				2.70E-07	2.70E-03
Fe-59		4.45E+01				1.56E-02	1.00E-05	1.31E-04				4.01E-77	4.01E-72
Co-58		7.08E+01				9.79E-03	2.00E-05	1.92E-03				5.08E-49	2.54E-44
Co-60		1.93E+03				3.59E-04	3.00E-06	2.22E-04				4.72E-06	1.57E+00
Br-83		9.96E-02				6.96E+00	9.00E-04	1.55E-02				0.00E+00	0.00E+00
	Kr-83m	7.63E-02	0.9998			9.09E+00	NA[f]					0.00E+00	
Br-84		2.21E-02				3.14E+01	4.00E-04	8.24E-03				0.00E+00	0.00E+00
Br-85		2.01E-03				3.44E+02	NA[f]	9.70E-04				0.00E+00	
Rb-88		1.24E-02				5.61E+01	4.00E-04	7.27E-01				0.00E+00	0.00E+00
Rb-89		1.06E-02				6.54E+01	9.00E-04	3.35E-02				0.00E+00	0.00E+00
	Sr-89	5.05E+01	1.0000			1.37E-02	8.00E-06	5.33E-04	–7.03E-06	5.40E-04		6.82E-68	8.52E-63
Sr-90		1.06E+04				6.54E-05	5.00E-07	2.38E-05				1.18E-05	2.36E+01
	Y-90	2.67E+00	1.0000			2.60E-01	7.00E-06	6.30E-06	2.38E-05	–1.75E-05		1.18E-05	1.69E+00
Sr-91		3.96E-01				1.75E+00	2.00E-05	8.24E-04				0.00E+00	0.00E+00
	Y-91m	3.45E-02	0.5780			2.01E+01	2.00E-03	4.46E-04	5.22E-04	–7.57E-05		0.00E+00	0.00E+00
	Y-91	5.85E+01		0.4220	1.0000	1.18E-02	8.00E-06	6.79E-05	–5.93E-06	4.47E-08	7.38E-05	5.10E-60	6.37E-55
Sr-92		1.13E-01				6.14E+00	4.00E-05	1.99E-04				0.00E+00	0.00E+00
	Y-92	1.48E-01	1.0000			4.68E+00	4.00E-05	1.65E-04	–6.40E-04	8.05E-04		0.00E+00	0.00E+00
Y-93		4.21E-01				1.65E+00	2.00E-05	5.33E-05				0.00E+00	0.00E+00
Zr-95		6.40E+01				1.08E-02	2.00E-05	7.76E-05				2.95E-55	1.47E-50
	Nb-95m	3.61E+00	0.0070			1.92E-01	3.00E-05		5.76E-07	–5.76E-07		2.19E-57	7.29E-53
	Nb-95	3.52E+01		0.9930	1.0000	1.97E-02	3.00E-05	7.76E-05	1.73E-04	6.58E-08	-9.50E-05	6.56E-55	2.19E-50
Mo-99		2.75E+00				2.52E-01	2.00E-05	1.02E-01				0.00E+00	0.00E+00
	Tc-99m	2.51E-01	0.8760			2.76E+00	1.00E-03	9.70E-02	9.83E-02	–1.33E-03		0.00E+00	0.00E+00
Ru-103		3.93E+01				1.76E-02	3.00E-05	6.79E-05				5.28E-87	1.76E-82
	Rh-103m	3.90E-02	0.9970			1.78E+01	6.00E-03	6.79E-05	6.78E-05	1.36E-07		5.27E-87	8.78E-85

Table 2.4-228 (Sheet 2 of 3)
Results of Transport Analysis with Radioactive Decay Only - Discharge to Unnamed Creek to South-Southwest from Unit 3 (Travel Time of 29.35 Years)

Parent Nuclide	Progeny in Chain	Radionuclide Characteristics						C_0[d] (μCi/cm^3)	Radioactive Decay				
		$t_{1/2}$[a] (days)	d_{12}	d_{13}	d_{23}	λ[b] (days^{-1})	MPC[c] (μCi/cm^3)		K_1	K_2	K_3	C[e] (μCi/cm^3)	C/MPC
Rh-106		3.45E-04				2.01E+03	NA[f]	2.18E-05				0.00E+00	
Ag-110m		2.50E+02				2.77E-03	6.00E-06	1.94E-04				2.40E-17	4.00E-12
	Ag-110	2.85E-04	0.0133			2.43E+03	NA[f]		2.58E-06	–2.58E-06		3.19E-19	
Te-127m		1.09E+02				6.36E-03	9.00E-06	3.68E-04				9.15E-34	1.02E-28
	Te-127	3.90E-01	0.9760			1.78E+00	1.00E-04		3.60E-04	–3.60E-04		8.96E-34	8.96E-30
Te-129m		3.36E+01				2.06E-02	7.00E-06	1.26E-03				1.15E-99	1.65E-94
	Te-129	4.83E-02	0.6500			1.44E+01	4.00E-04	1.84E-03	8.20E-04	1.02E-03		7.51E-100	1.88E-96
	I-129	5.73E+09		0.3500	1.0000	1.21E-10	2.00E-07	7.27E-09	–7.39E-12	–8.59E-15	7.28E-09	7.28E-09	3.64E-02
Te-131m		1.25E+00				5.55E-01	8.00E-06	3.25E-03				0.00E+00	0.00E+00
	Te-131	1.74E-02	0.2220			3.98E+01	8.00E-05	2.08E-03	7.32E-04	1.35E-03		0.00E+00	0.00E+00
	I-131	8.04E+00		0.7780	1.0000	8.62E-02	1.00E-06	3.44E-01	–6.00E-04	–2.92E-06	3.45E-01	0.00E+00	0.00E+00
Te-132		3.26E+00				2.13E-01	9.00E-06	3.83E-02				0.00E+00	0.00E+00
	I-132	9.58E-02	1.0000			7.24E+00	1.00E-04	4.56E-01	3.95E-02	4.17E-01		0.00E+00	0.00E+00
Te-134		2.90E-02				2.39E+01	3.00E-04	5.33E-03				0.00E+00	0.00E+00
	I-134	3.65E-02	1.0000			1.90E+01	4.00E-04	1.07E-01	–2.07E-02	1.28E-01		0.00E+00	0.00E+00
I-130		5.15E-01				1.35E+00	2.00E-05	5.33E-03				0.00E+00	0.00E+00
I-133		8.67E-01				7.99E-01	7.00E-06	6.30E-01				0.00E+00	0.00E+00
	Xe-133m	2.19E+00	0.0290			3.17E-01	NA[f]		–1.20E-02	1.20E-02		0.00E+00	
	Xe-133	5.25E+00		0.9710	1.0000	1.32E-01	NA[f]		–1.19E-01	–8.58E-03	1.27E-01	0.00E+00	
I-135		2.75E-01				2.52E+00	3.00E-05	3.78E-01				0.00E+00	0.00E+00
	Xe-135m	1.06E-02	0.1540			6.53E+01	NA[f]		6.05E-02	–6.05E-02		0.00E+00	
	Xe-135	3.79E-01		0.8460	1.0000	1.83E+00	NA[f]		–1.01E+00	1.75E-03	1.00E+00	0.00E+00	
Cs-134		7.53E+02				9.21E-04	9.00E-07	3.35E-01				1.74E-05	1.93E+01
Cs-136		1.31E+01				5.29E-02	6.00E-06	4.85E-01				2.28E-247	3.80E-242
Cs-137		1.10E+04				6.30E-05	1.00E-06	2.42E-01				1.23E-01	1.23E+05
	Ba-137m	1.77E-03	0.9460			3.91E+02	NA[f]	2.28E-01	2.29E-01	-9.32E-04		1.17E-01	
Cs-138		2.24E-02				3.09E+01	4.00E-04	1.79E-01				0.00E+00	0.00E+00
Ba-140		1.27E+01				5.46E-02	8.00E-06	4.85E-04				3.98E-258	4.97E-253
	La-140	1.68E+00	1.0000			4.13E-01	9.00E-06	1.50E-04	5.59E-04	-4.09E-04		4.59E-258	5.10E-253
Ce-141		3.25E+01				2.13E-02	3.00E-05	7.76E-05				3.99E-104	1.33E-99
Ce-143		1.38E+00				5.04E-01	2.00E-05	6.79E-05				0.00E+00	0.00E+00
	Pr-143	1.36E+01	1.0000			5.11E-02	2.00E-05	7.27E-05	–7.66E-06	8.04E-05		8.59E-243	4.29E-238

Table 2.4-228 (Sheet 3 of 3)
Results of Transport Analysis with Radioactive Decay Only - Discharge to Unnamed Creek to South-Southwest from Unit 3 (Travel Time of 29.35 Years)

Parent Nuclide	Progeny in Chain	Radionuclide Characteristics						C_0[d] (μCi/cm^3)	Radioactive Decay				
		$t_{1/2}$[a] (days)	d_{12}	d_{13}	d_{23}	λ[b] (days^{-1})	MPC[c] (μCi/cm^3)		K_1	K_2	K_3	C[e] (μCi/cm^3)	C/MPC
Ce-144		2.84E+02				2.44E-03	3.00E-06	5.82E-05				2.53E-16	8.42E-11
	Pr-144m	5.07E-03	0.0178			1.37E+02	NA[f]		1.04E-06	–1.04E-06		4.50E-18	
	Pr-144	1.20E-02		0.9822	0.9990	5.78E+01	6.00E-04	5.82E-05	5.82E-05	7.57E-07	–7.58E-07	2.53E-16	4.21E-13

(a) Values are taken from Table E.1 of NUREG/CR-5512 (Reference 216), with the exception of Sr-92, Rh-106, Ba-137m, Ag-110, Xe-133, Xe-133m, Xe-135m, and PR-144m, which are taken from ICRP Publication 38 (Reference 210)
(b) Values calculated from Equation 2.4-4.
(c) Values from 10 CFR Part 20, Appendix B, Table 2, Column 2.
(d) Values from Table 2.4-225
(e) Values calculated from Equation 2.4-8, 2.4-13, or 2.4-18 depending on position in decay chain.
(f) Maximum Permissible Concentration (MPC) is not available.

Table 2.4-229
Co, Sr, and Cs K_d values from Laboratory Testing

SRNL ID#	Boring	Lithology	Co k_d (mL/g)	Sr k_d (mL/g)	Cs k_d (mL/g)
416	B-212/212a #14 (48.5-50 feet bgs)	Saprolite	676.1	62.0	89.3
418	B-620 #12 (38.5–40 feet bgs)	Saprolite	415.9	38.5	71.0
419	B-627 #9 (23.5-25 feet bgs)	Saprolite	1576.4	67.9	512.6
	Lowest Reported Value:		415.9	38.5	71.0

Values from Mactec Data Report Attachment H – K_d Distribution Coefficient Test Results Table 5 (Reference 217).
bgs = below ground surface

Table 2.4-230
Transport Analysis Considering Radioactive Decay and Adsorption for Unit 2

Parent Radionuclide	Progeny in Chain	Radionuclide Characteristics					C_0(c)	Radioactive Decay				
		$t_{1/2}$(a) (days)	d_{12}	d_{13}	d_{23}	λ(b) ($days^{-1}$)	($\mu Ci/cm^3$)	K_d (cm^3/g)	R(d)	C(e) ($\mu Ci/cm^3$)	MPC(f) ($\mu Ci/cm^3$)	C/MPC
H-3		4.51E+03				1.54E-04	1.01E+00	0.00	1.00	3.81E-01	1.00E-03	3.81E+02
Fe-55		9.86E+02				7.03E-04	5.05E-04	0.00	1.00	5.81E-06	1.00E-04	5.81E-02
Co-60		1.93E+03				3.59E-04	2.22E-04	415.90	1504.64	0.00E+00	3.00E-06	0.00E+00
Sr-90		1.06E+04				6.54E-05	2.38E-05	38.50	140.19	1.24E-30	5.00E-07	2.48E-24
	Y-90	2.67E+00	1.0000			2.60E-01	6.30E-06	0.00	1.00	1.28E-30	7.00E-06	1.83E-25
Te-129m		3.36E+01				2.06E-02	1.26E-03	0.00	1.00	—	7.00E-06	—
	Te-129	4.83E-02	0.6500			1.44E+01	1.84E-03	0.00	1.00	—	4.00E-04	—
	I-129	5.73E+09		0.3500	1.0000	1.21E-10	7.27E-09	0.00	1.00	7.28E-09	2.00E-07	3.64E-02
Cs-134		7.53E+02				9.21E-04	3.35E-01	71.00	257.69	0.00E+00	9.00E-07	0.00E+00
Cs-137		1.10E+04				6.30E-05	2.42E-01	71.00	257.69	3.97E-46	1.00E-06	3.97E-40

Travel time	=	17.39	years
Porosity	=	0.49	
Effective porosity	=	0.39	
Bulk dry density	=	1.41	g/cm^3

Notes:
a) Values from Table E.1, NUREG/CR-5512 (Reference 216).
b) Values calculated from Equation 2.4-4.
c) Values from Table 2.4-225.
d) Values calculated from Equation 2.4-2.
e) Values calculated from Equation 2.4-8, 2.4-13, or 2.4-18 depending on position in decay chain.
f) Values from 10 CFR Part 20, Appendix B, Table 2, Column 2.

Table 2.4-231
Transport Analysis Considering Radioactive Decay and Adsorption for Unit 3

Parent Radionuclide	Progeny in Chain	Radionuclide Characteristics					C_0[c] (μCi/cm^3)	Radioactive Decay				
		$t_{1/2}$[a] (days)	d_{12}	d_{13}	d_{23}	λ[b] (days^{-1})		K_d (cm^3/g)	R[d]	C[e] (μCi/cm^3)	MPC[f] (μCi/cm^3)	C/MPC
H-3		4.51E+03				1.54E-04	1.01E+00	0.00	1.00	1.94E-01	1.00E-03	1.94E+02
Co-60		1.93E+03				3.59E-04	2.22E-04	415.90	1504.64	0.00E+00	3.00E-06	0.00E+00
Sr-90		1.06E+04				6.54E-05	2.38E-05	38.50	140.19	5.00E-48	5.00E-07	9.99E-42
	Y-90	2.67E+00	1.0000			2.60E-01	6.30E-06	0.00	1.00	5.18E-48	7.00E-06	7.40E-43
Te-129m		3.36E+01				2.06E-02	1.26E-03	0.00	1.00	—	7.00E-06	—
	Te-129	4.83E-02	0.6500			1.44E+01	1.84E-03	0.00	1.00	—	4.00E-04	—
	I-129	5.73E+09		0.3500	1.0000	1.21E-10	7.27E-09	0.00	1.00	7.28E-09	2.00E-07	3.64E-02
Cs-134		7.53E+02				9.21E-04	3.35E-01	71.00	257.69	0.00E+00	9.00E-07	0.00E+00
Cs-137		1.10E+04				6.30E-05	2.42E-01	71.00	257.69	6.15E-77	1.00E-06	6.15E-71

Travel time	=	29.35	years
Porosity	=	0.49	
Effective porosity	=	0.39	
Bulk dry density	=	1.41	g/cm^3

Notes:
a) Values from Table E.1, NUREG/CR-5512 (Reference 216).
b) Values calculated from Equation 2.4-4.
c) Values from Table 2.4-225.
d) Values calculated from Equation 2.4-2.
e) Values calculated from Equation 2.4-8, 2.4-13, or 2.4-18 depending on position in decay chain.
f) Values from 10 CFR Part 20, Appendix B, Table 2, Column 2.

Table 2.4-232
Calculation of Dilution Factor for Unit 2

Tank volume	28,000 gallons
Spill volume (80% tank volume)	22,400 gallons
	2,995 cf
Contaminated Aquifer Volume	7,679 cf
Contaminated Aquifer Area	767.9 feet
Effective Porosity	0.39
Darcy velocity	0.0522 feet/day
Unit thickness	10 feet
Contaminated Aquifer Width	27.7 feet
Contaminated cross-sectional area	277 ft^2
Flow Rate	14.46 cf/day
	1.67E-04 cfs
	7.51E-02 gpm
Minimum River flow[(a)]	125 cfs
Dilution Factor	1.34E-06

(a) 100-year daily-mean low flow in the Broad River at Parr Shoals Dam is 125 cfs.

Table 2.4-233
Calculation of Dilution Factor for Unit 3

Tank volume	28,000 Gallons
Spill volume (80% tank volume)	22,400 Gallons 2,995 cf
Contaminated Aquifer Volume	7,679 cf
Contaminated Aquifer Area	767.9 feet
Effective Porosity	0.39
Darcy velocity	0.0630 feet/day
Unit thickness	10 feet
Contaminated Aquifer Width	27.7 feet
Contaminated cross-sectional area	277 ft^2
Flow Rate	17.41 cf/day 2.01E-04 cfs 9.04E-02 gpm
Minimum River flow[(a)]	125 cfs
Dilution Factor	1.61E-06

(a) 100-year daily-mean low flow in the Broad River at Parr Shoals Dam is 125 cfs.

Table 2.4-234
Transport Analysis Considering Radioactive Decay, Adsorption, and Dilution

For Unit 2

Radionuclide	MPC[(a)] (μCi/cm^3)	Groundwater Concentration[(b)] (μCi/cm^3)	Surface Water Concentration[(c)] (μCi/cm^3)	Surface Water Concentration/MPC
H-3	1.00E-03	3.81E-01	5.10E-07	5.10E-04
Fe-55	1.00E-04	5.81E-06	7.79E-12	7.79E-08
I-129	2.00E-07	7.28E-09	9.75E-15	4.87E-08

For Unit 3

Radionuclide	MPC[(a)] (μCi/cm^3)	Groundwater Concentration[(d)] (μCi/cm^3)	Surface Water Concentration[(e)] (μCi/cm^3)	Surface Water Concentration/MPC
H-3	1.00E-03	1.94E-01	3.13E-07	3.13E-04
I-129	2.00E-07	7.28E-09	1.17E-14	5.86E-08

a) 10 CFR Part 20, Appendix B, Table 2, Column 2
b) Table 2.4-230
c) Surface water concentration = groundwater concentration*dilution factor (1.34E-06)
d) Table 2.4-231
e) Surface water concentration = groundwater concentration*dilution factor (1.61E-06)

Table 2.4-235 (Sheet 1 of 2)
Compliance with 10 CFR Part 20 for Unit 2

Parent Radionuclide	Progeny in Chain	Radioactive Decay	Radioactive Decay + Retardation	Radioactive Decay + Retardation + Dilution	Minimum C/MPC
H-3		3.81E+02	3.81E+02	5.10E-04	5.10E-04
Cr-51		2.53E-69			2.53E-69
Mn-54		1.76E-05			1.76E-05
Mn-56		0.00E+00			0.00E+00
Fe-55		5.81E-02	5.81E-02	7.79E-08	7.79E-08
Fe-59		1.44E-42			1.44E-42
Co-58		9.56E-26			9.56E-26
Co-60		7.56E+00	0.00E+00		0.00E+00
Br-83		0.00E+00			0.00E+00
	Kr-83m				0.00E+00
Br-84		0.00E+00			0.00E+00
Br-85					0.00E+00
Rb-88		0.00E+00			0.00E+00
Rb-89		0.00E+00			0.00E+00
	Sr-89	9.41E-37			9.41E-37
Sr-90		3.14E+01	2.48E-24		2.48E-24
	Y-90	2.25E+00	1.83E-25		1.83E-25
Sr-91		0.00E+00			0.00E+00
	Y-91m	0.00E+00			0.00E+00
	Y-91	1.93E-32			1.93E-32
Sr-92		0.00E+00			0.00E+00
	Y-92	0.00E+00			0.00E+00
Y-93		0.00E+00			0.00E+00
Zr-95		5.23E-30			5.23E-30
	Nb-95m	2.59E-32			2.59E-32
	Nb-95	7.75E-30			7.75E-30
Mo-99		0.00E+00			0.00E+00
	Tc-99m	0.00E+00			0.00E+00
Ru-103		5.14E-49			5.14E-49
	Rh-103m	2.56E-51			2.56E-51
Rh-106					0.00E+00
Ag-110m		7.29E-07			7.29E-07
	Ag-110				0.00E+00
Te-127m		1.18E-16			1.18E-16
	Te-127	1.04E-17			1.04E-17
Te-129m		2.29E-55			2.29E-55
	Te-129	2.60E-57			2.60E-57
	I-129	3.64E-02	3.64E-02	4.87E-08	4.87E-08
Te-131m		0.00E+00			0.00E+00
	Te-131	0.00E+00			0.00E+00
	I-131	5.78E-233			5.78E-233

Table 2.4-235 (Sheet 2 of 2)
Compliance with 10 CFR Part 20 for Unit 2

Parent Radionuclide	Progeny in Chain	Radioactive Decay	Radioactive Decay + Retardation	Radioactive Decay + Retardation + Dilution	Minimum C/MPC
Te-132		0.00E+00			0.00E+00
	I-132	0.00E+00			0.00E+00
Te-134		0.00E+00			0.00E+00
	I-134	0.00E+00			0.00E+00
I-130		0.00E+00			0.00E+00
I-133		0.00E+00			0.00E+00
	Xe-133m				0.00E+00
	Xe-133				0.00E+00
I-135		0.00E+00			0.00E+00
	Xe-135m				0.00E+00
	Xe-135				0.00E+00
Cs-134		1.08E+03	0.00E+00		0.00E+00
Cs-136		9.45E-142			9.45E-142
Cs-137		1.62E+05	3.97E-40		3.97E-40
	Ba-137m				0.00E+00
Cs-138		0.00E+00			0.00E+00
Ba-140		1.79E-149			1.79E-149
	La-140	1.84E-149			1.84E-149
Ce-141		3.90E-59			3.90E-59
Ce-143		0.00E+00			0.00E+00
	Pr-143	4.19E-141			4.19E-141
Ce-144		3.60E-06			3.60E-06
	Pr-144m				0.00E+00
	Pr-144	1.80E-08			1.80E-08
				Sum =	5.32E-04

Table 2.4-236 (Sheet 1 of 2)
Compliance with 10 CFR Part 20 for Unit 3

Parent Radionuclide	Progeny in Chain	Radioactive Decay	Radioactive Decay + Retardation	Radioactive Decay + Retardation + Dilution	Minimum C/MPC
H-3		1.94E+02	1.94E+02	3.13E-04	3.13E-04
Cr-51		8.40E-117			8.40E-117
Mn-54		1.11E-09			1.11E-09
Mn-56		0.00E+00			0.00E+00
Fe-55		2.70E-03			2.70E-03
Fe-59		4.01E-72			4.01E-72
Co-58		2.54E-44			2.54E-44
Co-60		1.57E+00	0.00E+00		0.00E+00
Br-83		0.00E+00			0.00E+00
	Kr-83m				0.00E+00
Br-84		0.00E+00			0.00E+00
Br-85					0.00E+00
Rb-88		0.00E+00			0.00E+00
Rb-89		0.00E+00			0.00E+00
	Sr-89	8.52E-63			8.52E-63
Sr-90		2.36E+01	9.99E-42		9.99E-42
	Y-90	1.69E+00	7.40E-43		7.40E-43
Sr-91		0.00E+00			0.00E+00
	Y-91m	0.00E+00			0.00E+00
	Y-91	6.37E-55			6.37E-55
Sr-92		0.00E+00			0.00E+00
	Y-92	0.00E+00			0.00E+00
Y-93		0.00E+00			0.00E+00
Zr-95		1.47E-50			1.47E-50
	Nb-95m	7.29E-53			7.29E-53
	Nb-95	2.19E-50			2.19E-50
Mo-99		0.00E+00			0.00E+00
	Tc-99m	0.00E+00			0.00E+00
Ru-103		1.76E-82			1.76E-82
	Rh-103m	8.78E-85			8.78E-85
Rh-106					0.00E+00
Ag-110m		4.00E-12			4.00E-12
	Ag-110				0.00E+00
Te-127m		1.02E-28			1.02E-28
	Te-127	8.96E-30			8.96E-30
Te-129m		1.65E-94			1.65E-94
	Te-129	1.88E-96			1.88E-96
	I-129	3.64E-02	3.64E-02	5.86E-08	5.86E-08
Te-131m		0.00E+00			0.00E+00
	Te-131	0.00E+00			0.00E+00
	I-131	0.00E+00			0.00E+00

Table 2.4-236 (Sheet 2 of 2)
Compliance with 10 CFR Part 20 for Unit 3

Parent Radionuclide	Progeny in Chain	Radioactive Decay	Radioactive Decay + Retardation	Radioactive Decay + Retardation + Dilution	Minimum C/MPC
Te-132		0.00E+00			0.00E+00
	I-132	0.00E+00			0.00E+00
Te-134		0.00E+00			0.00E+00
	I-134	0.00E+00			0.00E+00
I-130		0.00E+00			0.00E+00
I-133		0.00E+00			0.00E+00
	Xe-133m				0.00E+00
	Xe-133				0.00E+00
I-135		0.00E+00			0.00E+00
	Xe-135m				0.00E+00
	Xe-135				0.00E+00
Cs-134		1.93E+01	0.00E+00		0.00E+00
Cs-136		3.80E-242			3.80E-242
Cs-137		1.23E+05	6.15E-71		6.15E-71
	Ba-137m				0.00E+00
Cs-138		0.00E+00			0.00E+00
Ba-140		4.97E-253			4.97E-253
	La-140	5.10E-253			5.10E-253
Ce-141		1.33E-99			1.33E-99
Ce-143		0.00E+00			0.00E+00
	Pr-143	4.29E-238			4.29E-238
Ce-144		8.42E-11			8.42E-11
	Pr-144m				0.00E+00
	Pr-144	4.21E-13			4.21E-13
				Sum =	3.01E-03