**INTEGRAL
MONITORED RETRIEVABLE STORAGE (MRS)
FACILITY**

CONCEPTUAL DESIGN REPORT

**VOLUME VII — GEOTECHNICAL DESCRIPTION
CLINCH RIVER SITE
BASIS FOR DESIGN**

Prepared for

**UNITED STATES DEPARTMENT OF ENERGY
Richland Operations Office**

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**THE RALPH M. PARSONS COMPANY OF DELAWARE
Westinghouse Electric Corporation
Golder Associates**

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Report No. MRS 11

Contract No. DE-AC06-84RL 10436

September 1985

**THE RALPH M. PARSONS COMPANY OF DELAWARE
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ABSTRACT

The Nuclear Waste Policy Act of 1982 directed the Department of Energy (DOE) to complete a study of the need for and feasibility of, and to submit to Congress a proposal for, the construction of one or more monitored retrievable storage (MRS) facilities for high-level radioactive waste and spent nuclear fuel. The Act further directed that the proposal include at least three alternative sites and a designation of the site and storage technology concept deemed preferable by DOE. In 1985, DOE selected the Clinch River site as its preferred site for the new MRS Facility. This report, prepared in support of the MRS conceptual design effort, presents a geotechnical description of the Clinch River site, as based upon existing data.

This report has been prepared for the U.S. Department of Energy, Richland Operations, by The Ralph M. Parsons Company of Delaware and its subcontractor, Golder Associates.

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SECTION 1

INTRODUCTION

In April 1985, the Department of Energy (DOE) selected the Clinch River site as its preferred site for the construction and operation of the monitored retrievable storage (MRS) facility (USDOE, 1985). In support of the DOE MRS conceptual design activity, available data describing the site have been gathered and analyzed. A composite geotechnical description of the Clinch River site has been developed and is presented herein. No new data have been acquired for this site for the MRS Project. The data that exist for this site are substantially contained in the licensing and environmental documentation that was developed in the Clinch River Breeder Reactor Project (CRBRP). Additional data are contained in the open literature. Since the development of the licensing and environmental documentation for the CRBRP, some additional data have been collected by others. To the extent that these data have been acquired and reviewed, they are represented herein.

At some time before the licensing and construction of an MRS Facility, a site confirmation program will be undertaken by the Department of Energy. That effort has not now been fully planned. This report does not attempt to fully represent the need for data acquisition or confirmation, except when, in the professional judgments of the authors, such representations are warranted. Also, in some cases, the adequacy of the existing data base is commented upon.

This report presents Clinch River site description data in the following sections:

2. General Site Description
3. Surface Hydrologic Characteristics
4. Groundwater Characteristics
5. Geologic Characteristics
6. Vibratory Ground Motion
7. Surface Faulting
8. Stability of Subsurface Materials
9. Slope Stability
10. References

SECTION 2

GENERAL SITE DESCRIPTION

2.1 SITE DESCRIPTION

The Clinch River site is located in east-central Tennessee, in the eastern part of Roane County (latitude 35 degrees 53 minutes 24 seconds north/longitude 84 degrees 22 minutes 57 seconds west) (see Figure 2-1). The MRS Facility at the Clinch River site includes the entire site area for the cancelled CRBRP. The site is located on the southeast flank of Chestnut Ridge, on a peninsula formed by a meander of the Clinch River. It is 25 miles west of Knoxville, Tennessee, and 9 miles southwest of Oak Ridge, Tennessee, although the site lies within the Oak Ridge city limits. The site is adjacent to the U.S. Department of Energy Oak Ridge reservation.

The Clinch River site has a total area of 1,364 acres. As a part of the CRBRP, major site modifications were made, including the development of major excavations for the eventual construction of portions of the plant. Upon termination of that project, DOE has proceeded with the redress and reclamation of the site. The site is currently owned and under the control of DOE.

Figures 2-2, 2-3, and 2-4 show the Clinch River site. Figure 2-2 represents the topography of the site following the excavations for the CRBRP. Figure 2-3 shows the site topography planned upon the completion of the site redress. Figure 2-4 shows the site as anticipated following the completion of the MRS Facility.

The MRS Facility, as shown (Figures 2-3 and 2-4) for the primary storage concept, includes the Storage Area, the Protected Area, and the Limited Access Area. All of the Category I structures will be located in the Storage and Protected Areas. Only Non-Category I structures will be located in the Limited Access Area. The Storage Area will include the storage casks, supported on large concrete pads, and associated gravel roadways for the cask transporter and onsite vehicles. The majority of the Storage Area will be in cuts (up to 70 feet) with areas of fill (up to 50 feet deep in the southern portion of the Storage Area). The Receiving and Handling Building, located in the Protected Area, will be placed in an area of cut (up to 20 feet), which is predominantly on rock, and is located adjacent to the excavation developed for the CRBRP. The Receiving and Handling Building is a large structure, approximately 500 feet by 550 feet in size. The remainder of the Protected Area includes both cuts (up to 20 feet) and fills (up to 40 feet). The Limited Access Area will include various support structures. The majority of the Limited Access Area will be in fills ranging from less than 5 feet to about 40 feet thick.

Figure 2-1 - Clinch River Site Location Map

Figure 2-2 - Site Preredress Topography and CRBRP Excavations

Figure 2-3 - Site Postredress Topography

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Figure 2-4 - MRS Facility Site Plan

At present, the known activities at the site are (1) the redress and reclamation activities associated with the termination of the CRBRP and (2) various ecological and environmental studies that are a part of ongoing projects at the Oak Ridge National Laboratory. The redress activities will not be ongoing at the time of the construction and operation of an MRS Facility at the Clinch River site. The ongoing studies may continue on the Clinch River site, but will not be conducted within the controlled area of the MRS Facility.

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

Information on industrial, transportation, and military facilities was derived primarily from the CRBRP Preliminary Safety Analysis Report (PSAR) (Project Management Corporation, 1975), supplemented by more recent data gathered for the Environmental Assessment for the proposed MRS Facility (USDOE, 1986) and by a brief site visit in July 1985. The types and functions of the various facilities discussed below are representative of the site vicinity, although details, such as numbers of personnel employed, are dated.

In 1975, the land adjoining the Clinch River site was zoned as suitable for forestry, agriculture, industry, and residential use (F.A.I.R.).

Six nuclear facilities were located within 50 miles of the site in 1975: the Oak Ridge Gaseous Diffusion Plant (ORGDP), Oak Ridge National Laboratory (ORNL), and the Y-12 Facility locations are shown in Figure 2-5; and facilities owned or operated by U.S. Nuclear, Inc.; Nuclear Environmental Engineering, Inc.; and Nuclear Assurance Corporation., located in the Clinch River Consolidated Industrial Park (CRCIP) (Figure 2-6). During the brief site visit in July 1985, there was no evidence of activity in the CRCIP by any of the aforementioned firms; however, facilities owned by IT Corporation and Applied Science Laboratory were observed. Their locations are also shown in Figure 2-6.

The Oak Ridge Gaseous Diffusion Plant is located about 3 miles north-northwest of the site. The plant was built to produce enriched uranium, and employed 2,800 people in 1975. It is operated for the DOE by Martin-Marietta. Current DOE plans are to mothball the plant in the near future.

Oak Ridge National Laboratory is located about 4 miles east-northeast of the site. It is a large research and development facility, performing nuclear and other types of research for the Federal Government. In 1975, ORNL employed 4,500 people. It is reported that a facility for injecting radioactive waste into subsurface strata is located at ORNL. Injection wells have been used periodically since February 1954 to inject waste mixed with a cement grout slurry into the Conasauga shale along planes generated by hydrofracturing. In 1978, injection activities were forecast to continue into units stratigraphically above the subsurface projection of the Copper Creek thrust fault.

ORGDP Oak Ridge Gaseous Diffusion Plant
 CRCIP Clinch River Consolidated Industrial Park
 ORNL Oak Ridge National Laboratory

SOURCE: Project Management Corporation (1982).

Figure 2-5 - Nuclear and Industrial Plants Within 10 Miles

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The Y-12 Facility is located 9 miles northeast of the site, and performs nuclear-weapons production and research and development activities for the DOE. In 1975, the Y-12 Facility employed 6,500 people.

Clinch River Consolidated Industrial Park is located about 1-1/2 miles north of the site. In 1975, three small industrial operations had just begun operations. U.S. Nuclear, Inc., had 25 employees engaged in fabrication of neutron absorbers and fuel elements for power reactors. Nuclear Environmental Engineering, Inc., had about 20 employees engaged in calibration and resale of radioisotopes for use in education, research, and industry, and in the manufacture of radioisotope generators and radioactive tracers for oil field and other uses. Nuclear Assurance Corporation had six employees engaged in cleaning uranium hexafluoride containers. During the site visit in July 1985, no evidence of any of these operations was observed.

Two facilities not reported in the 1975 PSAR were observed in the CRCIP during the July 1985 site visit. Applied Science Laboratory has a facility located at 1550 Bear Creek Road, approximately 1.2 miles from the site. IT Corporation also has a facility on Bear Creek Road, approximately 1.8 miles from the site. The nature of these facilities has not yet been ascertained (although the IT Corporation facility appeared to be a warehouse operation). Both are small operations, on the scale of the facilities located in CRCIP discussed above. They are included in this section because it is assumed, based on their proximity to the Oak Ridge Reservation, that they handle or store nuclear materials. Their locations are indicated in Figure 2-6.

The only non-nuclear industrial facility within 5 miles of the site reported in the 1975 PSAR was the Tennessee Valley Authority (TVA) Melton Hill Dam, located about 4.5 miles east of the site. No other facilities were observed during the July 1985 site visit. The Melton Hill Dam provides hydroelectric power to the TVA system, and extends navigation up the Clinch River. A small crew attends the locks, but the power unit operates unattended.

Although not an industrial facility, Watts Bar Lake extends up the Clinch River past the site. This lake is formed by Watts Bar Dam, approximately 30 miles from the site.

The Clinch River site is served by one major highway, Interstate 40 (I-40), which passes approximately 1.25 miles south of the site. The closest interchanges are State Route (SR) 58 and SR 95, which are about 4 miles and 3 miles, respectively, from the site. The average traffic count on I-40 between the exits for SR 58 and 95 for a 24-hour period was 16,500 in 1975. Major highways are shown in Figure 2-7, as well as Bear Creek Road, which connects the two highways and provides access to the site entrance. From I-40, the site may be reached either by exiting at SR 58 and driving northeast via Bear Creek Road to the site or by exiting at SR 95 and driving northwest via Bear Creek Road to the site. State Route 58, SR 95, and Bear Creek Road are two-lane, undivided highways.

SOURCE: Project Management Corporation (1982).

Figure 2-7 - Site Major Highways and Railroads

The distance from I-40 to the site via SR 58 is approximately 4 miles; the first 3 miles are not within the Oak Ridge Reservation, and the road passes through a low-density residential area.

The distance from I-40 to the site on SR 95 is approximately 8.4 miles. The first 0.8 mile is through a low-density residential area. Once across the Clinch River, SR 95 is on the Oak Ridge Reservation; except for Applied Science Laboratory and IT Corporation in the CRCIP, there are no residences or commercial activities until the site entrance.

In 1975, the closest main rail line to the site was Harriman Junction, approximately 10 miles northwest. This line was served by both the Cincinnati, New Orleans, and Texas Pacific (CNO&TP) and the Southern Railway. A spur serves the ORGDP. Rail lines are shown in Figure 2-7.

Barge traffic on the Clinch River past the site totaled 10,000 tons in 1971, and was increasing slightly in volume.

Airports located near the site are McGhee-Tyson Airport, a commercial airport located in Knoxville, approximately 28 miles east-southeast of the site; Rockwood Municipal Airport, a business and recreational airport, located approximately 18 miles west of the site; Oak Ridge Air Park, a recreational airport, located approximately 11 miles north-northeast of the site; and Meadowlake Airport, a recreational airport, located approximately 10 miles southwest of the site (Project Management Corporation, 1975).

Only McGhee-Tyson Airport had scheduled commercial flights, and the nearest flight path was about 10 miles south of the site. U.S. Flight information indicated that aircraft approaching McGhee-Tyson Airport were at a minimum altitude of 5,000 feet as they passed 10 miles south of the site. The nearest holding pattern for McGhee-Tyson Airport was approximately 30 miles northeast of the site.

No military facilities are within a 5-mile radius of the site (Project Management Corporation, 1975).

In 1975, a 6-in.-dia natural gas pipeline extended north-south along the east boundary of the site. It supplied gas to Lenoir City, Tennessee (Project Management Corporation, 1975).

SECTION 3
SURFACE HYDROLOGIC
CHARACTERISTICS

SECTION 3

SURFACE WATER HYDROLOGIC CHARACTERISTICS

The primary source of information concerning the surface water hydrology of the MRS Facility on the Clinch River site is the PSAR for the CRBRP (Project Management Corporation, 1982). Most of the work done in preparation of the PSAR applies directly to the establishment of an MRS Facility. Where appropriate, data included in the PSAR are summarized and referenced.

3.1 HYDROLOGIC DESCRIPTION

3.1.1 SITE AND STRUCTURE

The location and topography of the site, after completion of the proposed MRS Facility at Clinch River, are shown in Figures 2-3 and 2-4. The site is located on the Clinch River. There are no perennial streams on the site. However, heavy rains generate surface runoff, which flows in small valleys and gullies that extend from the ridge above the site to the river.

Preliminary design specifications for the MRS Facility show placement of support structures nearest the river at an elevation of 815 feet mean sea level (MSL). This elevation was considered to be above the maximum water level predicted for probable maximum events (see Sections 3.3.1.5 and 3.3.2.4). The Receiving and Handling Building will be at elevation 820 feet and the storage silos will be between elevations 871 and 884 feet.

3.1.2 HYDROSPHERE

The Clinch River basin lies between the Cumberland Mountains to the northwest (elevations up to 4,200 feet) and Clinch Mountain and Black Oak Ridge to the southeast (elevations to 4,700 feet). The drainage area of the river above its confluence with the Tennessee River is approximately 4,500 square miles. The drainage area above the site (from Clinch River Mile 15.5) is 3,380 square miles. The basin is in the Valley and Ridge physiographic province. The province is characterized by relatively narrow parallel ridges (elevations to 1,200 feet), separated by somewhat wider valleys, generally trending northeast-southwest. Approximately one-half of the land in the basin is forested.

The Clinch River has its headwaters in Tazewell County, Virginia, about 175 miles northeast of the site. It flows approximately 350 miles in a south-westerly direction, and joins the Tennessee River near Kingston, Tennessee, at Tennessee River Mile (TRM) 567.8. Watts Bar Dam is on the Tennessee River at TRM 529.9, below the confluence with the Clinch River. Watts Bar Reservoir extends upstream from Watts Bar Dam on both rivers, and reaches Clinch River Mile (CRM) 18.8. The site is adjacent to the upper portion of the reservoir. Melton Hill and Norris Dams are upstream from the site at CRM

23.1 and 79.8, respectively. A map showing the Tennessee River Basin upstream of Watts Bar Dam and highlighting major surface water system components is shown in Figure 3-1.

The site is bounded on the west, south, and east by a meander bend of the Clinch River between CRM 14.5 and 18.6. The average annual discharge of the Clinch River at Kingston is 5,380 cfs. The elevation of the river at the site is approximately 741 feet MSL during normal operation of Watts Bar Reservoir. The elevation of the bottom of the channel ranges between 719 and 720 feet. The average slope of the river from the base of Norris Dam at CRM 79.8 to CRM 7.0 is approximately 1-1/2 feet per mile, including the lift at Melton Hill Dam. A profile of the channel bottom from the base of Melton Hill Dam to the confluence with the Tennessee River is shown in Figure 3-2.

The major tributaries of the Clinch River are the Powell and Emory Rivers. The Powell River, with a drainage area of 938 square miles, enters the Clinch River at CRM 88.8. The confluence is upstream of Norris Dam; therefore, annual and peak flows are not relevant to the site. The Emory River enters the Clinch River at CRM 4.4, and has a drainage area of 865 square miles. Its annual average discharge is 1,590 cfs, and the peak discharge of March 23, 1929, has been estimated at 195,000 cfs at the confluence with the Clinch River.

Beaver Creek, Bull Run Creek, and Hinds Creek enter the Clinch River between Norris and Melton Hill Dams. Hence, their flows are not relevant to the site. Poplar Creek is a minor tributary that enters the Clinch River below Melton Hill Dam at CRM 12.0. It has a drainage area of 136 square miles, and an average discharge of 260 cfs. Because of its close proximity to the site, flood levels on Poplar Creek and the resulting backwater effects at the site are of interest. Sparse data on Poplar Creek are currently available. However, the results of the runoff model supplied in the CRBRP PSAR show no detrimental effects at the site during the probable maximum flood caused by Poplar Creek flows (Project Management Corporation, 1982). Topographic maps show the site to be protected from any potential overland flow resulting from predicted flood levels in Poplar Creek.

Several other small streams enter the Clinch River below Melton Hill Dam, but their flow contributions are not considered significant. The combined average annual discharges of Craney Creek, Poplar Springs Creek, and Grassy Creek total 22 cfs, making 0.5 percent of the average Clinch River flow. Topographic maps show that flooding of these streams will not affect the site.

Water levels at the site are controlled and influenced directly by five dams operated by the TVA. The site is located on an arm of the Watts Bar Reservoir, which extends up the Clinch River. Water surface elevations at the site are influenced by the operation of Watts Bar Dam, which is located on the Tennessee River at TRM 529.9. Watts Bar Dam is a multiple-purpose structure, designed for power generation, flood control, and navigational aid. Pool elevation is maintained between 740 and 741 feet during summer months, and between 735 and 737 feet during the remainder of the year. Water

Figure 3-1 - Major Surface Water System Components
Tennessee River Basin Above Watts Bar Dam

SOURCE: Project Management Corporation (1982)

Figure 3-2 - Clinch River Bottom Profile From CRM 0.0 to CRM 23.1

surface elevation at the site is normally 1 to 2 feet higher than the elevation at the dam because of backwater effects. The minimum elevation of the reservoir was recorded on March 20, 1945, at 733.4 feet, and the maximum was recorded on March 17, 1973, at 745.4 feet.

Norris Dam is located upstream from the site on the Clinch River at CRM 79.8. It is a multiple-purpose structure, providing power generation, flood control, and (rarely) low flow augmentation. Normal minimum pool elevation is 960 feet MSL, and the maximum recorded elevation is 993.8 feet, which occurred in June 1954. Norris Dam is the primary regulator of flow on the Clinch River affecting the site.

Melton Hill Dam is also upstream from the site at CRM 23.1. The dam can be used to regulate flows at the site, but releases are normally used only for low flow augmentation. The normal minimum pool stage at Melton Hill Reservoir is 790 feet MSL.

There is also a submerged dam on the Clinch River at CRM 3.9. This dam is associated with the Kingston Steam Plant, and was formed by dumping quarry stone on the river bed and leveling the top surface to elevation 722 feet. Fort Loudoun Dam on the Tennessee River and Tellico Dam on the Little Tennessee River can influence pool levels in Watts Bar Reservoir and water surface elevation at the site. Fort Loudoun Dam is located 72.4 miles upstream from Watts Bar Dam at TRM 602.3. Normal minimum pool elevation is 807 feet MSL. Tellico Dam was completed in 1979, although the powerhouse was not finished at this time. The dam is located at Little Tennessee River Mile 0.3. Tellico Reservoir is connected to Fort Loudoun Reservoir by an uncontrolled canal. Inflows to Tellico Reservoir are normally discharged through Fort Loudoun Reservoir. Tellico spillway is used only during large flood events. Inflows into Watts Bar Reservoir from Fort Loudoun-Tellico Reservoirs are sufficient to maintain the minimum pool elevation of Watts Bar Reservoir under extreme conditions.

The U.S. Geological Survey has maintained stream gauges on the Clinch River at the three locations listed in Table 3-1 and shown in Figure 3-1. Gauges were maintained for various periods from 1936 through 1968. Discharge records are also maintained at Melton Hill Dam. Based on the Melton Hill records from 1963 through 1979, the average flow of the Clinch River at the site is 5,380 cfs (U.S. Nuclear Regulatory Commission, 1982). The maximum hourly release was 54,260 cfs on April 5, 1977, and the maximum average daily release was 34,966 cfs on January 11, 1974. The maximum recorded discharge at the site occurred on February 9, 1937, before the closure (completion of the impoundment structure) of Melton Hill Dam; therefore, it is not relevant to the existing situation. Other stream gauge stations with records of varying lengths are included in Table 3-1 and in Figure 3-1. No stream gauges are currently maintained by the U.S. Geological Survey on the Clinch River.

Water temperature records have been maintained at CRM 21.6 since May 1963. The maximum temperature observed from then through 1973 was 78°F on June 21, 1968. The minimum observed temperature was 33°F on January 22, 1970. Water temperature at the site may be affected by unusual flow regulation at Melton

Table 3-1 - Clinch River Stream Gauge Locations and Records

<u>Station</u>	<u>River Mile</u>	<u>Period of Record</u>
Kingston (near the mouth of the Clinch River)	0.0	1874 - 1877 1882 - Present
Near Wheat (several miles downstream from the site)	14.4	1937 - 1963 1967 - 1968
Near Scarboro	39.0	1941 - 1962
Clinton (upstream from the site)	59.0	1883 - 1948 1950 - Present
Norris Dam (upstream from the site)	79.8	Varied

Hill Dam. Water temperatures will remain cool during normal and high releases, but may rise during periods of low or no flow (Project Management Corporation, 1982).

Water quality monitoring stations were operated at CRM 79.8 and CRM 23.1 in 1967-68. Based on the data collected at these stations, the water quality is not significantly affected by municipal or industrial waste discharges (Project Management Corporation, 1982). A 36-mile reach of the river below Norris Dam is subject to elevated temperatures caused by releases in the fall. The upper 17 miles of this reach are subject to low dissolved-oxygen levels during late summer and early fall. Both of these conditions adversely affect coldwater fish. Additional information on water quality of the Clinch River may be found in the Environmental Report for the CRBRP (Project Management Corporation, 1975).

3.2 FLOODS

3.2.1 FLOOD HISTORY

Gauged stage or discharge records have been maintained on the Clinch River for the periods and locations listed in Table 3-1 and shown in Figure 3-1 (USDOl, 1958, 1964, 1971; USDOl, 1968-9). The greatest flood of record occurred in March 1886, reaching an elevation of approximately 758 feet at CRM 16 and 764 feet at CRM 18. Backwater effects from the maximum flood on the Tennessee River approach this level at CRM 16, but are lower at CRM 18 (Project Management Corporation, 1982).

The largest flood since the completion of the present TVA system occurred in 1973. Intense rainfall and high local inflow resulted in record high levels in Watts Bar Reservoir and the Emory River. Flooding on the Clinch River reached an elevation of 748.6 feet at CRM 14.5 and 749.6 feet at CRM 17.8 (Project Management Corporation, 1982).

There are no historic records of flooding caused by dam failure or ice jams. Historical icing is discussed in Section 3.3.5.

3.2.2 FLOOD DESIGN CONSIDERATIONS

The 100-year flood elevation for the Clinch River at the MRS site is approximately 750 feet (Nuclear Regulatory Commission, 1982; Federal Emergency Management Agency, 1980). The MRS Facility, with elevations above 800 feet, will not occupy the 100-year flood plain. Based upon current conceptual designs, no significant portion of the 100-year flood plain will be modified by construction of the facility at the Clinch River site.

Maximum levels for flooding from precipitation and seismic events were computed for the CRBRP PSAR. The results of those computations are presented in detail in later section of this document. Table 3-2 provides a summary of all predicted water surface elevations, and will be referenced throughout this report. For comparison, Table 3-2 includes recorded and estimated historical water levels and the proposed CRBRP site elevation (see Sections 3.2.1 and 3.1.1, above).

Table 3-2 - Predicted Water Surface Elevations at Clinch River Site

Event	<u>Predicted Water Surface Elevation (feet above MSL)</u>							
	<u>Level Pool EL</u>		<u>With 40-mph Wind</u>		<u>Runup Caused by Wind</u>			
	<u>CRM 16</u>	<u>CRM 18</u>	<u>CRM 16</u>	<u>CRM 18</u>	<u>3:1 Slope</u>		<u>Vertical Wall</u>	
					<u>CRM 16</u>	<u>CRM 18</u>	<u>CRM 16</u>	<u>CRM 18</u>
Clinch River Flood, 1886	758	764						
Clinch River Flood, 1973	748.6	749.6						
Present level for repeat of Clinch River 1886 event	-	748						
Present level for repeat of Emory River 1929 event	-	751						
Site PMF	772.2	778.8	778.8	-	780.0	-	781.0	-
Norris OBE failure + 1/2 PMF event	798.2	804.3	800.3	806.3	801.8	807.9	803.1	809.2
Instantaneous disappearance of Norris Dam	811.0	818.0						
Recommended minimum site elevation	815							

The estimated probable maximum flood (PMF) at the site, computed from the most severe storm sequence as defined by the National Weather Service, would produce an elevation of 777.2 feet at CRM 16 and 778.8 feet at CRM 18 (Project Management Corporation, 1982). Results of the PMF are discussed in Section 3.3.1 and, because the PMF involves potential dam failures, results are included in Section 3.3.2.2. A 40-mile-per-hour wind coincident with the PMF is applicable for the site only at CRM 16. A discussion of coincident wind effects is included in Section 3.3.1.6.

An extensive earthquake analysis, considering flooding caused by dam failure alone and during storm events, was done for the CRBRP (Project Management Corporation, 1982). The scenarios are based on work done in the Final Safety Analysis Report (FSAR) for the Sequoyah Nuclear Plant (Tennessee Valley Authority, 1973), modified to apply to the CRBRP. The most severe water surface elevations at the site were found to be caused by a seismic failure of Norris Dam coincident with a 1/2 PMF storm and a full reservoir. Discussion of the results of this scenario is included in Section 3.3.2.4. The event includes the predicted failure of Melton Hill Dam.

Preliminary designs locate the MRS Facility support structures above an elevation of 820 feet. This elevation is considered to be above the level likely to be inundated under the most adverse conditions (Project Management Corporation, 1982).

3.2.3 EFFECTS OF LOCAL INTENSE PRECIPITATION

The 8-hour local probable maximum precipitation (PMP) storm depth developed for the CRBRP is 29.5 inches, and the maximum 1-hour storm depth is 14 inches, as specified by the Hydrometeorological Branch of the National Weather Service. These data are for a local storm, rather than for a regional PMF event. The former was used for onsite drainage design, whereas the latter was used for major flood protection design (Project Management Corporation, 1982).

3.3 SURFACE HYDROLOGIC EVENTS

3.3.1 PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

The PMF would result from an occurrence of the probable maximum storm as defined by the National Weather Service. The flood flow from this storm defines an upper limit of potential flooding at the site from meteorological considerations. The combination of events postulated to form the PMF, as specified in the CRBRP PSAR, include a main storm representing the maximum rainfall volume that the regional climate can produce; an antecedent storm amounting to 40% of the main storm volume; centering of the storm to produce the maximum flood level at the site; and wave runoff caused by a coincident 40-mile-per-hour wind from the most critical direction. Predicted water surface elevations at the site for the PMF are discussed in Sections 3.3.1.5 and 3.3.1.6, and are summarized in Table 3-2.

3.3.1.1 Probable Maximum Precipitation

The PMP for storms creating maximum flood conditions at the site has been defined by the National Weather Service In Hydrometeorological Reports No. 41 and 45 (U.S. Weather Bureau, 1965, 1969). In addition, PMP information in Report 45 was extended to cover the entire Tennessee River watershed above Watts Bar Dam and to provide PMP for the Clinch River watershed (Riedel, 1973). These reports provide depth-area-duration characteristics of rainfall, seasonal variations in precipitation, and antecedent storm potential. Snowmelt is not a factor in generating maximum floods at the site because the watershed is located in a temperate zone.

Four basic storms and five isohyetal patterns have been analyzed to determine PMP. The PMP will result from a storm covering 7,980 square miles of the Tennessee River watershed above Chattanooga. The critical storm center would be located at Bull Cap, Tennessee, about 50 miles northeast of Knoxville. The event sequence includes an antecedent storm (40% of the main storm) yielding an average of 6.9 inches of rain in 3 days, followed by a 3-day dry period. The main storm would then yield an areal average of 17.2 inches of rain in the next 3-day period. The probable maximum storm depths superimposed on the Tennessee River watershed above Watts Bar Dam are shown in Figure 3-3. Analysis of the seasonal variation of precipitation indicated the PMP occurrence to be in mid-March (Project Management Corporation, 1982).

3.3.1.2 Precipitation Losses

Multivariate relationships, developed from daily TVA operations, allowed precipitation excess to be computed directly. Precipitation losses were computed as the difference between rainfall and precipitation excess. Average storm rainfall and precipitation excess for the Clinch River watershed were estimated to be 6.9 and 4.6 inches for the antecedent storm, and 17.2 and 15.4 inches for the main storm, respectively. Precipitation excess is related to rainfall, week of the year, geographic location, and antecedent precipitation index (API). In establishing the PMP, a median API is used for the antecedent storm. The precipitation excess of the main storm is unaffected by this initial condition because of the large magnitude of this event.

3.3.1.3 Runoff Model

Several numerical models were assembled and used, singularly and in combination, in the CRBRP PSAR (Project Management Corporation, 1982) to simulate runoff and flow in streams and reservoirs in the Tennessee River watershed above Watts Bar Dam. These models include unit hydrographs, level pool storage and channel routing routines, and an unsteady stream flow model.

Runoff and local inflow to streams were computed by deriving unit hydrographs for each of 41 subareas in the Tennessee River watershed above Watts Bar Dam. The subareas are shown in Figure 3-4. Unit hydrographs were based on data for the largest event recorded in that area, following

SOURCE: Project Management Corporation (1982)

Figure 3-3 - Probable Maximum Storm Depth (inches)
March Storm (7,980 sq. mi.) Centers at Bull Cap, Tennessee

SOURCE: Project Management Corporation (1982)

Figure 3-4 - Tennessee River System Hydrologic Model Subareas

procedures described by Newton and Vinyard (1967). Where data were unavailable, synthetic unit hydrographs were developed, using procedures recommended by the U.S. Army Corps of Engineers (U.S. Army Corps of Engineers, 1953).

The Goodrich semigraphical approach and level pool storage were used to route flows through tributary reservoirs, including Norris Reservoir. The unsteady routing method used by Garrison, Granju, and Price (1969) was used to route flows through Fort Loudoun-Tellico, Watts Bar, and Melton Hill Reservoirs. Boundary conditions for these models were unit hydrographs and local inflow at upstream boundaries, and operating rules or rating curves at downstream boundaries. Each of the reservoirs was modeled and calibrated separately, and then verified with floods of record.

The models were then integrated and used to simulate the PMP storm described in Section 3.3.1.1, above. The result was a runoff model that included the entire 17,310-sq.-mile Tennessee River watershed above Watts Bar Dam. Subarea flows were combined with appropriate time sequencing and channel routing to compute inflows to upstream reservoirs. Floods were routed through the reservoirs; resulting outflows were combined with additional local inflows and carried downstream, using appropriate time sequencing or routing procedures, discussed above.

Verification of PMF flow simulations was not possible because no data exist for flows of this magnitude. However, a series of steady flows of this magnitude were simulated for the Watts Bar and the Melton Hill Reservoirs by both the unsteady flow model and by steady-state methods.

3.3.1.4 Probable Maximum Flood Flows

Based on the results of the unsteady flow model discussed in Section 3.3.1.3, the estimated PMF peak discharge at the site caused by the PMP storm is 258,000 cfs. The corresponding water surface elevations at the site are listed in Table 3-2 and discussed below in Section 3.3.1.5. The peak discharge is the result of the failure of the concrete, nonoverflow section of Melton Hill Dam. The flood would also result in the overtopping of the earth embankments at Fort Loudoun and Tellico Dams upstream, and at Watts Bar Dam downstream. The estimated discharge hydrograph at the site is shown in Figure 3-5.

This estimate is made by considering TVA flood operating procedures for the reservoir system. Historic mid-March reservoir levels before the antecedent storm were assumed. Approximately 51% of the flood detention capacity of the reservoir system was occupied at the beginning of the main storm event. All of the gates on tributary reservoirs were found to be operable throughout the storm. Turbine discharge was not considered at any of the main river or tributary dams.

3.3.1.5 Water Level Determinations

The unsteady flow model for the Tennessee River watershed discussed in Section 3.3.1.3 computes both discharge and water surface elevation

SOURCE: Project Management Corporation (1982)

Pe = Precipitation Excess

Figure 3-5 - Clinch River Probable Maximum Flood Discharge Hydrograph

simultaneously. The elevation hydrographs at both the upstream (CRM 18) and downstream (CRM 16) boundaries of the site are shown in Figure 3-6. Estimated peak water surface elevation at CRM 16 is 777.2 feet and at CRM 18 is 778.8 feet, respectively (see Table 3-2).

3.3.1.6 Coincident Wind-Wave Activity

A 40-mile-per-hour wind coincident with the arrival of the crest of the flood wave at the site was assumed to be the maximum wind-wave activity concurrent with PMF flow levels. A 40-mile-per-hour wind of sufficient duration to produce maximum wave runoff in March was considered rare. In addition, the wind would have to occur after the main storm has ended to be coincident with the crest of the flood wave (Project Management Corporation, 1982). The wind wave would be a third major event in the PMF sequence.

For a 40-mile-per-hour overland wind, 99.6% of the waves would be less than 2.4 feet high from crest to trough. This produces a maximum water surface elevation of 778.8 feet at CRM 16. Wave runoff above peak flood elevation (777.2 feet) would be 2.8 feet to an elevation of 780.0 feet on a smooth 3:1 slope, and 3.8 feet to an elevation of 781.0 feet on a vertical wall (see Table 3-2) (Project Management Corporation).

3.3.2 POTENTIAL DAM FAILURES (SEISMICALLY AND OTHERWISE INDUCED)

The site is located in the Southern Valley and Ridge tectonic province, and is potentially subject to moderate earthquake forces. An analysis, considering flooding at the site caused by seismically induced dam failure alone and during storm events, is based on work done in the Final Safety Analysis Report (FSAR) for the Sequoyah Nuclear Plant (Tennessee Valley Authority, 1973), modified to apply to the CRBRP (PSAR Section 2.2.4) (Project Management Corporation, 1982). Two seismic events were defined for the CRBRP. The first is a safe shutdown earthquake (SSE) generating a horizontal acceleration of 0.25g at the rock foundation. The second seismic event is an operational basis earthquake (OBE) generating a horizontal acceleration of 0.125g at the rock foundation.

3.3.2.1 Reservoir Description

See Section 3.1.2 for descriptions of the 11 dams on the Tennessee and Clinch Rivers upstream of the Watts Bar Dam that influence flood levels at the site. The major characteristics of these TVA dams and reservoirs are summarized in Table 3-3.

3.3.2.2 Dam Failure Permutations

In this section, both hydrologic and combined seismic and hydrologic failure scenarios are discussed. The most severe scenario is from a combined failure. The most severe hydrologic failure is the PMF scenario for a PMP storm, discussed in Section 3.3.1. Predicted water surface elevations are summarized in Table 3-2. The specifics of the dam failure analysis for this event are discussed below.

SOURCE: Project Management Corporation (1982)

Pe = Precipitation Excess

Figure 3-6 - Clinch River Probable Maximum Flood Elevation Hydrograph

Table 3-3 - Facts About Relevant TVA Dams and Reservoirs^a

<u>Parameter</u>	<u>Major River Projects</u>			<u>Tributary Projects</u>	
	<u>Watts Bar</u>	<u>Fort Loudoun</u>	<u>Melton Hill</u>	<u>Norris</u>	<u>Tellico</u>
River	Tennessee	Tennessee	Clinch	Clinch	Little Tennessee
State	Tennessee	Tennessee	Tennessee	Tennessee	Tennessee
County	Meica/Rhea ^C	Loudoun	Loudoun/ Roane ^C	Anderson/ Campbell ^C	Loudoun
Nearby city	Spring City	Lenoir City	Oak Ridge	Knoxville	Lenoir City
Type of dam ^e	CGE	CGE	CG	CGE	CGE
Concrete in dam, locks, and powerhouse (cu yd)	480,200	586,700	246,800	1,002,300	78,000
Earth and/or rockfill (cu yd)	1,210,000	3,594,000	-	81,700	2,700,000
Max height (ft) ^b	112	122	103	265	129
Length (ft)	2,960	4,190	1,020	1,860	3,238
Drainage area above dam (sq. mi.)	17,310	9,550	3,343	2,912	2,627
Length of lake (mi.)	72.4	55.0	44.0	72.0	33.2
Maximum width of lake (mi.)	1.3	0.7	0.8	1.2	1.3
Area of lake at full pool (acres)	39,000	14,600	5,690	34,200	15,860
Shoreline at full pool (mi.)	783	360	173	800	373
Lake elevation (ft MSL)					
Ordinary minimum	735	807	709	930	807
Top of gates	745	815	790	1,034	815
Full pool ^g	741	813	795	1,020	813
Lake volume (acre-ft)					
Ordinary minimum el.	796,000	282,000	94,500	290,000	320,800
Full pool elevation	1,175,000	393,000	126,000	2,555,000	447,300

Table 3-3 (Contd)

Parameter	Major River Projects			Tributary Projects	
	Watts Bar	Fort Loudoun	Melton Hill	Norris	Tellico
Useful controlled storage (acre-ft)	379,000	111,000	31,500	2,265,000	126,000
Construction started	07-01-39	07-08-40	09-06-60	10-01-33	03-15-67
Closure	01-01-42	08-02-43	05-01-63	03-04-36	11-29-79
Construction completed (first unit on line)	02-11-42	11-09-43	07-03-64	07-28-36	^f
Cost of plant in service (06-30-07), millions ^d	35.6	42.4	36.2	33.3	136.9
Ultimate generating cap kW	150,000	131,190	72,000	100,800	^f
Number of units	5	4	2	2	
Lock size (ft)	60 x 360	60 x 360	75 x 400		^f
Lock maximum lift (ft)	70	80	60		

^aSource: Tennessee Valley Authority, 1973.

^bFoundation to operating deck.

^cRiver is county line.

^dOriginal construction or acquisition cost, including switchyard, subsequent acquisitions, retirements, and reclassifications. Includes estimated costs of projects under construction.

^eAbbreviations: CG - concrete gravity dam. CGE - concrete gravity dam with earth embankments.

^fTellico project has no lock or powerhouse. Streamflow through navigable channel to Fort Loudoun Reservoir will increase average annual energy through Fort Loudoun powerhouse by 200 million kWh.

^gFull pool elevation is the normal upper elevation to which the reservoirs may be filled. Where storage space is available above this level, additional filling may be made as needed for flood control.

All of the dams, both upstream and downstream, which are close enough to have significant influence on flood levels at the site, were examined for potential failure during the PMF (Project Management Corporation, 1982). It was concluded that the only potential failures during the PMF would be of the earth embankments at Fort Loudoun-Tellico and Watts Bar Dams caused by erosion from overtopping, and part of the concrete, nonoverflow portion of Melton Hill Dam above an elevation of 774.5 feet. The seismic dam failure analyses contained in the FSAR for the Sequoyah Nuclear Plant (Tennessee Valley Authority, 1973) with flood conditions specifically applicable to the CRBRP were performed. The analysis included single and multiple failures of Watts Bar Dam and all 11 dams in SSE and OBE scenarios (Project Management Corporation, 1982).

Assuming an OBE, the predicted seismic dam failure combinations with the potential to generate maximum flood levels at the site are Norris Dam singularly, and Cherokee and Douglas Dams concurrently. For an SSE, the combinations of concern are the failure of Norris Dam singularly, and the concurrent failure of Norris-Cherokee-Douglas and Norris-Douglas-Fort Loudoun-Tellico. Flow and elevations for potentially critical situations are summarized in Table 3-4. An OBE coincident with a 1/2 PMF flood event, resulting in the postulated failure of the Norris Dam, was determined to be the critical dam failure situation. The maximum estimated water surface elevation at Clinch River mile 16 is 798.2 feet, and at river mile 18 is 804.3 feet, excluding effects of wind waves (see Table 3-2) (Project Management Corporation, 1982).

In the Norris Dam failure analyses, Melton Hill Dam was postulated to fail when the flood wave reached headwater elevation 804 feet. Watts Bar Dam would be overtopped and the embankment breached from a Norris Dam failure during an OBE. However, failure would occur after the Clinch River flood peak had passed the site, and would have no lowering effect. Therefore, site flood levels were computed as if Watts Bar embankment did not fail (Project Management Corporation, 1982).

3.3.2.3 Unsteady Flow Analysis of Potential Dam Failures

An unsteady flow analysis of the postulated OBE plus 1/2 PMF failure of Norris Dam was performed for the CRBRP. The Norris Dam failure was judged to be the most severe of the failure scenarios (see Section 3.3.2.2). The unsteady flow model developed for the Norris Reservoir (see Section 3.3.1.3) had sufficient accuracy to determine discharges for breach simulations (Project Management Corporation, 1982). The boundary condition at the dam was supplied by data from 1:150 scale hydraulic model studies performed and verified at the TVA Engineering Laboratory. Results of the model match to within 1 foot those obtained by using simplified routing techniques for the 1/2 PMF. The analysis is performed by assuming complete and instantaneous disappearance of Melton Hill Dam, allowing the unsteady flow models for both reservoirs to be coupled. Corresponding water surface elevations are listed in Table 3-2 and discussed in Section 3.3.2.4.

Table 3-4 - Floods From Postulated Seismic Failure of Upstream Dams

OBE Failures With 1/2 PMF	Headwater		Peak Flow (cfs)	Conditions at Site Elevation	
	Watts Bar ^a	Norris		Mile 16	Mile 18
Norris	765.8	1,035	921,000	798.2	804.3
Cherokee-Douglas	765.0	--	35,000	765.4 ^c	765.6 ^c
SSE Failures With 25-year Flood					
Norris	754.5	1,027	744,000	790.5	796.3
Norris-Cherokee-Douglas ^b	754.5	1,024.3 ^d	770,000 ^d	791.6	797.7
Norris-Douglas-Fort Loudoun-Tellico ^b	764.5	1,024.3 ^d	754,000 ^d	791.6	797.7

^aLake level at mouth of Clinch River concurrent with peak CRBRP stage.

^bTaken from recent analyses for Sequoyah Nuclear Plant.

^cEstimated by steady flow backwater with starting elevation 765 at mouth of Clinch River from unsteady flow analysis.

^dDifference in Norris headwater elevations and peak site flows from that for Norris single failure results from use of the Sequoyah watershed 25-year flood.

NOTE: Postulated results are from the PSAR for the Clinch River Breeder Reactor Project (Project Management Corporation, 1982).
Results will be evaluated for applicability to an MRS Facility.

Routings of seismic surges upstream of Watts Bar Reservoir were made by using short-interval storage routing techniques. It was concluded that these define Watts Bar Lake inflows with sufficient accuracy to demonstrate that Norris Dam OBE failure is the controlling situation (Project Management Corporation, 1982).

The Melton Hill Dam failure under PMF flow was also simulated with an unsteady flow model. Headwater and tailwater rating curves derived for the failure configuration provided boundary conditions at the dam.

3.3.2.4 Water Level at Plantsite

The unsteady flow analyses discussed in Section 3.3.2.3 provide simultaneous computation of discharge and water surface elevation. The results for PMF conditions are presented in Section 3.3.1.5. The discharge and elevation hydrographs for the controlling OBE failure combined with a 1/2 PMF flood are shown in Figure 3-7. The peak flow for this event at the site is estimated to be 921,000 cfs. Corresponding maximum water surface elevation at CRM 16 is 798.2 feet, and at CRM 18 is 804.3 feet (see Table 3-2). Of the various scenarios making up the sensitivity analysis, only the hypothetical worst-case instantaneous disappearance of Norris Dam resulted in flood levels at the site that were higher than the OBE plus 1/2 PMF failure. These were predicted to be 811.0 feet at CRM 16 and 818.0 feet at CRM 18 (Project Management Corporation).

A coincidental 40-mile-per-hour wind was applied to the site for the fetch orientations shown in Figure 3-8. Critical directions are from the northeast for CRM 18 and from the southwest for CRM 16. Effective fetch lengths are both 0.8 mile. For these conditions, 99.6% of the waves are predicted to be less than 3.0 feet from crest to trough. Runup would be an estimated 3.6 feet on a smooth 3:1 slope and 4.9 feet on a vertical wall. Resulting maximum water surface elevations for an OBE plus 1/2 PMF condition with this wind are shown in Table 3-2.

3.3.3 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

The site is located in an inland drainage basin. No ocean or tidal effects are present in the basin. Fetches of sufficient length to produce significant wind-generated waves exist in only two directions (see Figure 3-8). Seiche flooding is minor. The only surge effects present are caused by the operation of turbine and gate control structures on dams and reservoirs. The only natural surge effects would be caused by rare landslide events along the shores of Watts Bar Lake. Neither surge nor seiche flooding will significantly affect the site (Project Management Corporation, 1982).

3.3.4 PROBABLE MAXIMUM TSUNAMI FLOODING

Because of the inland location of the site, tsunami flooding is not possible.

SOURCE: Project Management Corporation (1982)

Figure 3-7 - Clinch River Discharge and Elevation Hydrographs for OBE Failure of Norris Dam

SOURCE: Project Management Corporation (1982)

Figure 3-8 - Wind-Wave Fetch Orientation at Clinch River MRS Site for Extreme Event Analyses

3.3.5 ICE FLOODING

Significant amounts of ice do not form in the region; ice jams seldom occur, and are not a source of major flooding because of the temperate climate. The probability of severe icing was expected to be lower today than in the past because of fluctuating water levels from Watts Bar (closed 1942) and Melton Hill (closed 1963) Reservoirs. In addition, water depth near the site has been increased 19 feet with the completion of Watts Bar Reservoir, requiring a greater mass of water to be frozen to create a jam. Finally, releases of bottom water from Norris and Melton Hill Dams warm the Clinch River.

Records of icing are limited. The U.S. Weather Bureau records for Knoxville indicate that the Tennessee River has frozen over 16 times, and floating ice was observed six times between 1840 and 1940. The most severe event, in December-January 1917-18, jammed the Tennessee River for almost 2 weeks. In late January, rising temperatures and rain produced flooding on the Clinch River. There was no record of ice jamming. An ice jam sufficient to cause flooding of the 820-foot proposed plant elevation is inconceivable because a 95-foot-thick ice jam would be required to damage the plant directly (Project Management Corporation, 1982).

3.3.6 FLOODING PROTECTION REQUIREMENTS

Flood protection design requirements include protection from all probable maximum and seismic events discussed in the preceding sections. The flood analyses detailed above indicate that a minimum elevation for facility structures be established at or above 820 feet to place all buildings above the maximum water level predicted by realistic failure scenarios.

SECTION 4

GROUNDWATER CHARACTERISTICS

The primary sources of information concerning the groundwater hydrology of the MRS Facility at the Clinch River site are the PSAR and ER performed for the CRBRP (Project Management Corporation, 1982 and 1975).

4.1 HYDROGEOLOGIC UNITS

Groundwater occurs in three hydrogeologic units at the site: an upper unit consisting of soil and residuum; a middle unit consisting of weathered rock; and a lower unit consisting of unweathered rock (Project Management Corporation, 1982). In the borings done for the CRBRP, it was found that the upper soil/residuum unit ranges from 1 to 38 feet in thickness. The weathered rock zone ranges from 0 to 73 feet thick in areas underlain by limestone and from 0 to 56 feet thick in areas underlain by siltstone. The soil/residuum and weathered rock are collectively referred to as overburden, and obtain a maximum total thickness of 108 feet at the site (Project Management Corporation, 1982).

Active groundwater flow occurs principally in the overburden material. Below the overburden, minor quantities of groundwater exist in the unweathered rock (Project Management Corporation, 1982).

Alluvial and terrace deposits occur on the Clinch River floodplain and at a few locations above the floodplain. Although these deposits are permeable, they do not constitute a significant hydrogeologic unit at the Clinch River site.

4.2 HYDROGEOLOGIC PROPERTIES

The hydraulic conductivity of the hydrogeologic units at the Clinch River site was measured in 22 CRBRP borings, which involved 117 packer tests (Project Management Corporation, 1982). Almost one-third of the hydraulic conductivity values were in the range of nonmeasurable to $1\text{E-}5$ cm/s, and more than 90 percent of the values were less than $1\text{E-}3$ cm/s. Packer test data indicate a tendency for the hydraulic conductivity to decrease with depth. The maximum values of hydraulic conductivity were not significantly different in the limestone and siltstone at the site (Project Management Corporation, 1982).

Because of problems in maintaining hole stability, only a limited number of packer tests were conducted in the residual soils or in the more weathered bedrock (Project Management Corporation, 1982). If more tests had been conducted in these materials, higher values of hydraulic conductivity would

probably have resulted. Therefore, it is likely that maximum values of hydraulic conductivity are greater than is suggested by the 90th percentile value of $1\text{E-}3$ cm/s.

4.3 PIEZOMETRIC CONDITIONS

Figure 4-1 shows the groundwater contours for relatively dry conditions that occurred on December 24, 1973. The contour map in Figure 4-2, for January 2, 1974, shows higher groundwater levels that occurred after a period of precipitation. On both maps, the arrows indicate the approximate direction of groundwater flow (for an assumed isotropic medium). The water-level depression in the approximate center of the figures coincides with an area of intense weathering that lies about one-third of the way upslope from a topographic low (Project Management Corporation, 1982).

Water levels in 33 observation wells at the site were measured on a regular basis from December 19, 1973, to April 1, 1974. During this period, water levels fluctuated by as much as 20 feet in response to precipitation. A rapid response of water levels to precipitation tended to be observed in wells located in areas of exposed bedrock on topographic highs. This is probably because of the proximity of these wells to local areas of groundwater recharge. Subsequent water-level monitoring across the site has suggested that water levels follow an annual cycle, with maximum levels observed during the months of January and February, and low levels generally occurring in the months of October and November (Project Management Corporation, 1982).

Eleven piezometers were completed in groups of twos and threes at four locations at the site. Water levels in these installations were measured on a regular basis from February to April 1974. Because piezometers within a group were completed at different depths, water-level data could be used to determine vertical hydraulic gradients. Downward vertical gradients were observed in piezometer groups located on topographic highs, indicating the potential for recharge to the unconfined flow system. Upward gradients were observed in a group located within the water-level depression northwest of the proposed location of the MRS Receiving and Handling Building, indicating that groundwater discharge occurs in this area (Project Management Corporation, 1982).

4.4 GROUNDWATER FLOW PATHS

Groundwater movement at the Clinch River site is generally from topographically high areas to topographically low areas. This pattern is modified by the extent of weathering of the bedrock. All groundwater within the site ultimately discharges into the Clinch River. Because sound rock is more often encountered in the core of ridges at higher elevations than in the valley floors, it is unlikely that significant interbasin groundwater flow occurs. Thus, major surface water divides are assumed to coincide with groundwater divides (Project Management Corporation, 1982).

- Groundwater observation well at which data were collected.
- Flow direction
- Groundwater contour, dashed where inferred.

SOURCE: Project Management Corporation (1982)

NOTE: Clinch River site groundwater conditions are presented for various stages during construction of the CRRP. These conditions may change significantly upon completion of redress operations.

Contour interval 10 feet.

0 200 400 600 800
FEET

Figure 4-1 - Site Groundwater Contours
December 24, 1973

8

Figure 4-2 - Site Groundwater Contours

January 2, 1974

4.5 WATER QUALITY

A total of 18 groundwater samples were obtained from five observation wells and 11 piezometers at the site. The chemical and physical quality of groundwater at the site is not significantly different from the regional chemical and physical quality. Total dissolved solids range from 300 to 550 mg/l and pH varies from 7 to 8.5. Water is generally suitable for human consumption, but may require softening for industrial uses (Project Management Corporation, 1982).

SECTION 5

BASIC GEOLOGIC CHARACTERISTICS

The following discussion of the geology of the Clinch River site is based primarily on data and descriptions given in the CRBRP PSAR (Project Management Corporation, 1982). Additional reports that are based on investigations subsequent to the CRBRP PSAR (Project Management Corporation, 1982) were also used in the preparation of this section. Some of the data and interpretations of the geology at the site may be revised as a result of the site investigations for the MRS Facility.

5.1 GEOMORPHOLOGY

5.1.1 REGIONAL PHYSIOGRAPHIC SETTING

The Clinch River site is located in the southeast portion of the Valley and Ridge physiographic province of the eastern United States (Figure 5-1). The Valley and Ridge province is about 25 to 50 miles wide, and extends about 500 miles from Virginia to northeast Alabama. The province is characterized by narrow, elongate, northeast-trending ridges with slightly broader intervening valleys. The trend of the ridges and valleys reflects the orientation of the inclined sedimentary strata that underlie the province. The ridges tend to be underlain by sandstone, siltstone, and siliceous dolomite, and the valleys tend to be underlain by shale and limestone.

The drainage pattern in the Valley and Ridge province tends to follow the northeast-trending valleys. Major streams cross this trend for short distances because of entrenchment of ancient stream courses, which have gradually eroded downward to their present levels. Remnants of river terraces that represent older floodplains are often found more than 100 feet above the present floodplains (Project Management Corporation, 1982).

The Valley and Ridge province is bordered on the southeast by the Blue Ridge and Piedmont physiographic provinces and by the Appalachian Plateaus physiographic province to the northwest (Figure 5-1). The Blue Ridge province is about 15 to 70 miles wide, extends 600 miles from Pennsylvania to Georgia, and is characterized by closely spaced, northeast-trending ridges. The province encompasses the most rugged terrain and the highest elevations in the eastern United States, with surface elevations ranging from 1,500 to more than 6,500 feet (Project Management Corporation, 1982).

The Piedmont province is about 40 to 125 miles wide, and extends about 1,000 miles from Alabama to New York. Surface elevations in the southern portion of the province range from about 1,000 feet on the western boundary, with the Blue Ridge physiographic province, to about 500 feet on the eastern boundary near the Fall Line. Elevations gently decrease northward, and are about 100 to 500 feet near the northern terminus of the province (Project Management Corporation, 1982).

The Appalachian Plateaus physiographic province (Figure 5-1) is about 20 to 200 miles wide, and extends 1,000 miles from Alabama to New York. The south-

SOURCE: Project Management Corporation (1982).

Figure 5-1 - Regional Physiographic Provinces

eastern margin occurs at the abrupt topographic rise of the Allegheny Front. In Tennessee, this is referred to as the Cumberland Escarpment. Surface elevations in the Appalachian Plateaus province range from about 1,000 to 3,000 feet. The topography is gently sloping to undulating, with localized mountainous areas. The province is underlain by alternating sandstone and shale, which have been gently folded into a broad syncline with a few superimposed elongate anticlinal folds. Paleozoic bedding plane faults underlie much of the province (Project Management Corporation, 1982).

5.1.2 SITE GEOMORPHOLOGY

The geomorphology of the Clinch River site conforms with the general characteristics of the Valley and Ridge province. The topography in the vicinity of the site is characterized by northeast-trending subparallel ridges, with intervening valleys. The major ridges in the site area are Chestnut Ridge to the northwest and Dug and Hood Ridges to the southeast. The ridge crests range in elevation from 900 to 1,200 feet. Poplar Springs Valley and Bethel Valley separate these ridges (Figure 5-2). These valleys contain rolling hills, and range in elevation from 750 to 850 feet.

The Clinch River site is located on the southeastern flank of Chestnut Ridge, on a peninsula formed by an incised meander of the Clinch River. Within the site boundaries, Chestnut Ridge consists of a major ridge along the northwest edge of the site, with subordinate ridges (with crest elevations of about 900 feet) separated by minor valleys. A large portion of the southern part of the peninsula has been graded as a result of the CRBRP construction activities. The preredress elevations across the graded portion of the site range from about 840 to 720 feet. The CRBRP redress will result in the filling of the major CRBRP excavations. Upon completion of the redress, elevations across the graded portion of the site will range from 840 to 780 feet (Figure 2-3). The average slope from the Clinch River, at elevation 735 to 741 feet, to the graded portion of the site is about 13 degrees.

The Clinch River floodplain borders the western and southern sides of the peninsula, and is essentially absent on the eastern side of the site because of the presence of steep bluffs. The floodplain is relatively flat to gently sloping, and extends up to about elevation 752 feet. The maximum width of the floodplain is about 500 feet.

Active geomorphic processes occurring on the site include sheet erosion, localized gully erosion, soil creep on steeper slopes, and subsidence related to dissolution of the carbonate bedrock, which is locally accompanied by subsurface sediment transport through open solution cavities near the bedrock surface. No perennial streams occur on the site. Surface flow occurs only after periods of heavy rainfall (Project Management Corporation, 1982). Surface drainage from Chestnut Ridge occurs along northeast-trending ravines, and reflects the control of the underlying rock strata. Drainage in the Poplar Springs Valley portion of the site does not appear to be strongly structurally controlled.

In the site areas underlain by limestone and dolomite, drainage also occurs vertically. Sinkholes were noted in the northwest portion of the site, where

Figure 5-2 - Site Topography

the Knox Group crops out, and in the southeast portion of the site, where the Chickamauga Units B and Chickamauga Undifferentiated crop out (Project Management Corporation, 1982). The sinkholes in these areas can occur as (1) small-diameter vertical holes (generally less than 1 foot); (2) much larger features, with surface expressions as asymmetrical depressions that are generally less than 100 feet across and 10 to 15 feet deep; or (3) gentle depressions, which are generally several hundred feet across and 10 to 15 feet deep (Project Management Corporation, 1982).

5.2 REGIONAL GEOLOGY

5.2.1 GEOLOGIC HISTORY

During the early Paleozoic Era, the rocks currently exposed in the Appalachian Mountains were deposited in an elongated, narrow trough overlain by a shallow sea. Up to 40,000 feet of sediment accumulated in the gradually subsiding trough, the Appalachian Geocline, which consisted of two portions, a miogeocline and a eugeocline. The miogeocline occupied the northwestern portion of the Appalachian Geocline, and is represented by the present Valley and Ridge province. The miogeocline was a slowly subsiding area overlain by shallow water in which well-sorted sandstone, shale, and limestone accumulated. The Blue Ridge and Piedmont provinces to the east represent the eugeocline, a more rapidly subsiding, deeper water area. In the eugeocline, poorly sorted sandstone (including graywacke), shale, and some limestone, alternating with basaltic lava flows, were deposited (Project Management Corporation, 1982).

Major crustal movements in the Appalachian Geocline during the Paleozoic Era occurred primarily in the eugeoclinal area southeast of the miogeocline. Two significant disconformities (surfaces that represent missing intervals in the rock record) are recognized in the Paleozoic rocks of the Valley and Ridge province of east Tennessee. The lower disconformity separates the Early Ordovician Upper Knox Group from the Middle Ordovician Chickamauga Group. The upper disconformity separates the Early Silurian Rockwood Formation from the Devonian-Mississippian Chattanooga Shale. Generally the Strata above and below the disconformities are parallel, and only locally do slight angular unconformities occur at these contacts. This suggests that the depositional hiatus in the Valley and Ridge province during the early Paleozoic Era were simply the result of the rising of the land caused by broad upwarps or by a lowering of the sea level (Project Management Corporation, 1982).

During the early and middle Paleozoic, one and possibly two orogenies affected the eugeocline. The eugeoclinal sediments of the present Blue Ridge and Piedmont provinces were severely deformed, intruded with magmas, and metamorphosed during these events. Although the sediments of the Valley and Ridge province were not strongly deformed during these early and middle Paleozoic orogenies, the deformation and uplift that were occurring to the east are reflected in their sedimentary record. The Devonian Period is poorly represented in east Tennessee, but an eastward thickening and coarsening wedge of Devonian clastic formations in the northern Appalachians, in New York and Pennsylvania, record a Devonian orogeny.

Mississippian age formations also coarsen eastward, indicating a continued source area to the east, in the region of the Blue Ridge and Piedmont provinces (Project Management Corporation, 1982).

By Pennsylvanian time, the upland sediment source areas had moved westward, and a thick accumulation of alternating marine and nonmarine sediments was deposited. These sediments occur primarily in the Cumberland Plateau, but at least one occurrence of Pennsylvanian sediments in the Valley and Ridge province near Chattanooga suggests that they were originally more extensive. No Permian strata occur in east Tennessee (Project Management Corporation, 1982).

Deformation of the sediments that make up the present Valley and Ridge province was restricted to folding and major thrust faulting that occurred near the end of the Paleozoic Era. Widespread metamorphism did not accompany the deformation in the Valley and Ridge province. The entire region was also uplifted at this time (Project Management Corporation, 1982).

Since the end of the Paleozoic Era, the Appalachian region has remained relatively stable. Erosion and soil development have been the primary active processes that have occurred in the region during the Mesozoic and Cenozoic Eras (Project Management Corporation, 1982).

5.2.2 STRATIGRAPHY AND LITHOLOGY

The regional stratigraphic column for the site vicinity is shown in Figure 5-3. The Rome Formation and the Conasauga, Knox, and Chickamauga Groups make up the majority of the bedrock in the Valley and Ridge province in Tennessee, occurring as repeated northeast-trending rock assemblages. The Rome Formation is middle Cambrian in age, and has a maximum exposed thickness of about 1,200 feet. The total thickness of the Rome Formation is unknown because the base of the formation is always faulted. The Rome Formation consists predominantly of red, green, and yellow shale; siltstone; and sandstone, with minor amounts of gray dolomite. The Conasauga Group is late Cambrian in age, and is about 2,000 feet thick. It consists predominantly of alternating gray shale and limestone. The amount of limestone decreases to the northwest, where the Conasauga Group is nearly all shale. The Knox Group is late Cambrian and early Ordovician in age, and is 2,500 to 3,000 feet thick. The Knox Group consists predominantly of chert-bearing dolomite, with some limestone. The Chickamauga Group is middle Ordovician in age, and ranges in thickness from about 8,000 feet in the southeastern portion of the Valley and Ridge province to 2,000 feet in the northwest portion of the province. It consists of alternating layers of gray and maroon limestone, calcareous siltstone, and shale. Alternating sandstone, shale, and limestone formations of late Ordovician to Pennsylvanian age, and small amounts of early Cambrian dolomite, also occur within the Valley and Ridge province (Project Management Corporation, 1982).

5.2.3 REGIONAL TECTONICS

Deformation of the sediments within the Appalachian Geocline began in the middle Ordovician during the Taconic orogeny. The greatest deformation took

LOWER MISSISSIPPIAN		FORT PAYNE CHERT Limestone, extremely siliceous; weathers to beds of dense blue-white to white chert, less than 80 feet thick.
UPPER DEVONIAN		CHATTANOOGA SHALE AND MAURY FORMATION Shale, black, bituminous, fissile; disconformity at base, 25 feet thick.
LOWER SILURIAN		ROCKWOOD FORMATION Siltstone, shale, sandstone; brown, tan, and yellow, thin beds of hematitic limestone in upper half; 690 feet thick.
UPPER ORDOVICIAN		SEQUATCHIE FORMATION Silty and shaly limestone, maroon, maroon and green, and grey; maroon calcareous, siltstone, 360 feet thick.
	REEDVILLE SHALE Shale, with thin limestone lenses in lower half; tan to orange-brown, with black stains; about 200 feet thick.
MIDDLE ORDOVICIAN		CHICKAMAUGA LIMESTONE Shaly and silty limestone; cherty, dense to crystalline; gray to gray-blue; fossiliferous; thin to medium bedded; disconformity within formation and at base; about 2200 feet thick.
LOWER ORDOVICIAN		KNOX GROUP Dolomite, cherty; dense to crystalline; light to medium gray; thin to massive beds; 3000 feet thick.
UPPER CAMBRIAN		CONASAUGA GROUP Shale, siltstone, with thin limestone layers in lower two-thirds, massive limestone in upper third; 1500 feet thick.
MIDDLE CAMBRIAN		ROME FORMATION Siltstone, shale, sandstone, variegated; thin to medium bedded; primary structures common, thickness unknown.
LOWER CAMBRIAN		

Figure 5-3 - Regional Stratigraphic Column

place in the southeastern portion of the southern Appalachians, in the Piedmont and Blue Ridge provinces. Here the eugeoclinal sediments were metamorphosed and injected by magmas. The early Paleozoic deformation, however, was confined to the eugeoclinal area, and the only record of this event in the Valley and Ridge province is the change from a western sediment source to an eastern sediment source. During the mid-Paleozoic, the Acadian orogeny took place in the northern Appalachians. It is controversial whether or not this orogeny extended into the southern Appalachians (Project Management Corporation, 1982).

The late Paleozoic Alleghenian orogeny affected the entire Appalachian Geocline, and is largely responsible for the current configuration of the southern Appalachians (Figures 5-4 and 5-5). During the Alleghenian orogeny, northwest-directed tectonic stress deformed the rocks of the Blue Ridge and Piedmont provinces, and thrust these rocks northwest along a major decollement. The miogeoclinal rocks of the Valley and Ridge province were also deformed at this time because of northwest-directed stress, forming a series of northwest-verging, fold-and-thrust blocks (Figure 5-6). The amount of deformation abruptly diminishes northwest of the Valley and Ridge province, where a broad syncline was developed in the Appalachian Plateaus and large lateral movements occurred along bedding planes. Deformation subsequent to the Alleghenian orogeny has not been recognized in the Valley and Ridge province (Project Management Corporation, 1982).

It is generally accepted that the thrust faults in the southern Appalachians do not extend into the basement, but that the thrust blocks are bounded by a lateral sole fault or decollement in some relatively incompetent formation above the crystalline basement (Figure 5-6) (Project Management Corporation, 1982). This is referred to as the thin-skinned structural concept. The consistent repetition of the Rome Formation as the basal formation of the thrust blocks supports the occurrence of a major decollement above the basement. Recent deep seismic profiling within the southern Appalachians has identified prominent, near-horizontal, seismic reflectors beneath the Valley and Ridge province at depths of 9,000 to 15,000 feet, that can be traced southeast beneath the Blue Ridge province (Cook, et al., 1983). These reflectors are interpreted to represent the decollement and the top of the basement in this area. These data further support the thin-skinned deformation concept.

The Clinch River site is located between two thrust faults that formed during the Alleghenian orogeny - the Copper Creek fault and the Whiteoak Mountain fault (Figure 5-7). Movement on these faults has not occurred since the end of the Alleghenian orogeny, 230 million years ago (Project Management Corporation, 1982).

5.2.3.1 Copper Creek Fault

The Copper Creek fault is approximately 100 miles long, and the Clinch River site is located near its midpoint. The Copper Creek fault trace occurs about 3,000 feet south of the site at its closest approach (Figure 5-7) (Project Management Corporation, 1982). In the site vicinity, the fault strikes N52°E and dips 25 degrees southeast (away from the site). Borings that have

Figure 5-4 - Regional Geologic Map

SOURCE: Project Management Corporation (1982)

Figure 5-5 - Regional Tectonic Map

SOURCE: Cook et al. (1979)

Figure 5-6 - Regional Geologic Cross Section

SOURCE: Project Management Corp. (1982)

Figure 5-7 - Area Structural Map

penetrated the fault indicate that the dip angle decreases with depth (Project Management Corporation, 1982). In the site area the Copper Creek fault places the Rome Formation over younger rocks of the Chickamauga Group. The fault has a horizontal displacement estimated in miles and a stratigraphic displacement of approximately 7,200 feet (McMaster, 1963). About 65 miles southwest of the site, the fault becomes a complex zone of deformation, and merges with the Whiteoak Mountain fault.

Representative samples of the mylonite and the adjacent hanging wall (Rome Formation) and foot wall (Chickamauga Group) rocks of the Copper Creek fault were dated as part of the CRBRP investigations. Whole rock potassium-argon dating of these samples gives ages of about 760 and 530 million years for the hanging wall and foot wall rocks, respectively. The mylonite gave an age of about 285 million years (Project Management Corporation, 1982). These dates are consistent with stratigraphic and structural investigations in the Valley and Ridge province, and suggest that the last active period of Valley and Ridge tectonism occurred during the late Paleozoic Era.

5.2.3.2 Whiteoak Mountain Fault

The Whiteoak Mountain fault consists of a main thrust fault and several subsidiary faults that branch from the main thrust. The fault extends southwest across Tennessee from near Clinton, Tennessee, about 4 miles northeast of the Oak Ridge Reservation. The Cambrian Rome Formation has been placed over rocks of Ordovician age along the fault. The closest approach of the Whiteoak Mountain fault system to the site is 1.7 miles to the northwest (Figure 5-7). The main fault and its subsidiaries dip to the southeast at 45 to 50 degrees, with the dip angle probably decreasing with depth. Outcrop data and a deep core hole drilled about 4 miles east of the CRBRP site indicate that the Whiteoak Mountain fault trace and its subsidiaries are more than 2,000 feet beneath the site (Project Management Corporation, 1982).

5.3 SITE GEOLOGY

The following discussion of the Clinch River site geology is based almost entirely on the investigations done for the CRBRP. In the current layout (Figures 2-3 and 2-4), approximately one-half of the MRS Facility will be located on the area that was graded as part of the cancelled CRBRP. The remaining MRS area extends to the northeast, toward Chestnut Ridge. A large amount of geologic data exist in the CRBRP area, especially where the reactor was to be located (Figure 5-8). North of the reactor location, data are sparse to nonexistent.

5.3.1 STRATIGRAPHY AND LITHOLOGY

The Knox Group and Chickamauga Group are the two major geologic units that underlie the site. Scattered rock outcrops of these units occur in the central portion of the site; however, the rock is generally covered by a veneer of residual soil, except in the southern portion of the peninsula and near the river, where older terrace and recent alluvial soils have been deposited. The areal distribution of these rock units is shown in Figure 5-9 and a cross section through the site is shown in Figure 5-10.

Figure 5-8 - Site Layout

Figure 5-9 - Site Geologic Map

Figure 5-10 - Site Geologic Cross Section

5.3.1.1 Knox Group

The Knox Group underlies the northern portion of the site (Figure 5-9), and consists predominantly of dolomite with lesser amounts of limestone and thick, chert-bearing sequences. The Knox Group conformably overlies the Conasauga Group and is disconformably overlain by the Chickamauga Group. Nineteen borings for the CRBRP investigations penetrated the upper portion of the Knox Group, and encountered predominantly light to medium gray, crystalline dolomite. Pink and green shale laminations were also observed in the core. Pockets of chert-bearing calcareous siltstone, similar to the basal portion of the Chickamauga Group, were encountered at the top of the Knox Group, and represent sediments deposited in irregularities on the pre-Chickamauga erosion surface (Project Management Corporation, 1982).

In the vicinity of the site, the Knox Group is divided into four units: the Copper Ridge Dolomite, the Chepultepec dolomite, the Longview Dolomite, and the Newala Formation (Butz, 1984; Ketelle and Huff, 1984). The Chepultepec Dolomite, the Longview Dolomite, and the Newala Formation are the formations of the Knox Group that will underlie the MRS facilities.

The Chepultepec Dolomite unit conformably overlies the Copper Ridge unit. The base of this formation occurs at a prominent layer of sandstone, about 15 feet thick, which can be traced throughout the area. Outcrops of the Chepultepec unit are scarce, and the thickness of the Chepultepec Dolomite is estimated to be approximately 750 feet (Butz, 1984; Ketelle and Huff, 1984).

The Longview Dolomite overlies the Chepultepec Dolomite, and is the most siliceous and cherty formation of the Knox Group. The Longview Dolomite typically produces large blocks of dense, tough, white chert, which may be bedded. This formation often caps steep ridges because of the abundance of chert. The Longview unit is about 250 feet thick (Butz, 1984; Ketelle and Huff, 1984).

The Newala Formation is the uppermost formation of the Knox Group, and consists of well-bedded to massive, fine-grained, light gray dolomite. In the lower part of the Newala Formation, limestone layers, interbedded with the dolomite, are dense to well-bedded, finely to coarsely crystalline, gray to blue-gray, silty, and slightly fossiliferous. The upper beds of the unit are commonly red, pink, and green in color. Chert, while not abundant, occurs along bedding planes and as scattered nodules. The chert is dense; white, gray, black, or red in color; and chalcedonic or opaque. The thickness of the Newala Formation is estimated to be between 900 and 1,000 feet (Butz, 1984; Ketelle and Huff, 1984).

The top of rock identified in the CRBRP borings, in the areas underlain by the Knox Group, is highly variable, with soil thicknesses ranging from 0 to 78 feet (average 12 to 20 feet). The areas underlain by the Knox Group contain many buried pinnacles, sinkholes, and other forms of karstification. These are especially prominent in the Newala Formation at the contact with the overlying Chickamauga, where the Knox has been subjected to two stages of erosion and development of karst topography (Stone and Webster, 1984).

The first period of karstification occurred during the early Ordovician, before deposition of the Chickamauga Group. The second period of karstification began upon uplift of the Appalachians, and has continued to the present. The residual soil that overlies the Knox Group is further discussed in Section 5.3.1.4, and the solution-related features are discussed in Section 5.3.4.

5.3.1.2 Chickamauga Group

The Chickamauga Group consists of alternating layers and laminations of maroon and gray siltstone, limestone, and shale. Thin layers and nodules of chert occur in the lower portion of the group. The Chickamauga Group underlies the southeastern half of the Clinch River site, and is overlain by the Rome Formation to the southeast of the Clinch River. The boundary between the Chickamauga Group and the Rome Formation is the Copper Creek fault, which truncates the upper portion of the Chickamauga Group. The total stratigraphic thickness of the Chickamauga Group at the site is approximately 1,700 feet. The lower portion of the Chickamauga Group is subdivided into Units A and B (Project Management Corporation, 1982).

Unit A

Unit A is the basal portion of the Chickamauga Group, and is subdivided into the Unit A Lower Siltstone, Unit A Limestone, and Unit A Upper Siltstone (Figure 5-9). The Unit A Lower Siltstone consists of alternating layers and laminations of maroon and gray, calcareous siltstone; gray limestone; and maroon, gray, and black chert layers and nodules. The constituent percentages of the alternating layers of the Chickamauga units were determined during the CRBRP investigations by measurement of core samples and interpretation of geophysical logs. The Unit A Lower Siltstone consists of about 45 percent siltstone, about 50 percent limestone, and about 5 percent chert (Project Management Corporation, 1982). The limestone layers are about 80 to 98 percent carbonate minerals and 2 to 10 percent clay minerals, with the remainder being finely disseminated quartz. The thickness of the Unit A Lower Siltstone is variable because its lower boundary is the undulating, upper surface of the Newala Formation (Knox) (Figure 5-9). The stratigraphic thickness of the Lower Siltstone, determined from the borings, ranges from 151 to 207 feet averaging about 175 feet (Project Management Corporation, 1982).

The Unit A Limestone is light to dark gray limestone, with laminations and layers of maroon and gray, calcareous siltstone. About 75 percent of this unit is limestone and 15 percent is siltstone. Chert nodules constitute about 10 percent of the unit. The limestone contains about 5 to 45 percent chert and clay minerals. The thickness of the Unit A Limestone ranges from 67 to 100 feet in the borings, averaging 85 feet (Project Management Corporation, 1982).

The Unit A Upper Siltstone consists of maroon and gray, calcareous siltstone, with laminations and layers of gray, argillaceous limestone. The siltstone is about 80 percent of this unit, with the remaining portion being argillaceous limestone. The argillaceous limestone occurs as thin laminations

within predominantly siltstone sections and as scattered layers up to a few inches thick. The stratigraphic thickness of the Upper Siltstone in the borings ranges from 191 to 216 feet, averaging about 203 feet (Project Management Corporation, 1982).

Unit B

Unit B of the Chickamauga Group is light to dark gray limestone and argillaceous limestone that contains chert nodules. It contains laminations and layers of dark gray, calcareous siltstone, and is locally nodular and fossiliferous. Although no borings penetrated the entire Unit B sequence, its stratigraphic thickness was estimated to be 240 feet (Project Management Corporation, 1982).

Undifferentiated Chickamauga

The Chickamauga Group strata above Unit B have not been differentiated in the same manner as the lower units during the previous investigations. The Undifferentiated Chickamauga consists generally of alternating maroon and gray limestone, shale, and siltstone (Project Management Corporation, 1982).

5.3.1.3 Terrace and Alluvial Deposits

Terrace deposits form a veneer over portions of the site between elevations 750 and 840 feet (Figure 5-9). This material is older alluvium, deposited by the Clinch River when stream levels were higher than they are at present. The terrace deposits are generally interpreted to be Pleistocene or Pliocene in age (Project Management Corporation, 1982). The terrace deposits consist mainly of orange and red, silty clay, with thin layers of rounded quartz, chert, and quartzite gravel.

5.3.1.4 Residual Soil

The majority of the proposed site is underlain by residual soil overlying bedrock of either the Knox or Chickamauga Group. The residual soil typically consists of silty clay and clayey silt, with varying amounts of sand and gravel-size rock fragments. As shown in Figure 5-8, the majority of the existing boring information is concentrated in the area of the proposed CRBRP site. Thus, there is minimal coverage for much of the proposed MRS Facility site, particularly in the areas underlain by the Knox Group. Based on the limited information, it appears that the thickness of the residual soil is variable and somewhat erratic, ranging from less than 5 feet to more than 50 feet. In general, it is believed that the average thickness of residual soil overlying the Knox Group is about 10 to 20 feet, whereas the average thickness overlying the Chickamauga Group is on the order of 15 to 30 feet.

In general, the residual soil appears to be stiff to hard, with standard penetration resistances during sampling ranging from about 0 to more than 40 blows/foot. Locally, several borings encountered zones of soft soils, where up to 25 feet of soft compressible soils were reported. It is not known how extensive these soft zones might be, nor is their origin known. These zones

may be related prehistoric cavity infillings, which were subsequently weathered to residual soil. Based on comments by D'Appolonia (1975), the soft zones may be more prevalent below the water table.

5.3.2 STRUCTURE

The attitude of the bedding planes and joints at the CRBRP site were determined by using data obtained from surface mapping and by defining the orientation of key stratigraphic horizons traceable between borings (3-point solution). Interpretations of structural features are based on these data, as well as observations of outcrops, excavated faces, rock core, and geophysical borehole logs.

5.3.2.1 Bedding Orientation

Measurements of bedding orientation in outcrops and excavated faces provided the general attitude of the rocks over the site peninsula. Bedding strikes between N35°E and N60°E and dips 20 and 52 degrees to the southeast. The average strike is N52°E and the dip is 37 degrees to the southeast. The attitude of the rocks is quite consistent; however, small, tight drag folds and shears were observed at the site, resulting in distortions of bedding attitude. These features extend over only a few feet to a few inches (Project Management Corporation, 1982).

The strike of the bedding, based on marker beds within boreholes, is between N48°E and N60°E and the dip is 24 to 35 degrees to the southeast. The average strike from stratigraphic marker beds is N52°E with a 31-degree southeast dip.

5.3.2.2 Joints

Four joint sets were observed in the rock outcrops and excavations. One set occurs along bedding (weathered partings) having an average strike of N52°E, and dips 37 degrees to the southeast. A second joint set has the same strike as the bedding and dips at an average of 58 degrees to the northwest, or about perpendicular to the trend of bedding. Two joint sets were observed to strike across the trend of bedding at N25°W/80°SW dip and N65°W/75°NW dip. In outcrop, the bedding plane joints (partings) were observed to be spaced about 1 to 5 feet apart. Most joints observed were hairline cracks, with joint surfaces stained by weathering (Project Management Corporation, 1982).

5.3.2.3 Faults, Shears, and Folds

Faults, shears, and folds were found to occur at the Clinch River site during the previous investigations (Table 5-1) (Stone and Webster, 1984; Project Management Corporation, 1982). The shear zones differ from the faults in that they display evidence of greater cumulative displacement and thrust imbrication. The most common type of fault at the site is a bedding plane slip (Stone and Webster, 1984). The bedding plane slips form as a result of up-dip movement on the bedding planes. At a number of locations, the bedding plane slips ramp up-dip, down-dip, or sideways, across bedding, over a short distance to another bedding plane. In a few locations, the

Table 5-1 - Clinch River Site Fault Inventory

Fault Report Number	Bedding Plane Slippage	Ramping (Thrust Faults)		Faults Coinciding With Conjugate Shear Joints
		With Folds	Without Folds	
1	Yes	Yes	Yes (the western shear zone)	No
2	Yes	No	No	Yes
3	Yes	No	No	Yes
4	Yes	Yes	Yes (the eastern shear zone)	No
5	Yes	No	No	Yes
6	Yes	No	No	Yes
7	Yes	No	Yes	No

Source: Stone and Webster, 1984.

bedding plane slips turn into complex, thrust-and-fold structures. Strike-slip faults also occur at the site, and consist essentially of conjugate shear fractures that formed concomitant with the other faults (Stone and Webster, 1984).

Two shear zones were identified at the site. The western shear zone is the only shear zone that is located within the area that is to be developed for the MRS Facility (Figure 5-9). This shear zone occurs within the Unit A Limestone and the underlying Lower Siltstone, and consists of a highly deformed, thrust-and-fold structure, with minor cumulative displacement (Project Management Corporation, 1982).

The orientation of the slickensides on the fault and shear surfaces is parallel to the trend of bedding, and is fairly consistent across the site (Stone and Webster, 1984). In addition, faults of different types are confined to the same general location, displaying small-scale offsets and transitions from one type to another. These observations suggest that the faults are kinematically compatible and genetically related (Stone and Webster, 1984).

The faults, shears, and folds formed at the same time and in the same stress field, which was dominated by northwest-directed compression. These features are thought to be secondary and tertiary minor deformations within a much larger structure; i.e., a thrust block bounded by the Whiteoak Mountain and Copper Creek faults. The secondary deformations are minor thrust-type structures, which reflect internal differential motion in response to the regional translation and rotation of the sedimentary sequence caught between the two major faults. The secondary structures are predominantly up-dip slips of individual beds or bed sequences along stratification, with occasional ramping or concomitant folding, and they are arranged in a pattern similar to the one produced by a bent playing-card deck. The tertiary deformations are conjugate shear fractures, with occasional strike-slip displacements developed as a result of minor thrusting (Stone and Webster, 1984).

5.3.3 WEATHERING

The depth of weathering at the site and the kind of rock alteration imposed by weathering depend primarily on the lithology. Calcareous siltstone is generally weathered more completely and to a greater depth than limestone is (Project Management Corporation, 1982). In atmospheric conditions, the siltstone becomes thoroughly decomposed to saprolite silt, but often displays the original rock structure. Pure limestone, however, is mostly affected by solutioning that has produced elongated, intermittent cavities, which follow bedding planes and strike joints. Development of solution cavities in the argillaceous limestone is limited to the most intensely weathered zone, immediately below the ground surface. Chert sometimes occurs in the saprolite, and appears to be unweathered but fragmented (Project Management Corporation, 1982).

Weathering within the rock occurs as solution widening of joints and bedding features, and has produced soil seams and cavities that range from less than

1 inch to 16-1/2 feet in the borings. Frequency and size of soil seams and cavities produced by weathering and solutioning diminish with depth. With greater depth, the discoloring of the rock occurs only in the area immediately surrounding bedding planes or strike joints, suggesting that weathering has not extended much beyond the discontinuity itself (Project Management Corporation, 1982).

In areas where the limestone and dolomite crop out at the surface, rock weathering along joints is the greatest (0 to 73 feet below the top of rock). In areas where the Upper Siltstone crops out at the surface, rock weathering is less (0 to 57 feet below the top of rock). In the Unit A Lower Siltstone, the depth of weathering extends about 85 feet below the top of rock within a shear zone. Elsewhere, the depth of weathering is less (0 to 60 feet). The depth of maximum weathering and solutioning corresponds to elevations of approximately 750 to 830 feet (Project Management Corporation, 1982).

On the basis of data obtained during exploratory drilling activities and seismic profiling in the Knox Group on Chestnut Ridge, immediately northeast of the site, the following bedrock conditions were defined (Ketelle and Huff, 1984): Residual soils of ariable thickness (20 to 150 feet) typically overlie a zone of cavitose carbonate bedrock, with mud- and gravel-filled cavities. The thickness of the cavitose or weathered zone ranges from 0 to more than 100 feet. Vertical cavity dimensions range from 1 to 16 feet. Weathering tends to have penetrated the deepest in the Chepultepec Dolomite and in the upper one-third to one-half of the Copper Ridge Dolomite (Ketelle and Huff, 1982). Eight borings at the CRBRP site penetrated through the lower Chickamauga strata into the Knox Group, and did not encounter any weathering or solutioning at the contact (Project Management Corporation, 1982).

5.3.4 SOLUTIONING

Large and small depressions that are the result of solutioning of the underlying carbonate bedrock occur at the site. The depressions range in size from a few feet across to hundreds of feet across. The larger depressions are confined to the Knox Group, the Chickamauga Unit B Limestone, and the Chickamauga Undifferentiated. The depressions are circular to oval in appearance, and are as much as about 15 feet deep (Project Management Corporation, 1982). The depressions formed by weathering along steeply dipping joint planes and bedding planes, and are filled with soil that has ravelled into the depressions (Ketelle and Huff, 1984; Project Management Corporation, 1982).

Some of the sinkholes at the site have been investigated by borings (Project Management Corporation, 1982). The top of rock in these borings is irregular because of solutioning along joints. The maximum depth of weathering and solutioning beneath the sinkholes in the Unit B Limestone is about 57 feet. Unit A Upper Siltstone was encountered in each boring below the Unit B Limestone. Upon penetrating the Unit A Upper Siltstone, no solutioning, weathering, or drilling water losses occurred in the borings. In the Knox Group, it was found that, even in areas where sinkholes had formed to a

significant size, no major increase was noted in the thickness of weathered rock compared with areas where limestone strata had not been exposed to solutioning at the surface. The maximum depth of weathering in these borings is about 107 feet (Project Management Corporation, 1982).

Several types of karst features in the Knox Group, which generally parallel the strike of bedding, have been mapped on Chestnut Ridge, immediately northeast of the site (Ketelle and Huff, 1984), and include solution-pan features and sinkholes. Solution pans are gentle, sunken areas that have little or no actual topographic closure. One ephemeral surface stream northeast of the site flows into a shallow hole that is an open karst throat. Seasonal surface flows from this stream are conducted into the groundwater system through this karst feature. In addition to these three classic types of karst features, a hummocky topography, possibly caused by ravelling of soil into bedrock cavities, has been observed on Chestnut Ridge. An unusual juxtaposition of several ridge crest sinkholes and solution pans with gully erosion features has also been observed.

SECTION 6

VIBRATORY GROUND MOTION

The following discussion of vibratory ground motion for the Clinch River site is based primarily on discussions in the CRBRP PSAR (Project Management Corporation, 1982). Additional data and descriptions were obtained from reports referenced in this section.

6.1 ENGINEERING PROPERTIES OF MATERIALS

Engineering properties of materials for seismic wave propagation and soil-structure interaction analyses are discussed in Section 8.

6.2 EARTHQUAKE HISTORY

The local seismicity of Tennessee is characterized by a small number of events with Modified Mercalli (MM) intensities of generally VI or less. The earliest recorded event for the area was in 1699; however, earthquakes were not well documented until after about 1870. The instrumental recording of earthquakes began in the early 1920s, and provided data for events with magnitudes greater than about 4.0. The data base was greatly improved in 1962, when the World Wide Seismograph Station Network (WWSSN) was established. In the mid-1970s, microearthquake networks were operating in the region, and further improved the data base. As of 1983, more than 100 seismograph stations were operating in the southeastern United States (Figure 6-1). The events listed in Table 6-1 are earthquakes from Templeton and Spencer (1980) with intensities of VII or greater. Included are several larger earthquakes that occurred at distant locations, but were felt in the area. These events, along with more than 600 earthquakes with intensities of VI or less, make up the recorded earthquake history from 1699 to 1979, and are used to evaluate earthquake risk. The larger events outside the site region include the New Madrid, Missouri, earthquakes of 1811-1812 and the Charleston, South Carolina, earthquake of 1896.

The earthquakes listed by Templeton and Spencer (1980) include all earthquakes reported to have occurred within Tennessee and surrounding areas from 1699 to 1979 (Figure 6-2). The accuracy and completeness of the historical record are subject to bias resulting from numerous factors. These include the population distribution at the time of the earthquake, the unscientific manner in which the early history was recorded, the wide variation in building construction for applying damage reports to intensity scales, the large increase in earthquake awareness, and the increased number of seismograph stations operating in an area throughout the recorded history.

The observed effects of earthquakes vary with distance from the source location and as a result of local conditions and construction. Earthquake effects are generally described by using the MM intensity scale (Table 6-2). The MM intensities are based on reports compiled from the felt area of the

Figure 6-1 - Southeastern United States Seismograph Stations
in Operation January-June, 1983

Table 6-1 - Southeastern U.S. Earthquakes With Modified Mercalli Intensity VII and Greater
(From Templeton and Spencer, 1980)

<u>Year</u>	<u>Month/Day</u>	<u>Location</u>	<u>Lat</u>	<u>Long.</u>	<u>Intensity</u>	<u>Ref.^a</u>
1811	12/16	New Madrid, Missouri	35.8	90.3	XII ^b	TVA
1812	1/23	New Madrid, Missouri	36.4	90.0	XII ^b	TVA
1812	2/7	New Madrid, Missouri	36.3	89.9	XII ^b	TVA
1843	1/4	New Madrid, Missouri Region	35.5	90.5	IX-X	RGS
1865	8/17	Mississippi Valley	36.7	89.5	VII	RGS
1874	2/10	McDowell County, North Carolina	35.7	82.1	II-VII ^b	Other
1875	6/18	Western Ohio	40.2	84.0	VII	Other
1875	11/12	Northern Georgia - Western South Carolina	33.8	82.5	VI-VII	Other
1878	11/18-19	Mississippi Valley - New Madrid, Missouri	36.0	89.7	VI-VII ^b	RGS
1882	9/27	Southern Illinois	38.0	89.0	VI-VII	TVA
1883	4/12	Cairo, Illinois	37.0	89.2	VI-VIII	RGS
1883	12/6	St. Louis, Missouri	38.6	90.4	VII ^b	RGS
1886	8/31	Charleston, South Carolina	32.9	80.0	X	TVA, RGS
1889	7/19	Memphis, Tennessee	35.2	90.0	V-VII	RGS
1895	10/31	Charleston, Missouri	37.0	89.4	IX	RGS
1897	5/3	Giles County, Virginia	37.1	80.7	VII	Other
1897	5/31	Giles County, Virginia	37.3	80.7	VIII ^b	Other
1905	1/27	Gadsden, Alabama	34.0	86.0	VIII	Other
1905	1/28	Gadsden, Alabama	34.0	86.0	VIII	Other
1905	8/21	Sikeston, Missouri	36.9	89.6	VI-VII ^b	ON
1909	9/27	Wabash Valley, Indiana	39.0	87.7	VII ^b	RGS
1911	3/11	Pine Bluff and Rison, Arkansas	34.1	92.1	VI-Vii ^b	RGS
1913	1/1	Union County, South Carolina	34.7	81.7	VII-VIII	Other
1913	3/28	Knoxville, Tennessee	36.2	83.7	VII	Other
1916	2/21	Waynesville, North Carolina	35.5	82.5	VI-VII	Other
1916	10/18	Irondale, Alabama	33.5	86.2	VII-VIII	Other
1916	12/18	Hickman, Kentucky Area	36.6	89.3	VI-VII ^b	TVA
1922	11/26	Eldorado, Illinois	37.8	88.4	VI-VII	ON
1923	10/28	Marked Tree, Arkansas	35.5	90.4	VII-VIII	RGS
1925	4/26	Ohio Valley	38.0	87.0	VI-VII	ON
1926	7/8	Mitchell County, North Carolina	35.9	82.1	VI-VII	ON

Table 6-1 (Continued)

<u>Year</u>	<u>Month/Day</u>	<u>Location</u>	<u>Lat</u>	<u>Long.</u>	<u>Intensity</u>	<u>Ref.^a</u>
1927	5/7	Marked Tree, Arkansas	35.5	90.3	VII ^b	TVA
1928	11/3	Asheville, North Carolina	36.0	82.6	VI-VII	TVA
1928	11/3	Big Pine, North Carolina	35.8	82.8	VII	TVA
1931	12/16	Batesville, Mississippi	34.0	90.1	VI-VII	TVA
1934	8/19	Rodney, Missouri	37.0	89.2	VII ^b	TVA
1965	3/6	Jefferson County, Missouri	37.8	91.2	VI-VII	TVA
1968	11/9	Hamilton County, Illinois	38.0	88.5	VII ^b	ON
					(M=5.5; 19 km)	

^aTVA - Tennessee Valley Authority
RGS - Richard G. Stearns, 1978
ON - O.W. Nuttli, 1979
Other - Numerous sources

^bMultiple events or events with one or more aftershocks.

6-5

Table 6-2 - Abridged Version of Modified Mercalli Intensity Scale of 1931
(From Wood and Neumann, 1931)

<u>Intensity</u>	<u>Effects</u>
I	Not felt except by a very few under especially favorable circumstances.
II	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motorcars may rock slightly. Vibration like passing of truck. Duration estimated.
IV	During the day, felt indoors by many, outdoors by few. At night, some awakened. Dishes, windows, doors disturbed; walls make creaking sound. Sensation like heavy truck striking building. Standing motorcars rocked noticeably.
V	Felt by nearly everyone, many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.
VI	Felt by all, many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
VII	Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motorcars.
VIII	Damage slight in specially designed structures; considerable in ordinary substantial buildings, with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts, changes in well water. Persons driving motorcars disturbed.
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.

Table 6-2 (Continued)

<u>Intensity</u>	<u>Effects</u>
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with their foundations; ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.
XI	Few, if any, (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII	Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into air.

NOTE: Earthquake effects are influenced by local conditions, thus intensity designations are more reliable when a large number of high-quality reports are available.

earthquake, which are subdivided based on similar effects or damage. These intensities are then plotted on a map, and isoseismals (lines of equal apparent intensity) are drawn around the epicentral area. An isoseismal map can then be used to provide a visual representation of the probable effects of an earthquake or series of earthquakes. Figures 6-3 and 6-4 are isoseismal maps of the New Madrid earthquake of December 16, 1811, and the Charleston, South Carolina, earthquake of August 31, 1886, respectively.

A number of intensity/magnitude relationships have been developed for the southeastern U.S. (Figure 6-5). These generally correlate magnitude with one of several intensity parameters, including maximum intensity, total felt area, total area within the intensity IV MM isoseismal, and falloff intensity. The most common correlation is magnitude with maximum intensity; however, these data are not always available or, if available, can contain a great deal of uncertainty. The relationships are most appropriate when comparing shallow-focus events within the same area.

The earthquake history of the Clinch River site has been described in detail (Project Management Corporation, 1982). This description of earthquakes includes two listings: the first contains earthquakes with epicentral intensities exceeding MM IV or assigned magnitudes greater than 3.0. The second listing contains all earthquakes reported to have occurred within a 50-mile radius of the CRBRP site, regardless of intensity or magnitude estimates.

The largest earthquakes ever recorded in the southeastern United States are the New Madrid earthquakes of December 16, 1811; January 23, 1812; and February 7, 1812 (Figure 6-3) and the Charleston, South Carolina, earthquake of August 31, 1886 (Figure 6-4). The epicentral intensity at New Madrid, 300 miles west-northwest of the site, is estimated to be XII MM (Stearns and Wilson, 1972). The epicentral intensity at Charleston, South Carolina, 315 miles southeast of the site, is estimated to be X MM (from Thiel, 1983). Topographic changes reportedly resulted from these earthquakes over an area of 30,000 to 50,000 square miles, and the total felt area is estimated to have been over 2 million square miles. For an epicentral intensity of X, the felt area for the Charleston earthquake is extremely large (Project Management Corporation, 1982).

The only other moderately large earthquake within the southeastern United States was the May 31, 1897, Giles County, Virginia, earthquake, which had a reported epicentral intensity of VII-VIII MM (Bollinger, 1981). The epicenter of the Giles County earthquake is about 220 miles northeast of the site, and the estimated surface intensity at the site from this event was V MM (Project Management Corporation, 1982).

In general, the earthquakes that occur in the Southern Valley and Ridge Tectonic Province have a focal depth of 50,000 to 60,000 feet (Project Management Corporation, 1982). This is well within the basement and is below the regional decollement, which occurs at about 9,000 feet. Because of the apparent separation of surface features from the basement by this sole fault, it is unlikely that structures seen at the surface would be correlated to any significant earthquakes (Figure 5-6).

Figure 6-3 - Isoseismal Map, New Madrid Earthquakes, 1811-1812

Figure 6-4 - Isoseismal Map for the Charleston, South Carolina, Earthquake, 1896

- Gutenberg and Richter (1958)
- - - Aggarwal and Sykes (1978)
- Chiburis (1981), mean magnitude for each intensity, based on 134 earthquakes
- Chiburis (1981), all data, based on 134 earthquakes

SOURCE: Chiburis (1981)

In the vicinity of the Clinch River site, the surface intensity from the New Madrid events is estimated to be VI-VII MM (Project Management Corporation, 1982) (Figure 6-3) and the observed surface intensity from the Charleston, South Carolina, earthquake is estimated to be VI MM (Figure 6-4). Thus, the largest historic ground motion at the site is estimated to have had an intensity of VI-VII MM from the New Madrid earthquakes, which occurred about 300 miles from the site.

6.3 TECTONIC PROVINCE APPROACH AND EARTHQUAKE PROBABILITIES

Previous workers have not been able to correlate earthquake epicenters with observed tectonic structures. In fact, the focal depth for instrumentally recorded events suggests that the earthquakes are occurring well within the basement rock and below the depth at which surface tectonic structures are thought to exist (Project Management Corporation, 1982). Therefore, to determine seismic risk, previous workers have associated earthquakes with various regions or tectonic provinces, and assumed that a certain maximum earthquake could occur anywhere within a given province. These provinces were generally based on seismicity and observed geologic and tectonic patterns.

The Clinch River site is located in the Valley and Ridge physiographic province, which corresponds to the Southern Valley and Ridge Tectonic Province (Project Management Corporation, 1982). The province was designated by the U.S. Nuclear Regulatory Commission in its evaluation of the Sequoyah Nuclear Plant. As defined, the Southern Valley and Ridge Tectonic Province is bounded on the east by the Piedmont Province, on the west by the Cumberland Plateau, on the south by the overlap of the Gulf Coastal Plain Province, and on the north by reentrant in the Valley and Ridge Province near Roanoke, Virginia (Project Management Corporation, 1982).

Recent work by Seay and Hopkins (1981) suggests that, in addition to surface features, subsurface lineaments and geophysical patterns should be considered in delineating seismotectonic provinces. Braile, et al., (1982) suggest that earthquakes in the midcontinent region may be associated with preexisting structures of certain orientation that act as zones of weakness. Wentworth and Mergner-Keefer (1981), on the other hand, suggest that a more regional pattern is possible and that large, damaging earthquakes are possible in most parts of the eastern seaboard.

Figure 6-6 illustrates the peak horizontal acceleration (given as a percent of the acceleration of gravity) expected in the southeastern United States with a 90% probability of not being exceeded in 250 years. The figure is from Algermissen (1982), who used peak intensity values to estimate acceleration and calculated the probability of occurrence by using recurrence curves.

6.4 PROCEDURES TO DETERMINE DESIGN EARTHQUAKE

The design earthquake for sites such as Clinch River, which have been evaluated under the criteria of Appendix A, 10 CFR 100, is defined as equivalent to the Safe Shutdown Earthquake (SSE) for a nuclear power plant. Because

SOURCE: Algermissen et al, (1982)

NOTE: Contours shown are a percent of the acceleration of gravity.

Figure 6-6 - Southeastern United States Accelerations With 90% Probability of Not Being Exceeded in 250 Years

earthquakes have not been reasonably correlated with tectonic structures, the larger events are identified with the tectonic province in which they occurred. At the CRBRP site, the most severe earthquake associated with the Southern Valley and Ridge tectonic province, in which the site is located, was the Giles County, Virginia, earthquake that was reported to have an epicentral intensity of MM VII-VIII. An intensity of MM VIII (Figure 6-7) was used as the SSE in design of the CRBRP at the direction of the Nuclear Regulatory Commission (Project Management Corporation, 1982).

A peak horizontal ground acceleration of 0.25g was selected for design of the CRBRP, based on an intensity/acceleration relationship developed by Neumann (1954) and an evaluation of other such relationships (Gutenberg and Richter, 1956; Coulter, Waldron, and Devine, from Project Management Corporation, 1982) (Figure 6-7). The design earthquake response spectra for horizontal and vertical motions at the CRBRP site are described by Project Management Corporation (1982). These spectra were linearly scaled from USNRC Regulatory Guide 1.60, Design Response Spectra for Seismic Design of Nuclear Power Plants, in proportion to the maximum specific horizontal ground acceleration of 0.25g (Project Management Corporation, 1982).

SOURCE: Project Management Corporation (1982)

Figure 6-7 - Acceleration/Intensity Relationship

SECTION 7

SURFACE FAULTING

7.1 IDENTIFICATION OF CAPABLE FAULTS

No capable faults have been identified within 5 miles of the Clinch River site (Project Management Corporation, 1982).

7.2 DESCRIPTION OF CAPABLE FAULTS

No capable faults have been identified within 5 miles of the Clinch River site (Project Management Corporation, 1982).

7.3 EVIDENCE OF FAULT OFFSET

Crustal shortening along major thrust faults occurred during the Alleghenian Orogeny (230 to 280 million years ago), and is discussed in Section 5.3.3.

SECTION 8

STABILITY OF SUBSURFACE MATERIALS

8.1 GEOLOGIC FEATURES

The locations of the proposed structures are shown in Figure 5-8 and a generalized profile is shown in Figure 5-10. The primary subsurface units include the Knox Dolomite, the Chickamauga Lower Siltstone, residual soils, and fill. The comments in this and following sections on foundations relate to general preliminary design concepts. These are presented to provide a better understanding of the site, and are not intended to represent a complete evaluation of the foundation conditions.

Conceptually, the anticipated foundation conditions include:

- **Storage Areas:** As shown in Figure 5-8, there is limited subsurface information on the proposed Storage Area. Based on the available information, the area appears to be underlain by an average of about 10 to 20 feet of residual soil over the Knox Dolomite. The top of the bedrock surface is irregular, with buried bedrock pinnacles up to 5 to 10 feet of relief. The proposed cask pad elevations will be at about elevation 875 to 880 feet. At this elevation, the majority of the area will be in cuts ranging from 0 to 70 feet, with a limited zone of fill required in the southern and eastern areas. Thus, the majority of the cask pads, which are Category I structures, will probably be supported directly on unweathered Knox Dolomite. The pads located in zones of either limited cut or fill may be underlain by weathered rock, residual soil, and/or fill.
- **Protected Area:** The major structure in the Protected Area is the Receiving and Handling (R&H) Building, which will consist primarily of a Category I structure with small sections around the perimeter that are Non-Category I. The structure is to be located in the former parking area for the CRBRP, at elevation 830 to 840 feet, adjacent to the reactor excavation. The reactor excavation originally extended down to an elevation of about 720 feet. The planned redress grades will bring this area up to elevation 780 feet. The planned foundation-bearing level for the R&H Building is at about elevation 815 feet, or 15 to 25 feet below the redress grades of the parking area. Based on the available data (discussed in Section 5.3), approximately the western 40% of the R&H structure will bear on unweathered Knox Dolomite; the eastern 60% will bear on the unweathered Chickamauga Unit A Lower Siltstone. Locally, there may be 5 to 10 feet of weathered rock occurring below the proposed foundation level. In the northwest corner of the structure, the

top of unweathered bedrock may be as much as 15 to 30 feet below the foundation level. Several of the smaller structures in the Protected Area are located in areas underlain by fill.

- Limited Access Area: The proposed grades in the Limited Access Area are at about elevation 811 to 820 feet. To achieve these grades will require placing up to about 40 feet of fill. In the area of the original reactor excavations, which underlie the western side of the Limited Access Area, the fill will be up to about 40 feet thick.

8.2 PROPERTIES OF SUBSURFACE MATERIALS

A major subsurface investigation program was completed in conjunction with the design and construction phases of the originally planned CRBR project, and is summarized in several references. It is not the intent to reproduce this information in this document, but rather simply to summarize the results, with appropriate references. The appropriate data reviewed included:

- The CRBRP PSAR (Project Management Corporation, 1982): The investigation included 126 borings, with field and laboratory test data primarily on the bedrock formations. Laboratory data on rock core samples included unconfined compression tests with Poisson's ratio determinations, unit weights, and water contents. Field data included Goodman jack tests, continuous borehole velocity logging, seismic uphole surveys, and seismic cross-hole surveys. The majority of the tests were performed on the Chickamauga Unit A Upper Siltstones and Unit A Limestone. The data on the Chickamauga Lower Siltstone and Knox Dolomite were limited.
- Report of Engineering Properties for the On-Site Residual Borrow Soils, Geotechnical Report Number 1, Clinch River Breeder Reactor Plant (Law Engineering, 1974): The primary purpose of this work was to evaluate the engineering properties of compacted residual soil from onsite borrow areas, and included 22 test pits. Laboratory tests were performed on compacted bulk samples, and consisted of classification tests, Proctor compaction tests, consolidation tests, triaxial tests, and resonant column dynamic tests.
- Subsurface Investigation for Proposed Cooling Water Pipeline System and Cooling Tower (D'Appolonia, 1975): This investigation included 23 borings and laboratory soils testing. The tests consisted of classification tests, consolidation tests, unconfined compression tests, Proctor compaction tests, and triaxial compression tests.
- Report on Engineering Properties for On-Site Terrace Deposit Soils (Law, 1975): This investigation included 22 test pits, seven shallow borings, and laboratory tests on compacted terrace deposits. The tests included classification tests, Proctor compaction tests, triaxial compression tests, and consolidation tests.

- Report of Rock Slope Stability Analysis, Nuclear Plant Island Excavation Slopes (Law, 1975): This investigation involved the evaluation of the cut slopes planned for the original reactor excavation. Testing included 18 direct shear tests of selected rock samples.
- Report on Subsurface Investigation, Proposed Balance of Plant Structures and Borrow Area (D'Appolonia, 1975): This investigation involved an evaluation of the foundation support for several Non-Category I structures and borrow sources. It included 12 borings and soils laboratory testing. The testing included classification tests, sulfate content determinations, consolidation tests, Proctor compaction tests, swell tests, sensitivity determinations, unconfined compression tests, and triaxial compression tests.
- Draft Report of Test Grouting Program, Geotechnical Report Number 6 (Law, 1975): This report presents the results of a test grouting program in the Chickamauga Formation.

8.2.1 STATICALLY DETERMINED PROPERTIES

The foundations for the proposed facility will generally bear on Knox Dolomite (majority of storage area, portion of R&H Building, Chickamauga Group Unit A Lower Siltstone (portion of R&H Building), portions of Protected Area, and portions of Limited Access Area), residual soil (relatively limited areas), and fill over residual soil/bedrock (majority of Limited Access Area and portions of other areas).

The general static properties of the unweathered bedrock formations suitable for preliminary conceptual design (Project Management Corporation, 1982) are:

Unit weight	165 pcf
Poisson's ratio	0.3
Shear modulus	200 ksi
Young's modulus	500 ksi

These values were developed for the Chickamauga Group. Based on relative seismic velocities, it is believed that the modulus values for the Knox Dolomite would generally be higher.

There is relatively little information available on the engineering properties of the in situ residual soils, with the majority of the data relating to the area of the proposed CRBRP. The data indicate that the residual soil is generally a stiff to hard, silty-clay to clayey-silt, with varying amounts of sand. Locally, zones of relatively soft, compressible soils were encountered. The plastic index (liquid limit minus plastic limit) data ranged from about 5% to 70%, with an average of 30% (Law, 1974). The natural water content was near the plastic limit, which is consistent with the stiff-to-hard consistency of the soil. The specific gravity of the soil was reported to be about 2.55. The limited data indicate that the in situ dry unit weight ranged from 75 to 115 pcf, with undrained shear strength test

results ranging from 500 to 2,200 psf. A relatively limited number of consolidation tests were performed, and indicated that the soil was over-consolidated, with a preconsolidation pressure of about 1,500 to 3,000 psf, and exhibits a low to moderate compressibility (compression ratios on the order of 0.05 to 0.15).

Law (1974) performed a laboratory testing program on samples of compacted residual soil taken in the general area of the proposed CRBR facility. These test results indicated:

- **Effect of Parent Formation:** In general, the testing indicated only minor differences in the engineering properties for compacted samples from different parent formations.
- **Compaction Properties:** The maximum dry densities ranged from 82 to 118 pcf at optimum water contents ranging from 11% to 33%. The natural moisture contents are significantly higher than the optimum moisture contents, indicating that compaction may be difficult unless the materials can be dried.
- **Strength:** Based on the strength test results, Law (1974) recommended the following design values for the undrained shear strength of compacted fill:

	<u>Cherty Soil</u>	<u>Non-Cherty Soil</u>
Cohesion (ksf)	2.0	2.0
Friction (degrees)	17	12

- **Consolidation:** The measured compression ratios ranged from 0.04 to 0.26, with the high values produced by specimens prepared at high water contents. The compaction energy appeared to result in a soil that was overconsolidated by at least 2 ksf.
- **Shrink and Swell Properties:** Based on the results of the laboratory tests, Law (1974) concluded that the compacted residual soils would have a low to moderate susceptibility to volume change.

No data are available on the engineering properties of compacted, crushed rock fill for either the onsite Knox Dolomite or rocks of the Chickamauga Group. It is believed that the majority of the excavated rock will be Knox Dolomite. These types of rocks can normally be broken down, through a combination of blasting and crushing, to a gradation suitable for placement as compacted structural fill. Placed in lifts and properly compacted, this material should produce a rockfill of high strength and low compressibility.

8.2.2 SEISMICALLY DETERMINED ENGINEERING PROPERTIES

Based on a review of the limited available information, the following values are judged to be appropriate for conceptual seismic design purposes (based in part on results presented by Project Management Corporation, 1982):

	<u>Knox</u>	<u>Chickamauga</u>
Shear wave velocity (fps)	10,500	7,100
Young's modulus (ksi)	3,900	1,810
Shear modulus (ksi)	1,500	700
Poisson's ratio	0.3	0.3

Dynamic tests on samples of compacted onsite residual soil were performed by Law (1974). The report presents plots of dynamic shear modulus and damping ratio as a function of shear strain. At a shear strain of $3E-4$ in./in., the computed damping ratio ranged from 5% to 8%.

No data are available on either the in situ residual soil or the compacted rock fill.

8.3 PLOT PLAN

Figure 5-8 shows the locations of the primary borings drilled as part of the CRBR investigation. A general idealized cross section through the proposed site is shown in Figure 5-10. The actual boring logs are not included in this report.

8.4 SOIL AND ROCK CHARACTERISTICS

The general characteristics of the soil and rock underlying the site are discussed in Section 8.2. Although considerable data are available on selected subunits in the Chickamauga Group and compacted samples of onsite residual soil, there are limited subsurface and laboratory data for several of the main materials that will underlie the MRS Facility. These include the in situ residual soil, weathered rock, rock fill derived from excavation of the onsite bedrock, the Lower Siltstone unit of the Chickamauga Group, and the Knox Dolomite.

The material properties discussed in Sections 8.2.1 and 8.2.2 apply to continuous soil and rock strata. As discussed in Section 5.3.4., solution features may occur in the carbonate bedrock underlying the site. These features may contain open voids or be filled with soft infillings of soil.

8.5 EXCAVATION AND BACKFILL

The MRS project will involve substantial excavations and filling in order to establish the planned site grades. The maximum cuts are on the order of 70 feet, with the maximum fills on the order of 40 feet. In general, the structures will be above final grade, with no major below-grade construction except for the utilities.

The majority of the proposed excavation will be in the Storage Area, with minor excavations in the Protected Area. Based on limited information, it is estimated that excavation profiles will generally consist of approximately 15 feet of residual soil underlain by bedrock. In the areas underlain by the Knox Dolomite, the bedrock surface may contain rock pinnacles on the order of

5 to 10 feet high. Based on limited information, the Knox Dolomite does not appear to have a significant thickness of weathered rock, but does contain a cavitose zone of variable thickness (0 to 100 feet) beneath the soil cover. In areas of the Chickamauga Group, a weathered rock zone of variable thickness may exist. The weathered rock and residual soil can be excavated with conventional excavation equipment. Locally, the weathered rock will probably require ripping and possibly blasting in harder zones. The residual soil may be difficult to excavate during wet periods when the surface of the material will soften and provide poor support for earthmoving equipment. Boulder-size fragments of unweathered bedrock can be encountered in either unit. It is anticipated that the unweathered rock will require blasting to excavate.

Assuming that the majority of the fill will be derived from onsite excavated material, the fills may be composed of residual soil, weathered bedrock, and unweathered bedrock. Because of its high percentage of fines, plasticity, and high natural moisture content, excavated residual soil may be unsuitable for placement as compacted fill under structures. Compacted, weathered bedrock is expected to have similar properties to compacted residual soil (D'Appolonia, 1975). Thus, it may be necessary to limit the use of residual soil and weathered rock fills to general grading areas beyond the influence of structural loads. Provided that the fill derived from unweathered rock is well graded and properly compacted, it should provide an excellent structural fill material. A test excavation and fill program may be warranted to develop the optimum construction procedures for rock excavation, crushing, placement, and compaction (D'Appolonia, 1975).

8.6 GROUNDWATER CONDITIONS

The groundwater conditions at the site are discussed in Section 4.

8.7 RESPONSE OF SOIL AND ROCK TO DYNAMIC LOADS

Refer to Section 8.2 for a discussion of the dynamic properties of the bedrock and residual soil fill materials. There are no site-specific data on the dynamic properties of rock fill or residual soil.

8.8 LIQUEFACTION POTENTIAL

The proposed facility will be underlain by fine-grained fill (derived from weathered rock and residual soil), coarse-grained rock fill (derived from unweathered rock), fine-grained residual soil, weathered rock, and/or unweathered rock. None of these materials is considered to be susceptible to liquefaction. In addition, it is anticipated that the areal fills will be placed above the permanent water table, and will not be saturated.

Locally, zones of granular soil may be encountered in existing or prehistoric stream channels or adjacent to the Clinch River. These materials may be susceptible to liquefaction. Additional investigation would be required to identify the occurrence and extent of potential liquefiable areas.

8.9 EARTHQUAKE DESIGN BASIS

The earthquake design basis is discussed in Section 6, Vibratory Ground Motion.

8.10 STATIC ANALYSIS

The proposed structures include four general types:

- (1) **Receiving and Handling Building:** This is a large, heavy structure with safety equipment and materials. It is a Category I structure and will require conservative foundation support.
- (2) **Storage Casks:** The storage casks will be supported on large concrete pads. The required foundation performance may be relatively tolerant because the casks can probably withstand considerable movement without affecting their performance. This issue needs to be evaluated particularly in developing foundation criteria in fill areas. The actual foundation-bearing pressure imposed by the pads is expected to be relatively light, on the order of 1,500 psf.
- (3) **Non-Category I Buildings:** The Limited Access Area will include numerous Non-Category I, light to moderately loaded buildings. The majority of the buildings are relatively light, one- to two-story office and warehouse-type structures.
- (4) **Support Structures, Roads, and Utilities:** The facility will include numerous support structures, roads, and utilities, such as tanks, paved and gravel roads, and rails. In general, these will be light to moderate loads, and will not present any unusual foundation criteria.

8.10.1 RECEIVING AND HANDLING BUILDING

Based on the available information, the western 40% of the R&H Building will be underlain by Knox Dolomite; the eastern 60% will be underlain by calcareous siltstone (Lower Siltstone) of the Chickamauga Group. At the preliminary foundation grade of about 815 feet, the majority of the structure will bear directly on fresh bedrock. Locally, because of bedrock surface variability, 5 to 10 feet of additional excavation may be required to reach unweathered rock. In the northwest corner, up to 15 to 30 feet of additional excavation may be required to reach unweathered rock. In areas requiring additional excavation, the foundation grades could be lowered to bear directly on rock or the area could be backfilled with concrete.

Both the Knox and Chickamauga Lower Siltstone have relatively high strength and modulus properties, and will adequately support the R&H Building with minimal settlement. The foundation could be designed as either a continuous mat or placed on spread footings. Although the Knox Dolomite has somewhat greater strength and modulus than the siltstone, it is believed that this will not result in any significant difference in the foundation support capacity or settlement response.

There is a potential for solution features to occur in the bedrock underlying the proposed structures, as discussed in previous sections. Further investigation will be required to determine the occurrence and extent of these features. If they are located below the structures and found to be extensive enough to have a potential impact on foundation support, it may be necessary to perform a bedrock grouting program. Grouting of solution cavities below Category I structures has been an acceptable procedure at several nuclear power plant sites.

8.10.2 STORAGE AREA

The majority of the cask storage pads will be located in an area of substantial cut, and will probably be underlain by unweathered Knox Dolomite. In these areas, the bedrock will provide adequate support for the relatively light pad loads.

A small portion of some of the Storage Area cask pads will be underlain by fill and/or residual soil over bedrock. The maximum depth of fill underlying a cask pad will be about 30 feet. Placement of a substantial weight of new fill will induce settlement because of compression of the residual soils. Based on the limited available information, the magnitude of maximum settlement in areas requiring 20 to 30 feet of new fill is anticipated to be on the order of 1 to 6 inches, depending on the thickness and properties of the underlying residual soils. The length of time required to reach 90% of consolidation will be difficult to estimate because the residual soils are believed to be partially saturated (D'Appolonia, 1975). However, for conceptual design purposes, it could be as long as several months to more than a year. Thus, placement of cask pads on fill and/or residual soil will provide significantly different foundation-support characteristics than the bedrock. Except for local areas of soft, compressible residual soil, the residual soil and/or fill should be capable of supporting the relatively light pad foundation loads, provided that the settlements are acceptable. The suitability of the residual soils and acceptability of the potential settlements depend on the foundation performance criteria for the cask foundations. Several potential foundation design alternatives could be taken in the areas underlain by fill and/or residual soil, including:

- Relocate the cask pads to avoid areas underlain by residual soil and/or fill. If the areal extent of cask pads underlain by residual soil is limited, this may be a feasible alternative.
- Excavate the residual soils down to bedrock, and backfill with compacted structural fill.
- Pile support those sections of the pads underlain by residual soils and fills.
- Design for differential settlements while minimizing their magnitude. This might include providing structural joints in the pad to accommodate the differential settlement between the rock-supported and soil-supported sections. In addition, the magnitude of the post-

construction settlements could be significantly reduced by placing fill early to induce the majority of the settlement before constructing the pads and placing the casks. These settlements could be further reduced by surcharging the area (placing fill in excess of the final planned grades).

8.10.3 LIMITED ACCESS AREA

The structures in the Limited Access Area will generally be underlain by up to 40 feet of new fill over residual soil and bedrock. In addition, several structures will lie over the original CRBRP reactor excavations, and will be underlain by as much as 90 feet of fill. In general, the compacted fill and the residual soil will have adequate strength to support the relatively light loads imposed by the structures in the Limited Access Area.

The weight of new fill will cause compression of the underlying residual soils. In addition, even though carefully compacted, the fill itself will settle. The magnitude of settlement depends on the thickness of fill, the properties of the fill, and the depth and compressibility of the residual soils. Based on the current limited information, the magnitude of settlement is estimated to be on the order of 0.5 to 2 inches in areas where the fill thickness is less than about 20 to 20 feet. In areas where 20 to 40 feet of fill are required, the settlements could be on the order of several inches up to as much as 1 foot. Based on the available information, there appears to be considerable variability in the thickness of the residual soil layer. In addition, some local zones of soft, more compressible soils were encountered (D'Appolonia, 1975). These characteristics will act to increase the tendency to develop differential settlement.

Several alternative foundation designs may be feasible for the Limited Access Area, including:

- **Preload With Limited Excavation:** The entire area could be filled to grade with surcharge (fill in excess of final grade) placed in areas of the structures. The appropriate surcharge weight depends on many factors, but should at least be equal to the weight of the loaded building. After an appropriate preload period, the surcharge could be excavated and the structures placed on normal spread footings with slab-on-grade construction. The preload period will depend on how rapidly the soil compresses, but would probably be on the order of a few months, where the residual soil is thin, to as much as a year or more where it is thickest. Locally, where pockets of soft soil underlie structures, it may be prudent to excavate these zones before filling.

If scheduling is a problem because of the preload periods, it may be possible to accelerate the process through the use of sand drains or wick drains.

It is difficult to estimate the required preload period because of the occurrence of sandy zones, possible partial saturation condition of portions of the unit, and soil variability. A test preload section, with and without drains, would probably be cost effective in developing an optimum design, better estimating the required surcharge period, better estimating the long-term postconstruction settlements, and assessing the need for drains.

- **Excavation of Residual Soil:** In areas where the thickness is limited, it would be feasible to excavate the residual soil before filling. Although this may be practical in some areas, it is not believed to be cost effective for the majority of the site because of the depth of excavation and thickness of required backfill.
- **Normal Foundations With No Preload:** In areas of limited fill depth, particularly if the residual soils are found to be thin or relatively noncompressible, preloading may not be required.
- **Deep Foundations:** It would be feasible to place some or all of the structures on a deep foundation, such as piles bearing on bedrock. Locally, this might be desirable, particularly if the structure is settlement-sensitive or is underlain by a substantial thickness of soft soils. However, in general, pile-supported structures do not appear to be a cost-effective solution. Because of the potential for settlement and the thickness of structural fill penetrated by the piles, the downdrag forces may be significant.

Additional information and testing would be required to develop the optimum foundation alternatives in each area.

8.11 TECHNIQUES TO IMPROVE SUBSURFACE CONDITIONS

Depending upon the specific foundation conditions that are found beneath each area of the proposed facility, several techniques to improve subsurface conditions may be appropriate. Such techniques might include manual excavation, concrete backfill, slush grouting and consolidation grouting in solution-prone bedrock areas, rock bolting or dowelling in areas of unstable rock slopes, preloading and surcharging in areas underlain by compressible soils, deep soil densification in areas underlain by potentially compressible granular soils (such as the deep fills in the original reactor excavations), and others.

8.12 CRITERIA AND DESIGN METHODS

Because of the preliminary nature of this report, specific foundation design criteria have not been developed.

SECTION 9

SLOPE STABILITY

Slopes that could potentially affect the MRS development include both natural and manmade slopes.

9.1 SLOPE CHARACTERISTICS

9.1.1 NATURAL SLOPES

The principal natural slope that could potentially affect the facility lies on the western side of the northern end of the cask storage area. The slope rises to the northeast approximately 130 feet above the storage area final grade at a gentle average slope of about 10 degrees. A smaller local area of natural slope lies at the southeast corner of the storage area, rising a maximum of 50 feet above site grade and falling away eastward to the Clinch River. The primary effect of this area of natural slope is that it gives rise to a localized, excavated permanent slope at the southeast corner of the storage area.

Both of these areas of natural slopes are underlain by dolomitic rock of the Knox Group. Specific exploration data in these areas are limited, and detailed information concerning slope characteristics is not available. In general, the available information indicates that these slopes will be mantled by residual soil of variable thickness, overlying Knox Group rocks displaying evidence of karstification, such as sinkholes, solution widening of joints, and irregular topography (pinnacles) (Project Management Corporation, 1982).

9.1.2 MANMADE EXCAVATED SLOPES

Based on current conceptual layouts, excavated slopes will be permanently exposed in four areas of the site. The characteristics of the proposed slopes in each area are summarized below.

At the northern end of the cask storage area, a roughly 1,000-foot-long cut slope will rise to a maximum height of about 50 feet, oriented in an east-west direction and dipping at 27 degrees (2H:1V) to the south. At both the southeast and the southwest corners of the cask storage area, cut slopes of about 500-foot overall lengths will rise to maximum heights of 50 feet, both oriented in a north-south direction and both dipping to the west at 27 degrees (2H:1V). Underlying the above three slopes will be rocks of the Knox Group. Variable thicknesses of residual soil will overlie rocks showing evidence of karstification. Data regarding the orientation and nature of discontinuities in the rock have been collected and presented in various reports related to the CRBRP (notably Project Management Corporation, 1982,

and Stone and Webster, 1984). Although the bulk of these data deals with the Chickamauga Group, current evidence indicates that the general discontinuity sets, presented in these reports and summarized in Section 5.3, are valid within the Knox Group rocks. Further investigation of this assertion will be necessary.

The three general discontinuity sets identified are:

Bedding plane partings: N52°E/35°SE
Strike joints: N52°E/58°NW
High-angle cross joints: N65°W/75°NE, N25°W/80°SW

A fourth small area of excavated slope is currently planned at the eastern boundary of the site, about 500 feet east of the Standby Generator Building. Two short (300-foot-long) sections of 40-foot-high (maximum) slope, oriented north-south and dipping at 27 degrees (2H:1V) to the west, will exist in this area.

The orientations of the proposed excavated slopes and of the general bedrock discontinuities are summarized in the stereographic projection shown in Figure 9-1.

9.1.3 MANMADE FILL SLOPES

Several areas of manmade fill slopes will be constructed to heights ranging up to about 40 feet.

9.2 DESIGN CRITERIA AND ANALYSES

Existing reports concerning the stability of excavated slopes at CRBRP have been reviewed, including Project Management Corporation (1982); Law (1975); Stone and Webster (1984); and D'Appolonia (1975).

9.2.1 NATURAL SLOPES

There is no currently available information that explicitly addresses the stability of natural slopes at the site. Although investigation will be required to explicitly characterize the overburden and rock materials in the specific areas noted, it is not anticipated that stability of natural slopes will be an issue of concern. The gentle overall slope angles (10 degrees) will not be steep enough to permit any significant failures to occur within either the residual soil overburden or the bedrock.

9.2.2 MANMADE EXCAVATED SLOPES

Detailed review of the existing data base will be required, in addition to further investigation, to explicitly characterize the ground conditions at each designed slope location. In particular, requirements will include determination of the depth and engineering characteristics of the residual soil covers, depth and nature of weathering and solutioning effects

Figure 9-1 - Clinch River Orientations of Excavated Slopes and Bedrock Discontinuities

(particularly within slopes underlain by the Knox Group), and confirmation of the attitude and nature of the discontinuities within the rock mass of each slope area.

Based on the available data, the significant design or licensing issues regarding slope stability have been identified. Appropriate slopes, in terms of both stability and erosion, would probably be on the order of 2H:1V. Within the rock portion of the slope excavations, stability will be governed by the attitude and strength characteristics of the rock mass discontinuities. In order for a potential planar or wedge failure to be kinematically (i.e., geometrically) possible, it is necessary for the dip of the cut face to be greater than the dip of a potential planar sliding feature or, in the case of a wedge, greater than the dip (plunge) of the line of intersection of two planes. As indicated in Figure 9-1, these conditions are apparently not met for the conceptual MRS layout at Clinch River. Thus, stability of the rock cuts is not likely to be a problem.

Final design slopes must be specifically analyzed once the noted additional site-specific data are available.

9.2.3 MANMADE FILL SLOPES

Fill slopes up to about 40 feet will be required. These slopes will probably be composed of compacted residual soil and/or weathered bedrock material in noncritical areas, and composed of compacted, unweathered rock fill in areas adjacent to structures. These materials are not believed to have any detrimental properties that would create any unusual slope problems. Depending on the placement and compaction criteria and material properties, appropriate fill slopes will probably be on the order of 2H:1V to 3H:1V unless flatter slopes are required for erosion protection or aesthetics.

SECTION 10

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