2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING SUMMARY OF FOUNDATION CONDITIONS

The Watts Bar Nuclear Plant site is located on the right bank of Chickamauga Lake, approximately two miles downstream from Watts Bar Dam, at river mile 528.0 (Decatur quadrangle, 118-SW).

The site was first explored in 1950 when twenty holes, totaling 580 feet, were drilled. These shallow holes were predominantly wash borings to determine the approximate top of rock and were drilled toward Chickamauga Lake from the location of the present plant. No further work was done at this site until June 1970, when 49 top-of-rock determinations were made by refraction seismic methods. As shown on Figure 2.5-12, the seismic stations were laid out on a 400-foot grid with the lettered ranges oriented N22°W and the numbered sections oriented N68°E. Preliminary core drilling was started in July 1970, with 11 holes on 400-foot centers using the same grid that had been established for the seismic studies (Figure 2.5-12).

By the end of July 1970, engineering studies had progressed to the point where the portion of the site most suitable for the location of the major structures had been approximately determined. At this time 16 additional holes in this area were drilled on 200-foot centers. In mid-September, 29 more holes were added on 100-foot centers to fill in the area finally decided upon as the plant location. In all, 56 holes were drilled during exploration program.

The majority of the holes were drilled to elevation 635, which is 55 feet below foundation grade and from 60 to 65 feet below the general elevation of the top of rock. As shown on Figure 2.5-13, the area of concentrated exploration lies between Ranges J and R and between Sections 59+00 and 66+00. All holes within these limits were examined with TV borehole equipment to confirm that the areas of non-recovery of core represented grinding of soft material and not cavities in the bedrock. Detailed individual logs of each hole in this area are presented in Figures 2.5-14 through 2.5-69.

Pressure bulb tests to determine the static characteristics of the foundation material were made in six holes by Geocel, Incorporated, and dynamic seismic tests were made by TVA personnel and Birdwell Division of Seismograph Service Corporation (Figure 2.5-70).

Construction excavations began in July 1973, and were completed in mid-April 1974. All excavation floors as well as numerous cutfaces and terrace-rock contacts were mapped and photographed by the project geologist (Figures 2.5-110 through 2.5-122). Prior to concrete emplacement the foundation bedrock was inspected and released by the project geologist. Foundation treatment during the excavations consisted of dental work and minor grouting and is submitted as Figures 2.5-139 through 2.5-144.

Figures represented in this report are based upon the following areal coverage:

(1) Region - 200 mile radius from the site - ex: Regional Geologic Map (Figure 2.5-2)

(2) Subregion - 65 to 100 mile radius from the site ex: Subregional Fault Map (Figure 2.5-8)

(3) Plant area - five mile radius from the site - ex: Geologic Map of the Plant Area (Figures 2.5-9 and 2.5-10)

(4) Plant - Immediately related to location of the plant - ex: Drill Layouts, Photographs, and Individual Geologic Logs

Physiographically, the site is located in the Tennessee section of the Valley and Ridge Province of the Appalachian Highlands (Figure 2.5-1). This section is the southernmost of the three sections comprising the Valley and Ridge Province and extends from the Tennessee River-New River divide southwestward into central Alabama. It is bounded on the west by the Appalachian Plateaus Province and on the east by the Blue Ridge Province.

The site is located along the northeast-southwest trending portion of the Tennessee River drainage basin. At the site area the elevation of the flood plain is approximately 700 feet and to the west of the plant location is a series of knobs reaching elevation 900 feet. The plant lies on an alluvial terrace of approximate elevation 735 feet (Figure 2.5-11).

The site is located in the folded and faulted Southern Appalachian Structural Province and in the Southern Appalachian Tectonic Subdivision. A Modified Mercalli Intensity (MM) VIII earthquake is assumed to occur at the site with accelerations of 0.18 G's horizontal and 0.12 G's vertical for SSE requirements.

Regional and subregional fault maps are provided as Figures 2.5-7 and 2.5-8 and cover radii of 200 and 100 miles, respectively.

The plant site is situated in a bend of the Tennessee River that has been covered by alluvial terrace deposits (Figure 2.5-11). Beneath these deposits lies the Middle Cambrian Conasauga Formation, an interbedded shale and limestone upon which the Category I structures are founded. The regional strike of this formation is approximately N35°- 40°E (Figure 2.5-110) and beds for the most part dip to the southeast. However, because of relatively complex folding at the site, the attitudes of the bedding range from horizontal to vertical. Photographs, sections, and maps of the excavations of the plant showing rock quality and structural complexity are included in this report as Figures 2.5-110 through 2.5-122.

2.5.1 Basic Geology and Seismic Information

The Watts Bar Nuclear Plant site is located in Rhea County, Tennessee, on the right side of the Tennessee River at river mile 528, about two miles south of Watts Bar Dam (Figure 2.5-9). Exploration was started in 1950 when twenty holes, totaling 580 feet, were drilled. Drilling was on a grid with lettered ranges oriented N22° W and numbered stations oriented N68°E (Figures 2.5-12 and 2.5-13). The range lettering begins to the east at range A and extends westward on 100-foot intervals down the alphabet to Z. The letter "I" was omitted from all ranging. The station numbering begins to the south with station 89+00 and extends northward on 100-foot intervals to station 48+00. Exploration locations used throughout the program are found by the intersection of these lettered ranges and numbered stations such as L-68+00.

Drilling was begun with holes on 400-foot spacings, and as engineering studies progressed and subsurface information became available in July, 1970, the spacing was reduced first to 200-foot spacings and finally in mid-September to holes 100 feet on center under the major structures (Figure 2.5-13). The majority of the holes were drilled to elevation 635 feet, which is 55 feet below foundation grade and from 60 to 65 feet below the general elevation of the top of rock.

Layouts and summaries of all drilling are furnished as Figures 2.5-12, 2.5-13, and 2.5-70. Individual graphic logs of all holes are presented as Figures 2.5-14 through 2.5-69.

Dynamic seismic tests and pressure bulb tests were made in and between selected holes in the main plant area to determine the in situ dynamic characteristics of the foundation rock (Figures 2.5-70 through 2.5-109). Details of these investigations are in Sections 2.5.1.2.9 and 2.5.1.2.11.

2.5.1.1 Regional Geology

2.5.1.1.1 Regional Physiography

Except as otherwise indicated, this section is referenced from Fenneman[23].

The Watts Bar Nuclear Plant is located in the Tennessee section of the Valley and Ridge Province of the Appalachian Highlands (Figure 2.5-1).

The Valley and Ridge Province is a long narrow belt trending NE-SW that is bordered by the Appalachian Plateau on the west and by the Blue Ridge Province on the east. It extends for 1,200 miles, from eastern New York to central Alabama. Its maximum width is 80 miles, and its width barely reaches 35 miles in New York. The maximum width in east Tennessee is 40 miles, which is near the average for the south half of the province.

Geologically, this province represents the eastern margin of the Paleozoic interior sea. Structurally it is part of an anticlinorium, the successor to a geosyncline which sank intermittently for ages as it received sediments from the concurrently rising old land surface on the east.

Of the stratified Paleozoic rocks involved in the folding, all outcropping at one place or another, there are great differences in hardness. In the humid climate of eastern United States, limestone (unless cherty) is generally weathered away most rapidly. Shale is usually slightly more resistant, but both are subject to weathering. Well-cemented, siliceous sandstone and conglomerates are the most resistant. As selective erosion is particularly conspicuous in this region, and as folding and thrust faulting may bring the same stratum to the surface repeatedly, a few resistant formations produce several ridges. Sandstones are the most significant ridge makers.

Viewed empirically, the Valley and Ridge Province is a low-land (an assemblage of valley floors separated by long, narrow, even-topped mountain ridges). Either of these elements may predominate. The mountains may be widely spaced and isolated, or so closely ranged that the lowlands are disconnected or absent. The valley floors are entrenched by streams. Morphologically the province is one of folded mountains, in which resistant strata form ridges, and weaker rocks are worn down to lowlands, which in themselves further eroded in a still later cycle. The lateral compression that made the folds also caused thrust faults. Indeed, much of the eastern province boundary is delineated by thrust faults, along which the older and more resistant rocks were pushed westward over the younger and weaker rocks. By these means a much broader belt was reduced to one-half, perhaps to one-third, of its former width. It was from the extremely mountainous topography thus described that the present surface was developed. The major steps in the process were: (1) general peneplaning, (2) upwarping, (3) reduction of the weaker rocks to plains at lower levels, and (4) further uplift and dissection.

The physical characteristics of this province are intimately connected with its streams, which are primarily causes, not effects of the present topography. Adjustment of drainage to structure produces a condition in which streams follow the strike as much as possible, keeping on belts of soft rock. They cross the hard rocks through "water gaps" as rarely as possible and then by the most direct route (i.e., at right angles). The tendency is for drainage to become longitudinal, but this tendency stops when down cutting has reached a stage when every divide is being worn down equally on both sides.

The drainage of this province at an earlier stage was mainly transverse. Longitudinal streams were a subsequent development. In the struggle between longitudinal subsequent streams and transverse consequent streams, the former often captured the latter, and with each capture one hard-rock crossing was eliminated, and the "water gap" became a "wind gap." The final result of a succession of such captures tends to show a "trellised" pattern. The course of any larger stream is a series of rectangular offsets. An intelligent glance at the stream pattern of the Valley and Ridge Province is sufficient to show the great extent to which its drainage has been revamped. Most of the longitudinal streams have developed by slow headward growth, and most of the old transverse streams have vanished. The many deserted wind gaps are impressive.

The Valley and Ridge Province is divided into three parts, designated respectively as the Hudson Valley, Middle, and Tennessee (or southern) sections. The Hudson Valley section is not involved in the physiography of the site region and will not be discussed further. The boundary between the Middle and Tennessee sections is at the divide between the New and Tennessee Rivers in southwest Virginia. Within the Valley and Ridge Province, only the southwest portion of the Middle section and the northeast portion of the Tennessee section are involved in the regional physiography.

A gradual change in structure of the Middle section begins to be noticeable in central Virginia. Toward the south the folds become more closely compressed and then overturned toward the northwest. The number of thrust faults increases toward the south, so that in southwestern Virginia it is the rule rather than the exception that anticlines are broken by thrust faults dipping to the southeast. The combined effect of these factors is that nearly all beds dip toward the southeast. North of the New River-Tennessee River divide the valley peneplain in the New River drainage basin is traceable throughout a belt 15 miles wide for a distance of 60 miles. Abrupt highlands bound the section on both sides.

In the Tennessee section the width of the Valley and Ridge Province is somewhat less than in the northern two sections, and the area of valley floor is a larger fraction of the total. Boundaries against the highlands on both sides continue to be clear and abrupt. The major part of the drainage makes its final escape through a transverse valley west of Chattanooga, where the Tennessee River flow into the Appalachian Plateau Province.

Of the rocks found in the Tennessee section, the sandstones are the predominant ridge-makers. The Clinch sandstone for instance, makes the prominent mountain ridges north of Knoxville. The Rockwood is another. A number of low ridges or broad swells in the lowlands are made by cherty beds in the great mass of Cambrian and Ordovician dolomite, mainly the Knox Group, which underlies more of the valley surface than any other rock. Other low ridges are made by sandy members in the Cambrian formations which are mainly shale and limestone. Most of the lowland, not on the Knox dolomite, is on Cambrian and Ordovician limestones and shales or on similar weak rock of Mississippian age.

The Tennessee section is bounded on the northwest by the Appalachian Plateau Province. These two provinces are separated by a prominent southeast facing escarpment 1,000 to 1,200 feet high, whose summit is capped by Pennsylvanian sandstones resting on Mississippian limestone.

On the southeast the Tennessee section is bounded by the Blue Ridge Province, generally with a sharp physiographic contrast.

The northeastern limit of the Tennessee section is fixed arbitrarily at the Tennessee-New River divide, where mountain ridges are numerous and valley floors narrow and high, generally near 2,500 feet. Valley floors broaden and decline toward the southwest and some of the ridges terminate and new ones appear. Clinch Mountain near the median line of the province is the one great continuing feature. The southwestern terminus of the mountain ridges is about 10 miles northeast of Knoxville. From that latitude southward to Georgia the relief features are of the lower order, (i.e., carved from the valley floor). These features include low ridges, knobs, and all stream valleys. They also include strips of smoother lowland below 800 feet whose maximum width in Tennessee is five or six miles along the main rivers, such as the Tennessee and Hiwassee.

In regard to the drainage of the Tennessee section, a longitudinal profile of this section using the general level of the hilltops in the eroded valley floor is very flat below Knoxville and steepens progressively upstream from that point to the New River divide. A similar profile using stream levels would show a slight upturn in passing from the Tennessee River basin to that of the Coosa River basin. The Tennessee descends the slope for 250 miles to Chattanooga, then turns westward into the plateau. Just beyond the low divide the Coosa begins and flows down the gentle profile. Both streams are fed by long tributaries from the southeast and receive very little water from the plateau on the northwest.

There is little direct evidence as to the time or manner in which the main features of the present drainage plan were developed. The pattern suggests that the streams flowing northwestward from the Blue Ridge may have continued their courses as antecedents across the rising mountains and plateaus to the Ohio and Mississippi Rivers just as the New River continues to do. On the other hand, drainage may have been turned to the southwest in consequent streams following development of synclines during the formation of the folds. In the former case the change awaited the growth of subsequent streams, mainly along anticlinal axes, or at least on the weaker rocks. According to this hypothesis, the branches of the growing Tennessee captured the northwest-flowing streams, one at a time, from the Hiwassee to the Watauga.

Miller[85] suggests that East Tennessee is divided into three major physiographic regions, the Unaka Mountains Province, the Valley and Ridge Province, and the Cumberland Plateau Province.

Overall, the topographic and geologic "grain" of east Tennessee is elongated northeast-southwest in conformity with the trend of the Appalachinas region. Thus the rock formations, topography, and rock structures are generally arranged in belted patterns trending about 30 degrees east of north.

Northwest of the Valley and Ridge Province lies the Appalachian Plateaus Province (Figure 2.5-1). This province ranges in width from 30 to 200 miles, and is about 1,000 miles long, extending from New York to Alabama. Along its border with the Valley and Ridge Province, is an abrupt topographic rise known as the Allegheny Front, which is called the Cumberland Escarpment in Tennessee. This escarpment is breached by the Pine Mountain thrust fault and the Sequatchie fault and anticline which are the western-most of the Valley and Ridge thrust faults. The surface expression of the Appalachian Plateau Province ranges from 1,000 to 2,000 feet above sea level and is gently rolling with localized areas of higher elevations. As shown on Figure 2.5-2, most of this province is underlain by Pennsylvanian sandstones and shales. The rocks are gently folded into a broad syncline. The drainage of this province does not show the preferred NE-SW flow of the Valley and Ridge, but instead is random.

Farther to the northwest lies the Interior Low Plateaus (Figure 2.5-1). This province is about 300 miles by 300 miles and covers most of middle Tennessee and Kentucky. Toward the center of this province lie two large shallow basins called the Nashville basin and the Lexington plain. These two basins were formed by breaching and erosion along the Cincinnati Arch, and are underlain by the Nashville and Jessamine Domes. This province is underlain predominantly by Ordovician and Mississippian limestones. Drainage is at random. At the western edge of the region lies the East Gulf Coastal Plain along the Mississippi Embayment.

Immediately southeast of the Valley and Ridge Province lies the Blue Ridge Province, which is about 15 to 70 miles wide and extends for 600 miles from Pennsylvania to Georgia. Within the Blue Ridge are the highest mountains of the eastern United States with elevations generally ranging from 1,500 to 5,000 feet and reaching a maximum of 6,684 feet. These mountains are characterized by rugged terrain, heavily forested slopes, and rushing streams with waterfalls[84].

Farther to the southeast lies the Piedmont Province, which is about 40 to 130 miles wide, about 1,000 miles long, and extends from New York southwestward to Alabama. Surface elevations in the southern portion are about 1,000 feet near the border with the Blue Ridge and decrease eastward to about 500 feet near the Fall Line. Elevations gently decrease northward and are about 100 to 500 feet near the northern portion of the province. Erosion of the underlying saprolitic soils has produced a smooth, rolling landscape.

2.5.1.1.2 Regional Tectonics and Geology

The regional tectonics and regional geology of the Watts Bar Nuclear Plant are shown on Figures 2.5-4 and 2.5-2, respectively.

The Watts Bar Nuclear Plant site is located in the complexly folded and faulted Valley and Ridge Province of the Appalachian Highlands. Other physiographic divisions of the region are shown on Figure 2.5-1 and have been discussed in Section 2.5.1.1.1.

Within the site region (radius of 200 miles) sedimentary rocks from Tertiary to Precambrian age and igneous and metamorphic rocks of Paleozoic to Precambrian age are found (Figure 2.5-2).

In the Valley and Ridge Province, as well as the Appalachian Plateaus, Interior Low Plateaus, and Central Lowland Provinces to the west and northwest, sedimentary rocks of Paleozoic age predominate. In the Blue Ridge and Piedmont Provinces to the southeast, igneous and metamorphic rocks of Paleozoic to Precambrian age predominate. Tertiary rocks are present at the extreme south-southeastern edge of the region and Cretaceous aged rocks are found along the region's western edge.

As can be seen in Figures 2.5-2, 2.5-4, 2.5-7, and 2.5-8, most of the major faulting of the region lies within the Valley and Ridge Province. These northeast-southwest trending thrust faults were formed near the end of Paleozoic time when the part of the crust now represented by the Piedmont and Blue Ridge Provinces was pushed westward against the side of the geosynclinal trough that then existed in the northwestern part of the region[52].

Insofar as the Watts Bar Nuclear Plant is located in the Valley and Ridge Province, the following discussion relates to that province.

Within the Valley and Ridge Province sedimentary rocks from Pennsylvanian to Cambrian age are found with those of Cambrian and Ordovician age predominating. In Tennessee, the Rome Formation and the Conasauga, Knox and Chickamauga Groups makeup the majority of the bedrock of the Valley and Ridge Province. They outcrop as repeated belts that trend NE-SW as the result of major Paleozoic thrust faulting from the southeast. The maximum exposed thickness of the Middle Cambrian Rome is about 1,200 feet. It is composed mostly of shales, siltstones, and sandstones. The Middle Cambrian Conasauga Group is mainly alternating shale and limestone along the southeastern border of the province and nearly all shale along the northwest border of the province. It is about 2,000 feet thick and forms the bedrock for the Watts Bar Nuclear Plant. The Knox Group is 2,500 to 3,000 feet thick and is of Late Cambrian to Early Ordovician age. It is mostly dolomite with some limestone.

The Chickamauga Group is Middle Ordovician in age and ranges in thickness from about 8,000 feet in the southeast to 2,000 feet in the northwest. It is mainly alternating layers of limestone, siltstone, and shale. Elsewhere in the Valley and Ridge, are sandstones, shales, and limestones of Late Ordovician to Pennsylvanian age.

The geologic structure of the Valley and Ridge is characterized by numerous elongate folds and thrust faults that trend northeast-southwest (Figures 2.5-7 and 2.5-8). In the southern section of the province the faults, and in most places the bedding, dip southeast. These orientations are the result of folding and fracturing during a mountain building episode 230 to 260 million years ago[84].

There is no evidence that any of the thrust faults can be considered to be active faults still undergoing movement. Geologic evidence indicates that the final episode of movement occurred during the Pennsylvanian or Permian periods, or at least 230 million years ago[107].

Rodgers[102] points out that:

> Southeast of the Cumberland Escarpment (the border with the aforementioned Appalachian Plateau Province) is a belt 16 to 20 miles wide in which almost all the rocks dip southeast, the rock sequences being repeated in belt after belt between southeast-dipping faults of large throw. Major folds are rare, and even smaller folds are found mainly close to the thrust faults. On the other hand, minor cross faults are common and minor thrust faults abundant, especially close to the major faults. The major faults that dominate this belt can be classed into three groups or families, here named after prominent members, the Kingston, Whiteoak Mountain, and Saltville families of faults. Almost all these faults bring up the Rome (Cambrian) for many miles, generally in the northeastern part of their courses; elsewhere the Conasauga Group or in places the lower beds of the Knox Group lie next southeast of them. Almost any formation above the Rome may lie next northwest; perhaps the Chickamauga limestone and its shale equivalents are the most common. In general, the dips of these faults are moderate, probably about parallel in any given area to the average dips of the rocks above them, but in several areas they are nearly horizontal.

Approximately one mile northwest of the Watts Bar Nuclear Plant lies the Kingston fault and about four miles to the southeast lies the Whiteoak Mountain fault. These faults are prominent members of two of the three families of faults that dominate Rodgers' "belt of dominant folding"--the Kingston, Whiteoak Mountain, and Saltville families. The Kingston fault begins in Anderson County, Tennessee and runs for about 175 miles southwest through Tennessee, across the northwest corner of Georgia and may extend into Alabama (Figure 2.5-7). The Whiteoak Mountain fault begins in southwest Virginia and extends for a length of about 235 miles southwestward across Tennessee into northwest Georgia.

The site is within the folded and faulted Southern Appalachian Structural Province. The province is dominated by thrust faults that extend parallel to the NE-SW regional strike for many miles[102] and is shown on Figures 2.5-7 and 2.5-8). Elongate folds, usually the synclinal limbs of broken anticline - syncline couplets, parallel the regional strike within the major thrust blocks (Figures 2.5-2 and 2.5-3). The region is also characterized by shear in the vertical plane normal to regional strike and overturning of folds to the northwest.

There are at least three contrasting models for the Late-Paleozoic Alleghanian deformation: a thin-skinned tectonic compression model, a thick-skinned tectonic compression model, and a thin-skinned gravity spreading and slide (or glide) model, each related to uplift of an eastern mobile thermal core[83]. Further discussion of this deformation will be presented in Section 2.5.1.1.4.

No significant gravity or magnetic anomalies are found near the site as seen by Figures 2.5-5 and 2.5-6.

2.5.1.1.3 Regional Geologic Setting

The Watts Bar Nuclear Plant is located in the Valley and Ridge Province of the Appalachian Highlands. This province is made up of a series of folded and faulted mountains and valleys which are underlain by Paleozoic sedimentary formations totaling 40,000 feet in thickness[52].

The Valley and Ridge Province is bounded on the northwest by the Appalachian Plateaus Province, an assemblage of predominantly Mississippian and Pennsylvanian aged sediments which are basically flat-lying and form an abrupt escarpment along the border with the Valley and Ridge Province.

Farther to the west and northwest lies the Interior Low Plateaus Province of the Interior Plains. This province contains predominantly rocks of Ordovician through Mississippian age and again is basically flat-lying (Figure 2.5-2).

To the southeast the Valley and Ridge Province is bordered by the Blue Ridge Province which is made up mostly of Precambrian granite and gneiss and Late Precambrian sedimentary rocks somewhat metamorphosed but less so than the formations in the Piedmont Province (Reference 52 and Figure 2.5-2).

Along the western edge of the Blue Ridge Province, the lower Paleozoic formations of the Valley and Ridge Province are turned up steeply at the contact with the uplifted Precambrian rocks. In places this is a fault contact and provides a sharp structural boundary between the two provinces (Reference 52 and Figure 2.5-2).

Farther to the southeast, across the Blue Ridge Province, lies the Piedmont Province. The rocks here are mostly metamorphics, such as gneiss and schist, with some marble and quartzite, and were derived by metamorphism of older sedimentary and volcanic rocks. Some less intensively metamorphosed rocks, including considerable slate, occur along the eastern part of the province from southern Virginia to Georgia. This area, called the Carolina Slate Belt, makes up about 20% of the province. Another 20% of the province is granite, or granite gneiss.

Swingle[122] suggests that in regard to the geologic setting:

> The Appalachian Mountain System, the dominant tectonic element of the Southeastern U.S. extends from the Cumberland Plateau eastward to the Atlantic Coastal Plain. Composed of miogeosynclinal deposits to the west and eugeosynclinal rocks eastward many of the structural features of this system are generally inferred to have resulted from the presumably Late Paleozoic "Appalachian revolution." However, the sedimentary and deformational history of this region is quite lengthy, and also quite complex, extending from Late Precambrian throughout the Paleozoic. Evidence for Precambrian diastrophism and later periods of deformation including plutonism, vulcanism and metamorphism is found in the metamorphic and plutonic rocks as well as in the sedimentary sequences. After the Late Paleozoic deformation the mountain system was extensively eroded, peneplaned (perhaps three times) and the present landforms carved by differential erosion.

2.5.1.1.4 Regional Geologic History

The Watts Bar Nuclear Plant area lies near the western border of what was the active portion of the Appalachia geosyncline during most of the Paleozoic Era. During the early portion of the era, in Cambrian time, sands and clays were deposited in shallow, muddy water and these consolidated to form sandstone and shales of the Rome Formation. The syncline gradually depressed and the sea became deeper and broader. At the beginning of the deposition of the Conasauga, the sea received a small amount of sand and much clay. The sediment load gradually changed until at the end of the Conasauga only limy deposits were being laid down. Throughout the succeeding Knox deposition, the sea was deep and still as indicated by the great thickness of limestones and dolomites that were deposited. At the close of the Knox, most of the area was uplifted slightly and exposed to erosion. By the Middle Ordovician the land had subsided again and was covered by a shallow and oscillating sea in which a great thickness of limestone and calcareous shale was deposited. At the end of Paleozoic time, during the Allegheny orogenic episode, the rocks at the site were folded and faulted and tilted to the southeast. Since the Paleozoic time, weathering and erosion have been the dominant geologic processes at the site with sediment accumulation being restricted to the alluvial and flood plain deposits of the Holston River.

Precambrian rocks of the region are exposed on the southeast side of the line trending northeast-southwest approximately 35 miles southeast from the site (Figure 2.5-2). They are located in the Blue Ridge and Piedmont Provinces. They have been faulted, fractured, folded and metamorphosed. Some of them appear to be igneous in origin and some sedimentary. Beneath all of the rocks exposed west of the Blue Ridge, at depths ranging from 2,500 feet to 18,000 feet, are rocks similar to those exposed in the mountains. Called the 'basement complex' these rocks underwent many changes in the more than one billion years since they formed.

Miller[84] points out that:

> The ancient basement complex is overlain in the easternmost counties of Tennessee by very old, less metamorphosed or otherwise altered sedimentary rocks of Precambrian age. These rocks form the majority of the Precambrian exposed in Tennessee and are collectively called the Ocoee series. Their age ranges from one billion to 600 million years. Most of these rocks appear to have formed in a marine or transitional environment and may have had an original combined thickness of 50,000 feet.
>
> It appears that some of the basement rocks of granitic composition were also once sediments that have been transformed by a long episode of pressure, heat, and chemical activity. Some of these crystalline rocks are, however, probably igneous in origin. Also exposed in northeastern Tennessee are lavas and tuffs in the Mount Rodgers Group. These rocks were formed by volcanoes, although the Mount Rodgers Group also contains some rocks of sedimentary origin.

Lower Cambrian rocks in Tennessee are in the Chilhowee Group. Their area of outcrop is restricted to the far eastern part of the state, where they form some of the more prominent mountains.

Miller[84] suggests that:

> These Lower Cambrian rocks in Tennessee, like those of the underlying Precambrian, indicate that there must have been land areas nearby to supply the great amounts of erosional debris making up the rocks. These sediments may have been derived from a continental area to the west and from volcanic islands to the east. If the continent of Africa was juxtaposed with North America at the time, the sediment could have come from that source.
>
> Although no rocks equivalent to the Chilhowee Group are present beneath the Central Basin of the Interior Low Plateaus Province, post-Chilhowee Cambrian rocks are preserved there. They are presumed to be the equivalent of the Shady or Rome Formation and overlie a 'granite wash'- a weathered detritus of basement crystalline rocks. This indicates that the Nashville Dome (Figure 2.5-2) must have been part of an area above water at least as early as Late Precambrian time and was being eroded throughout Early Cambrian time. The products of this erosion were washed eastward to be incorporated in the thick wedge of sediments making up the Ocoee and Chilhowee rocks. Then beginning with the later part of Early Cambrian time, the seas advanced over the area that is now Middle Tennessee with associated deposition of

carbonates with interbedded mud and sand. This was a shallow continental shelf environment.

With regard to the Ordovician period of the regional geologic history, Miller[84] suggests that:

> The deposition of carbonates that began in Cambrian time continued into Ordovician time in what is now Tennessee. The rocks that formed from the sediments deposited from Upper Cambrian to Lower Ordovician time are called the Knox Group. These rocks are exposed over wide areas in the Valley and Ridge and extend beneath the surface throughout Tennessee and other areas of the east-central United States, although they are referred to by other names elsewhere... In the Valley and Ridge the Knox ranges in thickness from 2,500 to 3,000 feet, but it thickens progressively westward to 5,500 feet in northwest middle Tennessee.
>
> The origin of dolomite, such as that in the Knox Group, has been the subject of much study. It appears that subtle changes in the environment of deposition and variations in the chemistry of sea water play a critical role in whether limestone or dolomite forms. Recent studies indicate that the calcium carbonate forms in a quiet, shallow water environment just below the lowest tides. Evaporation of magnesium rich sea water results in the alteration of this sediment to calcium magnesium carbonate (dolomite).
>
> Emergence of the land in Tennessee at the end of the Early Ordovician exposed the carbonate sediments over many thousands of square miles. Weathering began immediately and cave systems began to develop. Many of these collapsed to form sinkholes. The irregular pattern of distribution of the Pond Spring formation indicates it may have been deposited in depressions (sinkholes) on the sea floor at the beginning of Chickamauga time. Also, the presence of extensive zones in the Upper Knox of angular fragments of rocks (breccia) have been interpreted by some as debris resulting from the collapse of cave roofs.
>
> Subsidence of this weathered landscape began after an undetermined length of time. As the sea advanced over the area, lime sediment again accumulated. Rocks formed from this sediment are called the Chickamauga Group, which is about 2,000 feet thick. As it was being deposited in the northwest part of what is now the Valley and Ridge, the Athens shale was forming as a 7,000 foot thick wedge of mud sediment to the southeast. This mud represents an abrupt change in the sedimentation pattern in the area. One interpretation of this change is that a new land area to the east was formed by mountain building forces. This episode of deformation has been named the Blountian orogeny in Tennessee. The maximum deformation was to the northeast of Tennessee. Some of the structures in the Great Smoky Mountains are thought to be the result of this orogeny, although most of them are due to Late Paleozoic deformation.

> As the sediments of the Chickamauga Group were being deposited in the east, lime was also being formed in shallow water westward in Middle Tennessee. The limestone beds formed from these sediments are now exposed in the Central Basin and are present in the subsurface beneath the Cumberland Plateau and Highland Rim surfaces.
>
> Toward the end of the time during which the Stone River Chickamauga sediments were deposited, there was volcanic activity in what is now North Carolina and Virginia. Ash from these volcanoes was carried by winds across Tennessee and the seven state area where it settled in the sea and collected as distinct beds interlayered with the Chickamauga rocks in East Tennessee and those of equivalent age in Middle Tennessee. Such ash beds are now chemically altered and are called bentonites. They range up to several feet in thickness and are exposed in various places in the Central Basin and in the Valley and Ridge.
>
> Toward the end of the Ordovician period there was uplift of the Nashville Dome in the western part of the region.

In the western part of the region, deposition of carbonate sediment continued into Silurian time although there was uplift with associated erosion in some places. Mud was sporadically washed into the sea, as evidenced by shales interbedded with Silurian age limestones. Depositional conditions were considerably different in East Tennessee in Silurian time. The Clinch sandstone appears to have formed as a beach deposit on the leading edge of a westward advancing sea[84].

In East Tennessee, Silurian rocks are exposed only along the eastern Cumberland Plateau Escarpment and on a few other ridges in the Valley and Ridge. Silurian sediments were deposited elsewhere in this area, but structural deformation and erosion have made their original extent difficult to determine[84].

With regard to the Devonian period, Miller[84] suggests that:

> There was uplift in the land and some erosion at the end of Silurian time west of the present Valley and Ridge in Tennessee, but in most places it is not possible to determine how much, for two subsequent major episodes of erosion during the Devonian in some places removed all the rocks overlying the Middle Ordovician. In the Early Devonian... the lime that was collecting at the end of Silurian time continued to be deposited...
>
> When the Late Devonian sea advanced across the land, conditions had changed dramatically compared with other invasions of the ocean, and the environment was like few others in all the geologic history of this region. This sea eventually spread over much of the east-central United States, depositing a black, carbonaceous mud over hundreds of thousands of square miles. This black mud, containing rotted organic matter, became the Chattanooga shale... The Chattanooga has a maximum thickness of 1,850 feet in East Tennessee...

Most of the sediment of the Chattanooga sea was fine mud, some derived locally from residual clays on the erosional surface, and some from land areas far to the east. At the base of the Chattanooga in some areas, and interbedded with the shale in others, is sandstone that was probably derived from nearby land areas such as islands created during the prior period of uplift.

Radioactive dating of micas in the shale has shown an age of 340 million years before the present, or Late Devonian, for the Chattanooga. The problem of reconstructing the correct environment of its deposition is, however, more difficult. At the time of its deposition it apparently covered all of the Tennessee west of the Unakas. Erosion has removed the shale from West Tennessee, most of the Central Basin, and most of the Valley and Ridge where structural deformation of the region has further obscured its distribution.

With regard to the Mississippian period, Miller[84] suggests that:

Deposition of greenish-gray mud marked the beginning of Mississippian time in Tennessee...

Deposition of carbonate sediment continued through most of Mississippian time in this region and the environments in which the sediments collected were complex. Cross-bedding in some of the limestones indicates they formed in a zone of wave or strong current action. Other units seem to have been deposited in quiet water. Clastic sediments were also being carried into the sea by currents or by the wind, for there is considerable sand and shale in these rocks. These limestones, shales and sandstones have a combined thickness of more than 700 feet in Middle Tennessee and are as much as 5,000 feet in total thickness in East Tennessee.

The last formation to be deposited in Mississippian time in Tennessee was the Pennington Formation. Some of the shale formed in a transistional environment and the presence of thin coal beds indicates some swamps existed. Most of the Pennington, however, is of marine origin. The shales were formed from mud washed in from land areas, and the limestone and dolomite zones represent times when the mud and silt were mostly absent. Mississippian rocks crop out over the entire Highland Rim. Some hills in the Central Basin underlie the entire Cumberland Plateau (beneath Pennsylvanian rocks) and are present on some ridges in East Tennessee...

In summary, during most of Mississippian time in Tennessee, shallow seas, teeming with life, covered the area. There were shifting currents, migrating shorelines, and various sources of clastic sediment. Mississippian sediments once extended without break across what is now the Central Basin, but almost all have been stripped away by erosion. At the close of the period much of the land was nearly at sea level, with tidal flats and the beginnings of swamp forest into which mud was washed. Dying vegetation collected on the bottom of these swamps and eventually formed thin coal beds as the plant material was buried and compressed.

With regard to the Pennsylvanian period, Miller[84] suggests that:

> Pennsylvanian rocks in Tennessee are dominated by two major lithologies, sandstone and shale. There are lesser amounts of siltstone, conglomerate, and coal. From the study of the characteristics and inter-relationships of these rocks, their environments of deposition have been reconstructed.
>
> It appears that in many places there is no definite boundary between the Mississippian and Pennsylvanian systems in Tennessee.
>
> Such gradational conditions may be interpreted as migration of environments of deposition. There were advancing shorelines along which beach barriers existed, with lagoons or coastal swamps landward where mud and organic matter collected, and on the seaward side of the barriers carbonate and mud collected. Today these beach barriers are preserved as the dense cross-bedded locally pebbly sandstones we see on the Cumberland Plateau. The coals and shales interbedded with the sands formed in lagoons, swamps, and tidal flats. The Upper Pennington shales and carbonates were formed in a transitional and marine environment.
>
> Since there are numerous complex sequences of the sandstones, shales and coals in the Pennsylvania, they can be interpreted as oscillations of the shorelines and their adjacent environments...
>
> As the shoreline migrated, the near, relatively narrow beach area, together with tidal deltas, became extensive covers of sand which advanced over the muds formed in the lagoonal swamp areas.
>
> Other features of the rocks exhibit more clues as to their origin. The effects of channeling in the rocks can be seen in several places. Some were caused by tidal scour cutting through the beach barrier, or by currents in the tidal flats behind the beach zones. In addition to the beach barrier sands, there were tidal deltas. These deltas were wedges of mostly sand created by tidal currents moving through gaps in the beach area. Similar channels in tidal deltas can be seen along the present Atlantic shores and other coastal areas.
>
> Pennsylvanian time was generally very warm, perhaps tropical in this area, for there was profuse growth of the swamp forest. As this vegetation died, it collected on the swamp floor where it decomposed in a low oxygen environment. At first peat formed, and later, as mud washed over it or advancing sand bars encroached and covered it, the peat was compacted to form lignite. With the passage of time and greater compaction from more and more overlying sediment it became bituminous coal.
>
> The original extent of Pennsylvanian sedimentation is not definitely known, but it is likely that sediments similar to those now preserved on the Cumberland Plateau extended completely across what is now the area of the Highland Rim and Central Basin and that they were connected to Pennsylvanian sediments preserved today in Western Kentucky... Although the former extent of these

> rocks east of the plateau is also unknown, they did extend at least 15 miles east of the Cumberland Plateau escarpment...
>
> Not all of Tennessee was a great single swamp at any one time. But as the shorelines advanced or retreated, swampy areas developed throughout much of Tennessee and large parts of what is now the east-central United States. There are numerous coal beds in rocks of this age in Kentucky, Alabama, Georgia, the Virginias, Pennsylvania, and other eastern states.
>
> Only the early part of Pennsylvanian history is recorded in rocks preserved in Tennessee. These sediments have a composite thickness of about 4,000 feet. Any younger rocks of this system that might have been deposited have been removed by erosion. Therefore, the topmost Pennsylvanian beds preserved mark the last known depositional history of the state during the Paleozoic Era. Toward the end of Pennsylvanian time, an episode of rock deformation began and profoundly altered the geologic structure...

With regard to the Late Paleozoic mountain building of the Southern Appalachians, Miller[84] stresses the plate tectonics theory as advocated by Dewey and Bird[19], and suggests that:

> The Appalachian Mountain chain has a long and complex history, with some deformation that occurred as early as Precambrian time. The last major episode of orogenic activity that affected the region of the Southern Appalachians occurred toward the end of the Paleozoic Era, when sediments that had been collecting along the eastern edge of North America for many millions of years were buckled and fractured into a long, high range of mountains. This mountain-building episode has been called the Allegheny orogeny, and it resulted in the basic structures of the Appalachian provinces from Pennsylvania southward into Alabama. The extent of deformation west and south of Central Alabama is not known because of the thick sequence of younger sediments overlying the structures.
>
> One of the basic problems in understanding the complex history of the earth has been to explain adequately the origin of such great belts of folded mountains as the Appalachians. What forces could have warped and fractured many thousands of feet of sediments into lofty ranges of mountains? The movement of great plates of the earth's outer mantle and crust is a process that could have built such mountains. The concept that segments of the earth's 'skin' have, throughout geologic time, been floating or drifting like vast 'rafts' on a plastic zone in the mantle is not new. It was first suggested at the turn of this century. But the theory of plate tectonics, which refers to the movements and associated structures of these plates as an explanation for various features of the earth, has only recently been accepted. Essentially the concept involves the movement of major segments or plates of the earth's crust and the upper part of the mantle, with the plates colliding or sliding under or past each other in some places, and moving apart in others. The rate of movement is extremely slow--a maximum of about six inches per year. Yet in light of the vast length of

time such movements have been occurring, this rate of movement can account for a great distance in a short geologic time span. For example, at this rate in only 10,000 years a plate could move a distance of one mile.

Where plates meet, the one composed of heavier material (oceanic crust, composed chiefly of basalt) slides under a plate composed of lighter material (continental crust, composed chiefly of granite). Where plates move apart, new material, in the form of molten rock, oozes up from the underlying mantle to fill the gap. The plates are apparently driven into motion by uneven heating within a plastic zone in the underlying mantle. The uneven heating creates movement called convection currents. An example of a 'gap' area today is the Mid-Atlantic Ridge, a sea floor mountain chain with a great rift valley in its center. The Western and Eastern Hemisphere plates are moving away from each other along this rift. The material filling the gap is produced by volcanic activity such as that presently occurring in Iceland, which is centered on the rift. An example of plates meeting each other is the western coast of South America where the Pacific plate is 'diving under' that continent. A deep oceanic trench is present where the plates meet. There the leading edge of the South American plate is buckled into a great range of fold mountains, the Andes. There are also numerous earthquakes and much volcanic activity along this boundary.

Such regional structural features as the Appalachians can be explained by plate tectonics. This may be illustrated by the following sequence of events in Late Paleozoic (Permian) time. Movement of the oceanic plate toward the edge of the North American continental plate cause a down-warping of the sediments and the development of a trench along the plate boundary. The heavy oceanic crust (basalt) descended beneath the continent and was consumed in the underlying mantle, where it was melted in part to form magma. The beat of the molten material caused the mass to rise and form a 'swelling' above it. This upward movement initiated faulting by the pressure it created, and also some gravity sliding of sediments on the flanks of the uplifted mass. This molten mass also moved outward, away from the trench, creating lateral forces which caused thrust faults and folding toward the continental shelf area. Granite intrusions were also formed in the upper zone of the molten core (Piedmont granites).

The great folds and thrust faults now seen in the Unakas may have been formed in this general manner. The combined belts of Piedmont (overlain by younger sediments to the east), Blue Ridge (Unakas), Valley and Ridge, and Cumberland Plateau (that portion that is structurally deformed) are over 300 miles wide in some places. Such massive deformation over a length of at least 1,000 miles required enormous upward and lateral pressures. The present topography of the Unakas and Valley and Ridge is the result of the erosion of rocks uplifted, tilted, or folded linear belts of these rock in the valley have variable resistance to erosion, and the weathering-erosion processes have left more resistant units as ridges, but 'weaker' rocks have been cut into valleys.

> The Cumberland Plateau makes the western-most deformation in Tennessee during the Allegheny orogeny, with thrust faults and bedding-plane faults extending well into this area. Some beds are vertical or are overturned.
>
> By the end of Permian time the structural framework of the Southern Appalachians was nearly complete. During the next period, the Triassic, there was block faulting and some associated igneous activity to the east (from the Connecticut Valley to South Carolina), but it cannot be determined that this deformation had important effects on Tennessee rocks.
>
> Although the present extent of Permian rocks in the Eastern United States is very restricted and none are present in Tennessee, this period was one of great importance in the geologic history of our state. After the Permian much of the eastern interior of the North American continent was above sea level never to be inundated to the present time. There began a long episode of erosion, lasting into Cretaceous time in West Tennessee and up to the present in East Tennessee. This represents a minimum of 135 million years with no preserved rock record. Only the features produced by mountain building and erosion give us clues as to what occurred during that great expanse of time in Tennessee.

In regard to the mechanics of Valley and Ridge deformation, reference [98] suggests that:

> There are presently at least three contrasting models for the Alleghanian deformation: a thin-skinned tectonic compression model, a thick-skinned tectonic compression model, and a thin-skinned gravity spreading and slide (or glide) model, each related to uplift of an eastern mobile thermal core[83].
>
> A thin-skinned tectonic compression model would consist first of uplift and thrusting in the eastern Appalachians. A thin slab of miogeosynclinal and shelf sediments many hundreds of square miles in extent would then be pushed westward and deformed from east to west above a regional decollement.
>
> A thick-skinned tectonic compression model would result from the generation of compressional forces in the eastern Appalachians (Piedmont-Blue Ridge) to such an extent that flowage and faulting were induced westward across the miogeosyncline to the shelf. Major faults would extend into the basement, and basement rocks would flow upward into the cores of Alleghanian anticlines.
>
> A gravity spreading and slide model requires vertical uplift in the eastern Appalachians. Rocks of the Cumberland Plateau, Valley and Ridge and Blue Ridge would slide westward above a regional decollement which had a net westward slope, and would be deformed from west to east when the block broke up along its toe.

The attitude of southern Appalachian thrust faults at depth has been of major concern to geologists during the past few decades. Rodgers argues that the major thrusts do not extend downward into the subjacent basement. Rather, he maintains that the faults flatten at depth in or beneath the Rome Formation, and extend under the Valley and Ridge as a regional decollement that separates sedimentary strata beneath the decollement and the crystalline basement rock from younger Paleozoic strata. Cooper, following the ideas of E. O. Ulrich and Charles Butts, denies the regional decollement concept. Rather, he maintains that Appalachian folds and faults are rooted and reflect ancient movements in the subjacent basement. Geophysical studies have been used to illustrate both structural concepts. Sears and Robinson used gravity data to show that basement was involved under the Bane anticline in Virginia. Edsall has cast doubt on their findings with his seismic reflection study of the same structure. Watkins, using gravity and magnetic data, has supported the thin-skinned concept. A recent reflection seismic profile made in the site area by Geophysical Services Incorporated clearly supports the thin-skinned concept (Figure 2.5-160).

Articles that have been published regarding the Consortium for Continental Reflection Profiling (COCORP) reflection profiles support the concept of thin-skinned tectonics for the Appalachians. Brown and others[157], and Cook and others[157], in their abstracts submitted at the American Geophysical Union National Meeting in Washington, D.C. (May 27 to June 1, 1979), presented their interpretation that the Blue Ridge and much of the Piedmont are allocthonous and that the Brevard fault zone is rooted to a larger horizontal thrust. Cook and others subsequently published their findings in Geology[157]. TVA continues to support the thin-skinned concept and anticipates that future COCORP and other findings will prove this concept factual.

The oldest faults and folds of the Alleghanian orogeny are beneath the Cumberland Plateau and northwest side of the Valley and Ridge Province suggesting that thin-skinned gravity sliding was the mechanism of formation of Valley and Ridge structure[83]. Repetition of the Rome Formation on the seven thrust faults along the western side of the Valley and Ridge is evidence that a regional decollement extends under the Valley and Ridge in the Rome.

Pennsylvanian strata are involved in folds and faults on the northwest side of the Valley and Ridge, and Mississippian strata are broken and folded across the Valley and Ridge. This is evidence that latest Valley and Ridge deformation is post-Carboniferous. Coastal plain strate of Cretaceous age extend unfaulted across the Valley and Ridge in Alabama, and Triassic igneous dikes transect the Valley and Ridge in Virginia, proving the age of Valley and Ridge deformation is Permian (Late Paleozoic).

There is no direct evidence regarding fluid pore pressures in the Rome during sliding. However, if high pressure existed it seems likely that elevated pore pressures were in the Rome, which is considerably more sandy than the overlying Conasauga and probably had more porosity and permeability. The thick shales and limestones of the Conasauga could act as an excellent seal for any high pressure zone below. Upon thrusting, the seal eventually was broken, pore water could then escape, and movement no longer occurred in the decollement zone.

The Rome presently is highly indurated and well cemented. For this reason it is very unlikely that pore fluids could be reintroduced into the formation in sufficient quantities over a large region in order to reestablish the conditions that existed during thrusting. Before thrusting could resume, a tilt of the sedimentary units would have to occur (gravity slide model) or severe compressional forces would have to be initiated (thin-skinned compressional or thick-skinned compressional model). The tangential forces, whether compressional or gravity, that provided the impetus for thrust movements are very unlikely to recur.

Since early 1900, many prominent geologists, including Keith, Rodgers, Gwinn, Harris, King, Furguson, Milici, Miller, Stearnes, Wilson and Swingle, have recognized that the faults in the Valley and Ridge flatten with depth. The significance of the recent confirmation of the thin-skinned concept (Figure 2.5-160) is that the faults outcropping at the surface are not related to geologic structures in the basement. In 1973, G. D. Swingle estimated that basement surface to be at a depth of 13,000 feet. This means that the infrequent earthquakes which occur in the Valley and Ridge with normal hypocenters do not relate in any way with the ancient (noncapable) faults outcropping at the surface.

Further discussion of the Valley and Ridge faults is found in Section 2.5.3.

The Mesozoic Era was characterized within the region mostly by erosion that has continued until the present. However, deposits are found along the region edge from west-northwest to southwest along the Mississippi Embayment (Figure 2.5-2).

The Cenozoic Era is represented in the region only by the Tertiary deposits that lie at the extreme south-southeastern edge of the region.

2.5.1.1.5 Regional Lithologic, Stratigraphic, and Structural Geology

The majority of the bedrock within the Valley and Ridge Province is comprised of the Rome Formation and the Conasauga, Knox, and Chickamauga Groups. As can be seen on Figures 2.5-2, 2.5-3 and 2.5-9, these formations occur repeatedly in northeast-southwest trending bands as a result of Paleozoic thrust faulting that is described in Sections 2.5.1.1.2, 2.5.1.1.4, and 2.5.1.1.6.

The Rome Formation is Middle Cambrian in age and it is about 1,200-1,500 feet thick[33]. It is predominantly a variegated (red, green, yellow) shale and siltstone; however, there is gray, fine-grained sandstone in the middle and western parts of the Valley and Ridge and abundant limestone and dolomite in the eastern parts. The Rome is considered the basal sedimentary formation in the Valley and Ridge and as shown on Figure 2.5-3, at or near its base probably lies the 'sole fault' from which the major thrust faults emerge and come toward the surface. A complete discussion of the theory of this faulting is provided in Section 2.5.1.1.4. Beneath the Rome in the Valley and Ridge Province is the Precambrian 'basement complex.'

The Conasauga Group is Middle and Late Cambrian in age and is about 2,000 feet thick. It is composed mainly of shale and limestone in the southeastern portion of the province, and is predominantly shale near the northwestern border. The Knox Group is of Late Cambrian to Early Ordovician age and is 2,500-3,000 feet thick. The Knox in the center and northwestern belts of the Valley and Ridge is siliceous, well bedded, and predominantly dolomite, and magnesian limestone. To the southeast, much dark limestone is present and the rocks are only sparsely cherty.

The Chickamauga Group is of Middle Ordovician age and ranges in thickness from about 2,000 feet in the northwestern part of the region to about 8,000 feet in the southeastern part. It is composed of alternating layers of gray and maroon limestone, calcareous siltstone, and shale.

The remaining portion of the Valley and Ridge is underlain by sandstones, shales and limestones of Late Ordovician to Pennsylvanian age, and lesser amounts of Early Cambrian dolomite.

The geologic structure of the Valley and Ridge is characterized by elongate folds and thrust faults that trend northeast-southwest. The faults dip toward the southeast. Two geologic sections (Figures 2.5-3 and 2.5-11) are provided to show the structural relationships of the Watts Bar Nuclear Plant site to the region. Figure 2.5-3 is essentially normal to the regional trend and indicates the relationship of the Valley and Ridge to the adjacent Blue Ridge and Appalachian Plateau Provinces. Figure 2.5-11 is a geologic section through the site area and illustrates the structural relationship of the site to the site area. It also illustrates the relationship of the Kingston and Whiteoak Mountain faults to the site. Discussions of this faulting have been represented in Sections 2.5.1.1.2, 2.5.1.1.4, and 2.5.1.1.6.

Immediately southeast of the Valley and Ridge Province lies the Blue Ridge Province (Figure 2.5-1). Within this province, rock units predominantly consist of slate, phyllite, schist, gneiss, granite, pegmatite, and quartzite. These are Precambrian and Lower Paleozoic metamorphics generally of amphibolite grade. The schist and gneiss are considered the oldest rocks of the region[98]. Recent radiometric dating places the peak of Paleozoic metamorphism at a minimum of 430 million years before present (BP)[9]. Pegmatites are younger with recorded age determinations of 380 million years BP[9]. The Blue Ridge Province is highly deformed and its northwestern boundary generally coincides with major faults, along which metamorphic rocks have been thrust to the northwest over younger unmetamorphosed sedimentary rocks of the Valley and Ridge (Figure 2.5-3).

Farther to the southeast lies the Piedmont Province (Figure 2.5-1) which is underlain by metamorphosed volcanic and sedimentary and igneous rocks (Figure 2.5-2). These rocks have been faulted and intensely distorted. Radiometric dates indicate that regional metamorphism occurred in this province at the time period of 300 to 520 million years BP[9].

Immediately northwest of the Valley and Ridge Province, lies the Appalachian Plateaus Province (Figure 2.5-1), which is underlain predominantly by sandstones, and shales of Pennsylvanian age (Figures 2.5-2 and 2.5-3). Rock strata are gently folded into a broad syncline (Figure 2.5-3).

Farther to the northwest lies the Interior Low Plateaus Province (Figure 2.5-1), which is underlain predominantly by sedimentary rock mostly limestone of Ordovician and Mississippian age with lesser amounts of sandstone and shale of Pennsylvanian age

(Figure 2.5-2). The rock strata are gently inclined over the Cincinnati Arch, which includes the Nashville and Jessamine Domes (Figure 2.5-4).

2.5.1.1.6 Regional Tectonics

The tectonic map of the region surrounding the Watts Bar Nuclear Plant is given as Figure 2.5-4. This site is located in the folded and faulted Valley and Ridge physiographic province, as described in Section 2.5.1.1.1. This province is characterized by elongate northeast-southwest trending ridges and valleys formed by a series of echelon thrust faults that dip to the southeast and commonly cause the overturning of the strata to the northwest (Figures 2.5-3, 2.5-7, and 2.5-8). Immediately northwest of the Valley and Ridge Province lies the Appalachian Plateaus Province that regionally exhibits strata that are gently folded into a broad syncline. Farther to the northwest within the Interior Low Plateaus Province lies the Cincinnati Arch, a northeast-southwest trending arch with two structural domes, the Nashville and Messamine Domes, located on either end with the Cumberland Saddle between. To the southeast of the Valley and Ridge Province lies the Blue Ridge Province, composed of an assemblage of severely contorted Precambrian sedimentary and metamorphic rocks. Farther to the southeast, beyond the Bervard fault zone, lie the Paleozoic metamorphics and intrusives of the Piedmont. Swingle[122] describes the basic tectonic units of the region as follows:

Cumberland Plateau

West of the Valley and Ridge this province is mainly comprised of relatively flat-lying Pennsylvanian and older sediments totaling a mile or more in thickness. The main structural features are the Pine Mountain and Cumberland overthrusts along which upper layers of the sedimentary sequence have moved short distances westward. A spectacular faulted anticline (Sequatchie) occurs near the middle of the region extending over 150 miles in Tennessee and Alabama. Its axis is nearly straight, paralleling the main structural lineament of the Appalachians, and is essentially horizontal. The fold is breached by erosion resulting in a valley, three miles or so wide, whose floor is 1,000 feet or more below the general level of the Plateau.

Valley and Ridge

In this province, 40 or so more miles across in Tennessee, is a generally conformable sequence of Paleozoic (Cambrian to Pennsylvanian) sediments of varying thickness but which probably averages three or so miles. Imbricate thrusts, mainly southeast dipping, repeat segments of the sequence across the province. These faults are of large stratigraphic throw and some extend for hundreds of miles along the structural grain of the Appalachians. The faults in the southeastern portion of the province have in their thrust blocks clastic sediments which coarsen and thicken south-eastward in marked contrast to the generally carbonate sequence to the northwest. The Valley and Ridge rocks although folded tightly in places and locally strongly crushed, are relatively unmetamorphosed. Weak fracture cleavage is present in select rock types near the middle and eastern portions of the region but slaty cleavage has not been observed in the rocks of this province.

Blue Ridge

This province can be conveniently divided into two north-east trending segments, one dominantly sedimentary in the northwest and the second, dominantly crystalline, to the southeast. The crystalline segment, about 30 miles wide at the latitude of Gatlinburg, is composed chiefly of mica, hornblende and other types of gneisses and schists plus granitic and ultramafic intrusives. These rocks are considered mainly Precambrian and at least in part constitute the basement complex upon which the sediments of the eastern Blue Ridge were deposited. The dominantly sedimentary portion of the Blue Ridge, near Gatlinburg, is Precambrain (with but minor early Cambrian rocks). Two sequences of chiefly clastic rocks are present, the Lower Cambrian Chilhowee Group, about a mile thick, which extends for several hundred miles eastern Blue Ridge, and the underlying Ocoee sediments which are really quite restricted. The Ocoee clastics are at least five miles thick and possibly twice this amount. They are, in contrast with the overlying Chilhowee, metamorphosed to slates, phyllites, and locally schists, and have obviously been deposited in quite a different environment than the overlying Paleozoic rocks.

The structural features of this portion of the Blue Ridge are world famous, this being the classic Appalachian area of tremendous overthrusts as evidenced by several fenster areas and klippen. The Grandfather Mountain Window, 75 miles northeast of Gatlinburg, is evidence of the 30 mile westward transport of the Blue Ridge. Near Gatlinburg the limestone-floored cove areas (Tuckaleechee, Cades, Wears, and others) are windows in the Blue Ridge thrust sheet and indicate westward horizontal movements approximating 10 miles.

Piedmont

This province, over 150 miles across, is the least deciphered of the Southern Appalachians. The surficial distribution of its rocks is generally known but their ages, stratigraphic, structural and metamorphic relationships are poorly understood. The

Piedmont is commonly divided into somewhat linear belts which generally parallel the northeast strike of the Appalachians. From the Atlantic Coastal Plain westward to the Blue Ridge the more conspicuous of these belts are:

(1) The Carolina slate belt, some 50 miles wide and having a strike length of over 400 miles. The rocks of this area are characteristically felsic and mafic volcanics intercalated with partly tuffaceous siltstones and slates. Their thickness is unknown but is probably over two miles. These rocks are but slightly dynamically metamorphosed but near plutons are strongly altered. Their age is unknown but they are presumably Precambrian or Early Paleozoic.

(2) The Charlotte Plutonic Belt west of the slates is about 30 miles across and typified by rocks of approximately granitic composition. These intrusives are believed to be of Paleozoic age, possibly spanning several periods of that era.

(3) The Kings Mountain Belt, a narrow dejective zone of Late Precambrian or Early Paleozoic quartzite, marble, conglomerate and schist, separates the slate belt and the Inner Piedmont.

(4) The Inner or Western Piedmont, 50 miles wide, dominated by mica gneisses and schists. Here the regional metamorphism of the Appalachians peaks in the sillimanite-garnet zone. The age of the rocks of this area was long believed to be Precambrian but the possibility of a Paleozoic age is now entertained. Granitic rocks are also present here as well as significant ultramafic plugs and dikes.

(5) The Brevard Belt separates the Inner Piedmont and Blue Ridge. This belt, a narrow zone of phyllite, schist and locally limestone, is characterized by its rocks being less metamorphosed than those in adjacent belts. Some workers surmise that this belt is a major fault zone; others believe it is synclinal.

As can be seen on Figure 2.5-2, most of the major faulting of the region lies within the Valley and Ridge Province. These northeast-southwest trending thrust faults were formed near the end of Paleozoic time, when the part above the crust now represented by the Piedmont and Blue Ridge Provinces, was pushed westward against the side of the geosynclinal trough that then existed in the northwestern part of the region[52].

There were several episodes of tectonic activity during the Paleozoic Era. However, one episode caused the major deformations of the rock strata in the vicinity of the Watts Bar Nuclear Plant[107]. This is referred to as the Allegheny episode, which occurred during either the Pennsylvanian or Permian periods, at least 230 million years ago.

It is generally accepted that these thrust faults do not extend into the basement, but are bounded below by a lateral sole fault in some relatively incompetent formation above the basement. The consistent repetition of the Rome Formation as the basal formation of the thrust blocks substantiates that the thrust blocks do not extend to the basement. In 1973 G. D. Swingle[123] prepared a cross section within the central Tennessee section of the Valley and Ridge Province, which shows the basement to be at a depth of 13,000 feet and a sole fault to be at a depth of 9,000 feet. This recent publication supports the previous concept of a sole fault above the basement rocks[145].

Additional discussion of these thrust faults concern their age and mechanics of formation and are found in Sections 2.5.1.1.2, and 2.5.1.1.4.

There are no specific leveling data which can be used to determine if regional uplift or subsidence is occurring in the area. However, if minor adjustments are occurring, they are regional phenomena whose effects would be uniform over the site and the surrounding area and no differential stresses would be imposed on the plant structures.

Consultation with members of the Tennessee Department of Transportation and inter-TVA organizations reveals no evidence of foundation stress relief resulting in the movement of any major structure, such as a bridge, dam, or steam plant within the Valley and Ridge.

Slightly acidic ground water produces solutioning in carbonate rocks. The extent of solutioning is dependent upon the mineralogic composition of the rocks. In those areas where the rooks are limestones and dolomites, solutioning is most severe. The degree of solutioning decreases as the rocks grade toward more siliceous and clayey sediments. In those sediments which do not contain carbonate materials, solution is negligible[98]. Solutioning in Valley and Ridge carbonates generally advances along structural features, such as joints and bedding[36, 50]. Advanced stages of solutioning produces nearly planar zones which diminish in size with depth.

Solutioning of carbonate rock is expressed at the ground surface by surface depressions and dropouts commonly referred to as sinks. Sinks generally result from the raveling of soil overburden into the underlying caves and fissures. The shapes of the sinks are governed by the extent to which raveling has progressed, and in the overburden thickness. As expressed above, the higher the carbonate content, the more extensive and severe is the solutioning. The rock at the plant is the Conasauga Formation, an interbedded limestone and shale. Severe karstic features typical of high carbonate rocks, such as the Knox, are not found anywhere within the Conasauga Formation.

There are no waste injection wells in use in East Tennessee, except at the Oak Ridge reservation, located approximately 35 miles to the northeast from the plant. Therefore, no problems will be encountered as a result of injection of materials into the subsurface strata.

No problems will be encountered by man's activities, such as mining or fluid extraction since none of these activities, except the removal of minor amounts of ground water, have been or will be carried on beneath any of the structures. Even if ground water were extracted near the plant, it would not affect the geologic competence of the foundations since we are dealing with highly consolidated hard rocks. Water beneath the plant site flows through fractures in the rock and in pore spaces in the overburden. The permeability of the foundation rock is low. A discussion of site ground water conditions is presented in Section 2.4.13.

TVA will assure that no mineral extractions will take place within the site exclusion radius.

Coal and petroleum are presently being extracted from beneath the Cumberland Plateau several miles to the west of the Watts Bar Nuclear Plant. But no known commercial quantities of either are known to exist in the Valley and Ridge Province of Tennessee[32]. There are no existing or abandoned coal mines, nor gas or oil wells, within five miles of the site.

2.5.1.1.7 Groundwater

The Watts Bar Nuclear Plant is regionally a part of the Valley and Ridge ground-water system, as described in Section 2.4.13. The source of recharge is precipitation, which averages about 50 inches annually, of which an estimated 8 to 10 inches reaches the water table. In this region, water levels normally reach peak elevations in February and March and are at minimum levels in late summer and early fall. Water occurs regionally under watertable conditions and moves relatively short distances, generally less than one mile before being discharged to springs and watercourses. Most ground water occurrence and active movement are at depths of less than 300 feet.

Geologic formations of the region consist of dolomite, limestone, shale, and sandstone. Regionally, few of the shale formations and none of the sandstone formations have significant groundwater potential.

The most significant water bearing formation in the region is the Knox Dolomite, in which water occurs in solutionally enlarged openings formed along bedding planes and fractures. Discharge from the Knox is a major source of base flow in streams. Large springs, yielding a million gallons per day or more, are fairly common in outcrop belts of the Knox.

The Conasauga Formation, on which the plant site is located, is one of the poorer aquifers of the region and does not normally yield more than 10 gpm to a well.

2.5.1.2 Site Geology

2.5.1.2.1 Site Physiography

The Watts Bar Nuclear Plant is located in the Tennessee section of the Valley and Ridge Province of the Appalachian Highland (Figure 2.5-1). The regional physiography has been discussed in Section 2.5.1.1.1.

The Watts Bar Nuclear Plant is located near the western edge at a shallow bend of the Tennessee River known locally as McDonald Bend. This bend is on the west side of the river and lies between river miles 528 and 529. The plant is located in Rhea County, Tennessee, about one air mile downstream from the Watts Bar Dam.

Figures 2.5-9 and 2.5-10 are submitted as a composite geologic map of the plant area and cover a radius of five miles from the site. Topography is shown on these figures based on a 20-foot contour interval. As can be seen on these figures, topography is controlled by the underlying formations. The plant is founded on bedrock that was overlain by alluvial terrace deposits with the average elevation around 735. To the west of the plant lie knobs of the Rome Formation reaching elevation 900. A floodplain surrounds the bend on the inside of the meander. The general elevation of this flood plain is about 700. The general water surface of the Tennessee River is elevation 683.

With regard to the age of these alluvial deposits, Rodgers[102] suggests that:

> Much of the bottom alluvium along the rivers and smaller streams is clearly very young, some bodies having been deposited in their present position within historic time. The older parts of it, however, may date back several tens of thousands of years. The alluvium of the terraces is older, of course, than the bottom alluvium, and that of the higher terraces than that of the lower. Though there is little direct evidence on the age of these deposits, they are probably largely, if not entirely, Pleistocene.

As shown on Figures 2.5-9 and 2.5-10, southeast across the Tennessee River are northeast-southwest trending ridges rising to elevation 1100. Northwest from the plant lie knobs reaching to elevation 1000.

2.5.1.2.2 Site Lithologic, Stratigraphic, and Structural Geologic Conditions

Of the numerous sedimentary formations of Paleozoic age in the plant area (Figures 2.5-9 and 2.5-10) only one, the Conasauga formation of Middle Cambrian age, is involved in the foundation for the proposed plant. At the site the Conasauga is overlain by high level terrace deposits laid down by the Tennessee River in one of its ancestral courses and by more recent alluvial deposits near the present lake shore. The Rome Formation underlies the Conasauga to the northwest and to the southeast, across Chickamauga Lake, the Conasauga in turn is overlain by limestone and dolomite of the Knox group. Neither of these formations will be involved in plant construction.

The unconsolidated deposits overlying bedrock are composed primarily of alluvial deposits on the elevated flood plain near the lake shore and terrace materials, deposited by the Tennessee River when flowing at a higher level, over the bench that covers most of the site area. The alluvium is composed of fine-grained, finely sorted, silts and clays, with micaceous sand and some quartz gravel. The thickness of the unit varies, but excavations showed an average thickness of 30 feet. Near the base of the terrace bench the alluvial deposits thin out to a feather edge. Included in the alluvial material are some fairly well defined beds of tough, blue-gray clay, containing carbonized fragments of wood. These are interpreted as old slough fillings.

The terrace deposits are much older than the recent flood plain deposits and their edge is marked by a distinct topographic bench some 30 feet high which lies from 200 to 1000 feet northwest of the edge of Chickamauga Lake. Drillings showed the thickness of the terrace deposits to vary from a minimum of 30 feet to a maximum of 46 feet.

Approximately the upper half of the unit is composed of sandy, silty, clay and the lower half is much coarser, consisting of pebbles, cobbles, and small boulders of quartz or quartzitic sandstone embedded in a sandy clay matrix.

In contrast to the conditions at the Sequoyah site, very little residual material derived from weathering of the underlying shale is present under the terrace deposits at the Watts Bar site. In excavations a foot or two of residual clay was encountered, but in most instances the terrace deposits are immediately underlain by a few feet of soft, but unweathered, shale.

In regard to the residuum, Rodgers[102] suggests:

> The residual mantle in East Tennessee, as elsewhere in the south, is commonly very characteristic of the individual formations over which it lies, and can often be used for their identification;... Relatively pure limestone and dolomite produce a deep fairly clay residuum, normally sharply set off from the bedrock... In places residuum over limestone or dolomite has accumulated to depths of hundreds of feet, especially where the bedrock contains chert, or silica that on weathering forms chert, so that the residuum is protected from sheet erosion... Impure limestone, on the other hand weathers less deeply (though equally irregularly); the weathered material is less sharply set off from the unweathered and commonly it grades from merely leached material next to the bedrock into thoroughly reconstituted residuum or soil toward the surface of the ground.
>
> Much of the generally shallow residuum over shale retains the original volume of the bedrock and such structures as bedding and fossils, but its calcium carbonate or other cementing material is leached, so that typically the rock is converted into weak, punky silty clay, and its iron is oxidized and redistributed along cracks. In general the residuum grades down into the unweathered shale. Over some shale the upper part of the residuum is more thoroughly broken down to a clay soil, but over much of it weathered shale chips persist to the grass roots and are turned up abundantly by the plow...

As to the age of the residuum, Rodgers[102] further reveals that:

> The age of the residuum is even less definite. Weathering is going on and presumably some residuum is being formed now...

As previously suggested by Rodgers[102], there is no sharp interface between the residuum and the sound rock. Instead, a variation of weathering conditions exists, grading from soil down to unweathered rock, unlike most limestone areas where sharp interface exists between soil residuum and virtually unweathered rock.

The bedrock at the site is the lower portion of the Conasauga Formation of the Middle Cambrian age. The Conasauga is of interbedded shale and limestone.

The shale, where fresh and unweathered, is dark greenish-gray to maroon, banded, and fissile. Because of the structural deformation to which the Conasauga has been subjected, the fissility of the shale is emphasized. In many instances the shale in cores from a structurally complex zone is recovered as fissile, platy fragments. The limestone interbeds are medium gray, medium crystalline, sandy and contain thin zones of glauconite grains scattered throughout. Thickness of the individual limestone beds is usually less than six inches.

The composite Geologic Map of the Plant Area is submitted as Figures 2.5-9 and 2.5-10. The subsurface geologic structure through this area is exhibited as Figure 2.5-11 entitled Geologic Section through the Plant Area. These three drawings are based on a circle of five-mile radius centered on the site.

As shown on Figures 2.5-9 and 2.5-10, approximately 1000 feet northwest of the plant is the inferred belt of the Rome Formation, which is subsequently in fault contact with the Chickamauga Group. Approximately one mile to the southeast lies the nearest outcrop of the Knox Group. Descriptions of these formations are provided on Figures 2.5-9 and 2.5-10.

The structural relationships of these formations to the site are shown on Figure 2.5-11. Insofar as they are quite distant from the site and are of no concern in regard to the plant, they will not be discussed further.

The presence of intense folding and minor faulting within the foundation bedrock at the Watts Bar Nuclear Plant is shown in Figures 2.5-109 through 2.5-138.

Cores taken from the site prior to excavation were contorted, folded, sheared, faulted, contained calcite-filled fractures and often contained slicken-sided bedding surfaces.

Dips of most cores (Figures 2.5-14 through 2.5-69) were taken periodically down the core and are shown on the geologic log. Cores frequently exhibited dips ranging from vertical to horizontal and containing every angle between. Abrupt changes in dip were seen in the cores, indicating minor faulting or shearing. Excavations revealed this to be the case. Geologic logs of borings are provided as Figures 2.5-14 through 2.5-69.

2.5.1.2.3 Site Structural Geology

The geologic structure, as well as the lithologic and stratigraphic sequences, have been discussed in Section 2.5.1.2.2 and will not be repeated here. Figures 2.5-9 and 2.5-10 present formation sequences and descriptions.

As was true at the Sequoyah site, the controlling feature of the geologic structure at the Watts Bar site is the Kingston fault. The trace of the fault lies along the prominent ridge approximately one mile from the site area (Figures 2.5-9 and 2.5-10). This is one of the major overthrusts characteristic of the Valley and Ridge province which developed at or before the close of the Paleozoic movement.

As shown on the geologic section on Figure 2.5-11, the Kingston fault dips to the southeast, under the plant site, and along it steeply dipping beds of the Rome Formation have been thrust over gently dipping strata of the Chickamauga Limestone. The distance of the fault from the plant site, slightly over one mile, and the dip of the fault plane, 30° or more, indicate that the plane of the fault is at least 2,000 feet deep at the site.

The highly deformed character of the Conasauga Formation at the Watts Bar site is a function of its lithology and structural history. Lithologically, the formation consists of several hundred feet of interstratified shale and limestone. In the plant site area, shale beds make up 84% of the formation and limestone beds the remaining 16%. The shale strata are much less competent than the interstratified limestone strata. The general strike of the strata is N30°E, and the overall dip is to the southeast, but the many small, tightly folded, steeply pitching anticlines and synclines result in many local variations to the normal trend.

Stratigraphically, the Conasauga Formation is overlain by 2,500 to 3,000 feet of massive dolomite and limestone of the Knox Group and is underlain by 800 to 1,200 feet of sandstone and shale of the Rome Formation. Sometime in the course of the Appalachian orogeny, these formations were thrust northwestward on the Kingston thrust sheet, which overrode the underlying rocks for an undetermined distance. Before the thrusting ceased, the belt of Conasauga on which the plant site is located was compressed between the two massive blocks of the much more competent underlying Rome Formation and the overlying Knox Group. As a result of the very marked difference in competency between the limestone and shale in the Conasauga, and the much greater disparity between the competency of the Rome and Knox and that of the Conasauga, the latter was folded, contorted, crumpled, sheared, and broken by small faults.

2.5.1.2.4 Surface Geology

An area based on a circle of a five-mile radius extending from the site was mapped in detail and presented as Figures 2.5-9 and 2.5-10, Geologic Maps of the Plant Area. These two figures each contain segments that together cover the area investigated. This procedure was followed in order to minimize the drawing reduction for inclusion in the FSAR. Full-size copies of a single-sheet drawing of this map can be provided informally.

The geologic findings presented on Figure 2.5-11 conform with the previous broad interpretations of the Tennessee Division of Geology's published and unpublished field sheets, Rodgers[102], and TVA file maps of the Watts Bar area. These sources of information were consulted frequently during the course of the mapping program. All strikes and dips represent locations of actual data collection by TVA, although more were collected and plotted but for the sake of clarity only the representative ones were plotted on the final drawings. Dashed lines indicate approximate contacts and dotted lines represent inferred contacts. Detailed mapping of the foundation bedrock was performed by the project geologist and maps, sections and photographs are submitted as Figures 2.5-110 through 2.5-138.

2.5.1.2.5 Site Geologic History

The geologic history of the area including the site is discussed in Sections 2.5.1.1.2, 2.5.1.1.5 and 2.5.1.1.6. The depositional and tectonic history described in these sections apply to the site as well and will not be repeated here.

The Watts Bar Nuclear Plant area has been above sea level since at least the close of the Paleozoic Era, and the present physiographic configuration is the result of erosional processes over the last 230 million years.

Rodgers[102] points out that:

> The age of the Valley Floor and associated surfaces is not certain; the relatively small amount of subsequent erosion argues for a Late Cenozoic through pre-pleistocene age, but certain sinkhole deposits found on the Valley Floor surface are thought to be very Early Cenozoic...
>
> No evidence of the age of the Upland surface can be obtained in East Tennessee and vicinity, but it must be much older than the Valley Floor surface.

The local stratigraphic sequence is provided on Figures 2.5-9 and 2.5-10. This is a composite explanation for both figures, as described in the figure notes.

2.5.1.2.6 Plot Plan

The site was first explored in 1950 when twenty holes, totaling 580 feet, were drilled. Subsequent work was done in June, 1970, when 49 top of rock determinations were made by refraction seismic methods. Core drilling of 56 holes was begun in July, 1970, and continued into September, 1970. Drilling was performed using NX-wireline core drills. Drill layouts are provided as Figures 2.5-12 and 2.5-13. Special studies consisting of borehole television observations of selected core holes, down hole elastic moduli determinations, Menard pressure bulb tests, and cross-hole rock dynamic studies were carried out and are submitted as Figures 2.5-70 through 2.5-109.

Excavation maps, sections, and photographs are supplied as Figures 2.5-110 through 2.5-138.

2.5.1.2.7 Bedrock Foundation Characteristics

Figures 2.5-110 through 2.5-111 show the relationship of the Category I structures and turbine buildings to the underlying geologic conditions. The in situ engineering characteristics of the bedrock beneath the two reactors are provided on Figure 2.5-109. These surveys were performed using the cross-hole velocity technique between selected drill holes in each reactor.

Rock properties are discussed in Sections 2.5.1.2.12 and 2.5.4.2.2.

2.5.1.2.8 Excavation and Backfill

Reference Section 2.5.4.5.

2.5.1.2.9 Evaluation of Geologic Conditions

Behavior of surficial material and substrata during earthquakes can only be inferred in a general way, because no evidence of movement in recent times has been found, as discussed in Section 2.5.1.1.2 through 2.5.1.1.6. The fact that the site is located in the folded and faulted Valley and Ridge Province indicates that the area was subjected to movement, which has been previously discussed and dated as pre-Mesozoic.

As shown on Figures 2.5-110 through 2.5-138, faults and folds exist in the foundation bedrock. The mechanics of this folding has been related to stresses imparted during the Paleozoic when the less competent Conasauga Formation was folded between two competent formations - the Knox above and the Rome below. The mechanics of this folding has been discussed fully in Sections 2.5.1.2.2 and 2.5.1.2.3. Age dating of wood particles from the terrace deposits at Watts Bar indicated the terraces are at least 32,400 + years BP. Although faults were seen in the Watts Bar bedrock, none were seen to intersect or offset the overlying terrace deposits. Therefore the minimum age of the faults is considered to be 32,400 years BP. Further discussion of these faults and age dating methods is in Section 2.5.3.2.

Weathering was seen to be structurally controlled in the bedrock at the Watts Bar Nuclear Plant. At the plant the weathering was seen to extend more deeply into the troughs of tight synclinal folds than into the rocks beneath anticlinal folds. This is because the weathering solutions can more easily penetrate down the bedding toward the bottom of the syncline.

As a result of removing approximately 40 feet of overburden in the general plant area, de-stressing of the freshly exposed shale caused some local raveling. Tests and experience both showed the raveling to be a function of de-stressing rather than air slaking and weathering processes normally associated with the degradation of shale.

The de-stressing process was restrained by requiring all horizontal surfaces to be covered with at least 4 inches of concrete within 48 hours after completion of excavation to final grade. In general, complete coverage of horizontal surfaces was accomplished within 8 hours after excavation to final grade. Cleanup before placement of concrete was done with hand labor and light air or water jets so as to not disturb the shale left in place. Vertical surfaces were covered within two weeks after exposure with a cast-in-place concrete wall.

No problems will be encountered by man's activities such as mining or fluid extraction or injection, since none of these activities, except the removal of minor amounts of ground water, have been or will be carried out beneath the structures. Even if ground mater were extracted near the plant, it would not affect the geologic competence of the foundation because we are dealing with a highly consolidated hard rock condition. Further discussions of fluid extraction or injection and mineral extraction have been discussed in Section 2.5.1.1.6 and will not be repeated here.

For soil properties see Section 2.5.4.2.1.

2.5.1.2.10 Groundwater

Groundwater occurs under water-table conditions in very small openings along fractures and bedding planes in the Conasauga Shale. The porosity appears to be very low, probably not in excess of 0.01%. The thickness of the water-bearing zone in bedrock is less than 50 feet.

Water occurs in the alluvial material overlying bedrock in pore spaces between particles. Because of small grain size and poor sorting, this material is poorly water-bearing. Average saturated thickness is about 23 feet.

The average depth to the water table in the plant area in the period of high water levels is 11 feet. Groundwater discharge is to Chickamauga Lake or to Yellow Creek and its tributaries.

2.5.1.2.11 Geophysical Surveys

A Regional Bouguer Gravity Anomaly Map and a Regional Magnetic Map are presented as Figures 2.5-5 and 2.5-6, respectively. No significant site-related anomalies are indicated on either map.

Dynamic studies were initiated at the Watts Bar site to obtain data for computation of elastic moduli for earthquake design criteria. Laboratory tests were deemed inadequate because of a wide variation in the attitude and physical character of the rock. Tests consisted of the following:

(1) Continuous logging procedures for seven holes located in the reactor, turbine and auxiliary buildings (Figure 2.5-70).

(2) Cross hole shooting at one location within each of the reactor foundations (Figure 2.5-109).

The seven test hole series was done by the Birdwell Division of Seismograph Service Corporation. Using their standard 3-D velocity, gamma-gamma density, and caliper logging tools to make velocity, bulk density, and hole diameter measurements. Cross hole surveys were made by TVA using a Hall-Sears MP-4 type geophone coupled to a Geo Space GT-2B refraction seismograph and Model DRO-6-32 recording oscillograph.

Results of Dynamic Testing Program

The Birdwell 3-D velocity system produced excellent records, but they were unable to differentiate shear wave arrival times. In the folded shale strata, the shear wave velocity is probably very close to the compressional wave velocity of the water filling the holes, thus masking the shear arrival times. To offset the lack of true shear velocities, Birdwell applied an empirical formula using compressional velocities and bulk densities to derive shear velocities. The Birdwell survey gives detailed information on the elastic properties of the foundation rock. This information is presented in the form of strip logs and computer printouts in Figures 2.5-71 through 2.5-108. No attempt was made to record shear velocities in the cross hole studies; however, compressional velocities were very close those computed by Birdwell. Cross hole data are presented in Figure 2.5-109.

2.5.1.2.12 Soil and Rock Properties

Uphole logs were run on seven of the core holes and the results are provided on the individual geologic logs and printouts (Figures 2.5-71 through 2.5-108). Dynamic moduli were determined using the cross-hole technique and the results are provided as Figure 2.5-109.

Additional rock properties as well as soil properties are discussed in Section 2.5.4.2.

2.5.2 Vibratory Ground Motion

2.5.2.1 Seismicity

The evaluation of the earthquake hazard at the Watts Bar site involves consideration of the seismic history not only of the immediate area but of the entire southeast and adjacent areas. The most seismically active areas in the region under consideration are:

(1) The Upper Mississippi Embayment, Especially the New Madrid Region of Arkansas, Kentucky, Missouri, and Tennessee

This region has been active seismically since the appearance of Europeans and very probably long before that. A few great earthquakes and thousands of light to moderately strong shocks have been centered in the Upper Mississippi Embayment area[95,120]. Light to moderate shocks are still occurring at a frequency of a few per year in this zone. The 1811-1812 epicentral area is 285 miles west-northwest of the Watts Bar site and the only site effects have been and would be the attenuated effects from major events in the New Madrid area.

(2) The Lower Wabash Valley of Illinois and Indiana

This area has been the focus of several moderately strong historic earthquakes. Here again, the only effects at the Watts Bar site from future shocks epicentered in this area would be greatly attenuated as the mouth of the Wabash River is 235 miles to the northwest.

(3) South Carolina Area

There is an apparent zone of seismic activity extending from Charleston, South Carolina, on the southeast northwestward across the Piedmont[1]. One of the country's greatest earthquakes occurred near Charleston in 1886[21]. Minor to moderate shocks have occurred subsequently along this alignment. Charleston is 285 miles southeast of the Watts Bar site and effects from any major shocks in this zone would be attenuated in the site area.

(4) Southern Appalachian Tectonic Province

This zone extends from central Virginia to central Alabama from the western edge of the Piedmont across the Cumberland Plateau. The Watts Bar site lies within this province which is a region of continuing minor earthquake activity. Light to moderate shocks occur at an average frequency of one or two per year. The activity is not uniform, as periods of several shocks per year are followed by periods of no perceptible shocks.

In addition to these four major areas, shocks of light to moderate intensity from widely scattered epicenters have occurred at other localities in the southeastern United States.

The destructive and near destructive earthquakes in the United States through 1972 (as compiled by the U.S. Geological Survey) are shown on Figure 2.5-146.

Figures 2.5-161 through 2.5-184 list in chronological order seismic events that have occurred within an approximate 250-mile radius of the Watts Bar site, as well as larger events beyond the 250-mile radius which have affected the site.

Of the events listed in the tabulation, only 40 are known, or can reasonably be inferred to have been felt at the Watts Bar site, and five of these events were major shocks at distances of over 250 miles from the site.

An annotated list of the earthquakes which have affected the Watts Bar site follows:

December 16, 1811 36.6°N - 89.8°W

January 23, 1812 36.6°N - 89.8°W

February 7, 1812 36.6°N - 89.8°W

These were the strongest shocks of the great series of earthquakes of 1811-1812 centered in the Mississippi Valley and known collectively as the New Madrid earthquake. This series consisted of thousands of individual shocks and the three strongest have had an intensity of MM XII assigned to their epicentral areas, and were felt over an area of about 2,000,000 square miles. According to isoseismal maps (Figures 2.5-147 and 2.5-148) the strongest shocks of this series resulted in intensities of MM VI in the Watts Bar area.

January 4, 1843 35.2°N - 90°W

A severe earthquake centered in the Mississippi Valley was felt over some 400,000 square miles in a 12-state area. Chimneys were thrown down in Memphis, Nashville, and St. Louis. The intensity was perhaps as high as MM X in the epicentral area. This shock was perceptibly felt over the entire Tennessee Valley and may have had an intensity as high as MM IV or V in the Watts Bar area.

August 31, 1861 36.6°N - 78.5°W

An earthquake with an epicentral intensity of MM VI near Wilkesboro, North Carolina, affected an area of 300,000 miles[49]. The intensity at the Watts Bar site is from the felt area map (Figure 2.5-149) to have been MM II - III.

August 31, 1886 32.9°N - 80.0°W

The great Charleston, South Carolina, earthquake was felt over the entire eastern United States. Its maximum intensity in the epicentral area was MM X, but in the Watts Bar area it was MM VI (Reference 21 and Figure 2.5-150).

October 31, 1895 37.0°N - 89.4°W

A strong earthquake centered at Charleston, Missouri, affected an area of 1,000,000 square miles in 23 states. It threw down chimneys and damaged buildings at various places in the Mississippi Valley, including Memphis, Tennessee. The earthquake was felt over the entire Tennessee Valley, but it was of low intensity in eastern Tennessee.

May 31, 1897 37.3°N - 80.7°W

The Giles County, Virginia, quake of May 31, 1897, is reported to have been the maximum event to have occurred in the Southern Appalachian Tectonic Province. Hopper and Bollinger[49] indicate that the felt area was approximately 280,000 square miles. They evaluate the intensity as having been MM VII to MM VIII. Earlier sources, as summarized in the 1965 edition of "Earthquake History of the United States, Part 1" by R. A. Eppley[22], had assigned a maximum intensity of MM VIII to this event. In the 1973 edition of "Earthquake History of the United States" by Coffman and von Hake[14], the maximum intensity has been downrated to MM VII. This was done apparently on the size of the felt area, which more nearly approximates the felt area to be expected in the eastern United States from a MM VII event than from a MM VIII event.

The isoseismal map (Figure 2.5-149) indicates that the Watts Bar site lies near the MM IV - MM V boundary so the intensity experienced at the site was either a low MM V or a high MM IV.

May 29, 1902 35.1°N - 85.3°W

A "strong shock" (intensity V) shook houses and awakened sleepers in Chattanooga[22,87].

October 18, 1902 35.0°N - 85.3°W

A moderate shock affected some 1,500 square miles in Georgia and Tennessee. It was felt from Dalton to Chattanooga. The maximum intensity was IV-V, but is not known to have been felt as far to the northeast as the Watts Bar plant site[22,87,131].

March 4, 1904 35.7°N - 83.5°W

The epicenter of this earthquake was between Maryville and Sevierville, but the disturbance was felt along the mountain front over a distance of 90 to 100 miles. The shock affected an area of about 5,000 square miles, but the intensity was nowhere above V and over much of the felt area it was much lower[22,87,131].

March 28, 1913 36.2°N - 83.7°W

This event with an epicentral intensity of MM VII occurred a few miles northeast of Knoxville, Tennessee. Although it was of moderately strong intensity, it only affected an area from 2000 square miles[87] to 2,700 square miles[1]. It is probable that the intensity at Watts Bar was MM III or less.

April 17, 1913 35.3°N - 84.2°W

This moderately strong earthquake was felt over an area of about 3500 square miles in eastern Tennessee, western North Carolina, northern Georgia, and northwestern South Carolina. The intensity was higher (V-VI) along the major axis of the affected area between Ducktown and Kiser. As shown by the map (Figure 2.5-151), the earthquake was not felt in the Watts Bar area, but it was felt some miles away[20,22,87].

May 2, 1913 35.5°N - 84.4°W

A light shock of several seconds duration was felt near Madisonville, Tennessee. This shock, intensity III, was centered approximately 35 miles from the plant site[114,131].

January 23, 1914 35.6°N - 85.5°W

A sharp local shock (V) was felt at Niota and Sweetwater, some 25 miles from the plant site[22,76,114,131].

February 21, 1916 35-5°N - 82.5°W

This strong earthquake, intensity VII, was centered in the mountains of western North Carolina. It affected an area of 500,000 square miles in the Carolinas, Georgia, Tennessee, Alabama, Kentucky, and Virginia. It was felt over nearly all of Tennessee, but was most severe in the mountains of eastern Tennessee. Chimneys were damaged at Sevierville and plaster was shaken from walls at Bristol, Morristown, and Knoxville. At Memphis, there was considerable motion in the higher stories of buildings. The earthquake affected the Watts Bar area at intensities between III and IV (Figure 2.5-152)[22,51,124].

October 18, 1916 33.5°N - 86.2°W

A strong earthquake centered near Easonville, Alabama, was felt over an area of 100,000 square miles in a seven-state area. About two-thirds of Tennessee was affected by this earthquake, but there was no damage in the state. The disturbance was felt strongly at Chattanooga, Nashville, Waynesboro, Carthage, Sparta, McMinnville, Lewisburg, and other points in central Tennessee. A light shock was noticed in Knoxville and Clinton. At the Watts Bar plant site, the intensity was not more than III (Figure 2.5-153).

June 21, 1918 36.1°N - 84.1°W

Centered near Lenoir City, this moderate shock (IV-V) affected an area of 3,000 square miles. The epicenter was about 30 miles from the Watts Bar area and probably had an intensity not exceeding III at the Watts Bar site[22,76,87,131].

December 24, 1920 36°N - 85°W

A moderately strong shock was felt at a number of localities in eastern Tennessee including Rockwood, Glen Alice, Spring City, Harriman, Decatur, and Crossville. Many sleepers were awakened and the entire village of Glen Alice was aroused. This earthquake, with a maximum intensity of V, was centered about 25 miles from the Watts Bar plant site and is not known to have affected the site area[22,114,131].

December 15, 1921 35.8 °N - 84.6°W

An earthquake of "considerable intensity" was felt along the western portion of the Appalachian Valley from Kingston and Rockwood to Decatur and Dayton and as far eastward as Athens. The maximum intensity was V, with the epicenter located approximately 15 miles northeast of Watts Bar. This shock probably resulted in the highest intensity (IV-V) at the site from a nearby earthquake[87,88,114,131].

October 20, 1924 35.0°N - 82.6°W

A strong earthquake (V-VI) centered in Pickens County, South Carolina, was felt over 56,000 square miles in the Carolinas, Georgia, Tennessee, Virginia, and Florida. Although buildings were strongly shaken in the epicentral area, there was little damage. The intensity in eastern Tennessee was nowhere greater than III. At the Watts Bar plant site, the intensity was less than II (Figure 2.5-154)[22,87,93,115].

October 8, 1927 35.0°N - 85.3°W

A moderately strong earthquake was felt in all parts of Chattanooga and suburban areas, including North Chattanooga, East Ridge, Lookout Mountain, Signal Mountain, St. Elmo, and Red Bank. The shock was felt in small and large buildings. Lights trembled and loose objects were disturbed. Other mild shocks were reported within a few hours following this shock. The shock is not known to have been felt in the Watts Bar area[87,115].

November 2, 1928 35.8°N - 82.8°W

A strong earthquake centered in the mountains of Madison County, North Carolina, was felt over an area of 40,000 square miles in a six-state area. The maximum intensity was VII, but in Tennessee the intensity diminished from VI along the state line to extinction somewhere in central Tennessee. At the Watts Bar plant site, the intensity was less than III[22,91,127].

August 30, 1930 35.9°N - 84.4°W

This earthquake was felt at Kingston, Lenoir City, Lawnville, Oliver Springs, and other points west and southwest of Knoxville. The maximum intensity was V. The epicenter was located 30 miles northeast of Watts Bar and probably the intensity did not exceed III to IV at the site[114,127].

March 31, 1938 35.6°N - 83.6°W

An earthquake centered in the mountains in the Little Tennessee Basin was widely felt in Tennessee and North Carolina. In Tennessee it was felt at Copperhill, Parksville, Knoxville, and Sweetwater where the intensities ranged from III to I. The shock is not known to have affected any part of Tennessee west of Sweetwater[87,127].

October 19, 1940 35.0°N - 85.0°W

An earthquake which shook houses and rattled loose objects awoke thousands of sleepers in Chattanooga. It affected some 1,100 square miles in Tennessee and Georgia. It was felt as far north as Charleston and Birchwood but at very low intensities (Figure 2.5-155). It was not reported to have been felt in the Watts Bar area[81,114,127].

September 8, 1941 35.0°N - 85.3°W

An earthquake was felt throughout Chattanooga and as far west as Jasper. It was especially strong in the Lookout Mountain area where walls vibrated, loose objects rattled, and glassware was broken. This earthquake is not known to have been felt upstream from Chattanooga[114,127].

June 14, 1945 35.0°N - 84.5°W

This shock, centered near Cleveland, Tennessee, where the intensity was V, was felt over an area of 4,000 square miles in southeastern Tennessee and northwestern Georgia. It was felt northeastward to Knoxville, southwestward to Chattanooga, and southeastward to Blue Ridge, Georgia. The felt area of this shock was never mapped, but the shock may have affected the Watts Bar area at an intensity of III or less[22,127].

April 6, 1946 35.2°N - 84.9°W

Another light shock was felt at Cleveland, Tennessee. This shock was not reported felt outside of the city[127].

December 27, 1947 35.0°N - 85.3°W

A light earthquake (IV) felt in Chattanooga, Tennessee, and Fort Oglethorpe, Rossville, Ringgold, and Boynton, Georgia, affected an area of 300 miles. It was centered east of the Missionary Ridge fault, where houses shook, loose objects rattled and piano wires popped. The shock is not known to have been felt any nearer to Watts Bar than Chattanooga[114,127].

January 22, 1954 35.3°N - 84.4°W

A light earthquake was felt over much of McMinn County from Athens to Etowah and Englewood. It is not known to have been felt outside of the county[87,127].

September 7, 1956 35.5°N - 84.0°W

A quake of epicentral intensity MM VI centered between Knoxville, Tennessee, and Middlesboro, Kentucky, was felt over an area estimated to have been from 8,000 square miles[1] to 10,000 square miles[9] in Tennessee, Kentucky, North Carolina, and Virginia. The Watts Bar site lies outside the limits of the felt area.

June 23, 1957 35°54'N - 84°14'W

A light local earthquake was felt in western Knox County and nearby sections of Anderson and Loudon Counties. At Dixie Lee Junction and in neighboring communities, people were awakened by the 'jumping' of houses and the rattling of loose objects[114,127].

June 12, 1959 35°21'N - 84°20'W

A light earthquake was felt over an area of 900 square miles in eastern Tennessee and western North Carolina. It was most strongly felt at Tellico Plains and Mount Vernon where an intensity of IV was attained[127].

April 15, 1960 35.8°N - 83.9°W

A shock of intensity V, centered near Knoxville, Tennessee, was felt over a 1,300 square mile area. In the vicinity of Watts Bar the intensity was not more than II to III[87].

August 24, 1966 35.9°N - 83.9°W

There is no record that this shock of intensity IV, centered near Knoxville, Tennessee, was felt in the Watts Bar area[87].

November 9, 1968 38.0°N - 88.5°W

This earthquake, centered in southern Illinois, with an epicentral intensity of VII was felt over a 400,000 square mile area in 23 states, including Tennessee, and in Canada. In the Watts Bar area it had an approximate intensity between II and III (Figure 2.5-156)[26,87].

July 13, 1969 36.1°N - 83.7°W

The epicenter of this intensity IV shock was located northeast of Knoxville, Tennessee. There is no record of this shock being felt in the Watts Bar area (Figure 2.5-157)[87].

November 19, 1969 37.4°N - 81.0°W

This intensity V shock, with its epicenter in southern West Virginia, was felt in Knoxville, Tennessee, with an intensity between II and III. There is no indication it was felt at the Watts Bar site (Figure 2.5-158)[4].

November 30, 1973 35.8°N - 84.0°W

This shock affected a relatively small area in and around Maryville, Tennessee with an intensity of MM VI; however, the felt area was approximately 25,000 square miles in Tennessee, North Carolina, Georgia, South Carolina, Virginia, and Kentucky[5,6,89]. The isoseismal map (Figure 2.5-159) indicates that the intensity at the Watts Bar site did not exceed MM IV and may have been less. This event is of interest in that after-shock measurements indicate the possibility of a hypocentral depth of less than four kilometers. For other shocks in the region hypocenters have been reported at depths of over five kilometers, indicating that the energy release normally occurred in basement rocks (+15,000 foot depth) below the sedimentary cover.

Light shocks have been centered in the Chattanooga area, near Cleveland, and in the Rockwood-Spring City area near the site. Great distant earthquakes have affected the area with intensities equal or greater than the maximum intensities of the several shocks centered within 50 or 60 miles of the site. Of the 35 earthquakes identified in the foregoing annotated list, only 11 are positively known to have been felt at Watts Bar. Of these, three were centered in the Mississippi Valley, one at Charleston, South Carolina, one in Alabama, one in Illinois, and five at various centers in East Tennessee, Virginia, and western North Carolina. In addition to these, it is probable that a few other shocks might have affected the area at very low intensities.

Presented as Figures 2.5-161 through 2.5-184 are four tabulations of earthquake data based on epicentral intensities and geodetic coordinates. These were prepared at TVA's request by the Health Physics Division of the Oak Ridge National Laboratory from computerized data assembled for the recent report entitled Seismic History and Seismicity of the Southeastern United States (W. C. McClain and 0. B. Myers, Report ORNL-4582, UC-51, June 1970).

The first tabulation lists all historic earthquakes in an area encompassing slightly more than a 250 mile radius from the Watts Bar site.

The second tabulation lists all historic earthquakes with a Richter scale magnitude greater than 4.3 (MMV+) in the same area.

The third tabulation lists all historic earthquakes with a Richter scale magnitude greater than 4.3 (MMV+) in the southeastern United States.

The fourth tabulation lists all historic earthquakes with a Richter scale magnitude greater than 6.3 (MMVIII+) in the southeastern United States.

On Figure 2.5-145 epicenters of all historic quakes within 200 miles of the Watts Bar site are plotted. No earthquake induced geologic hazards have been reported within the plant region.

2.5.2.2 Geologic Structures and Tectonic Activity

'Seismic and Geologic Siting Criteria for Nuclear Power Plants' (10 CFR Part 100 - Appendix A) states in Section IV (a) (6) that the tectonics of a specified site shall be identified with either (1) a 'tectonic structure' or if reasonable correlation cannot be established with a 'tectonic structure', the site shall be identified with (2) a 'tectonic province.' In Sections III (h) and (i) a tectonic structure and a tectonic province are defined. A 'tectonic structure' is defined as a large scale dislocation or distortion within the earth's crust whose extent is measured in miles. A 'tectonic province' is defined as a region of the North American continent characterized by a uniformity of the geologic structures contained therein.

In recognition of the fact that sites in the southern Appalachians cannot reasonably be tied to any one 'tectonic structure,' AEC in evaluation of the Sequoyah Nuclear Plant defined a 'Southern Appalachian Tectonic Province.' This province is bounded on the east by the western margin of the Piedmont Province; on the west by the western limits of the Cumberland Plateau; on the south by the overlap of the Gulf Coastal Plain Province; and on the north by the re-entrant in the Valley and Ridge Province near Roanoke, Virginia. The limits of the province are shown on Figure 2.5-145. Under this concept, maximum accelerations at the site shall be determined by assuming that the largest historic earthquake known in the province occurred adjacent to the site. For the Watts Bar site, this earthquake would be the May 31, 1897, quake in Giles County, Virginia, which had a reported epicentral intensity of MM VIII.

Regional geologic and tectonic maps are submitted as Figures 2.5-2 and 2.5-4, respectively. An earthquake epicenter map of the region is presented as Figure 2.5-145. Although no capable faults exist in the region, regional and subregional fault maps are submitted as Figures 2.5-7 and 2.5-8, respectively. Discussions of the regional geology and regional tectonics were provided previously in Sections 2.5.1.1.2 and 2.5.1.1.6, respectively.

2.5.2.3 Correlation of Earthquake Activity With Geologic Structures to Tectonic Provinces

There is no known correlation between earthquakes which have occurred in the region and any surficial tectonic structures. As discussed in Section 2.5.2.2, the Watts Bar site lies in the Southern-Appalachian Tectonic Province, as defined in the PSAR for the Sequoyah Nuclear Plant.

The maximum historic felt intensity at the site (MM VI) was derived from the 1811-1812 New Madrid events, the 1886 Charleston, South Carolina, shock, and MM IV - V from the 1897 Giles County, Virginia, quake. These epicenters were, respectively, 285, 285, and 255 miles from the site which indicates that the most severe site intensities have resulted from major earthquakes centered at distant points rather than from local shocks.

2.5.2.4 Maximum Earthquake Potential

The maximum historic earthquake reported in this province was of intensity MM VIII and occurred in Giles County, Virginia, in 1897. Although this earthquake is listed as an intensity MM VIII, there is considerable evidence that it should be reevaluated as an intensity MM VII. This is well documented in the McGuire Nuclear Station PSAR. Even though this earthquake occurred 255 miles northeast of Watts Bar, this intensity, MM VIII, is assumed to occur adjacent to the site for the purpose of defining the Safe Shutdown Earthquake (SSE).

Watts Bar Nuclear Plant is located in the same province as TVA's Sequoyah Nuclear Plant, and the Sequoyah PSAR and FSAR describe in detail some selection procedures for determining maximum ground acceleration values. Briefly, empirical relationships by Gutenberg and Richter, Hershberger, Kanai-Kawasumi, Wiggins, and Blume were used to estimate the maximum ground acceleration.

The relationships by Kansi-Kawasumi, Wiggins, and Blume result in very small ground accelerations. The Gutenberg-Richter and Hershberger relationships were developed from accelerograms obtained from practically all overburden sites. The Gutenberg-Richter relationship is felt to be conservative for bedrock since it, in itself, is conservative among other intensity-acceleration relationships. Using the Gutenberg-Richter relationship for a ground motion of intensity MM VIII resulted in an acceleration of 0.14 g. If one assumes that ground surface accelerations are greater than bedrock accelerations by a factor of approximately 1-1/2, this factor gives 0.093 g for bedrock accelerations.

Similar reasoning was applied to the selection of a maximum acceleration level for TVA's Sequoyah Nuclear Plant. The empirical relationships mentioned above indicated that a maximum acceleration level of 0.14 g for the SSE would be conservative. The Sequoyah FSAR (Section 2.5.2.4) contains a summary of a meeting held on November 13, 1969, between DRL staff members, AEC structural and geological-seismological consultants for Sequoyah, and TVA. The purpose of this meeting was to discuss earthquake design criteria for Sequoyah. In that meeting AEC's consultants were of the opinion that the maximum top of rock acceleration should be 0.18 g for the SSE. Accordingly, Sequoyah was designed for a maximum horizontal acceleration of 0.18 g and a maximum vertical ground acceleration of 0.12g.

Both Sequoyah and Watts Bar Nuclear Plants are located in the Southern Appalachian Tectonic Province approximately 40 to 50 miles apart. The Giles County, Virginia, earthquake of 1897, rated as an MMVIII, was assumed to occur at each of the respective sites for purposes of defining the SSE.

Therefore, in view of the agreement reached in the meeting discussed previously and the factors discussed in the previous paragraph, the Watts Bar Nuclear Plant has been designed for a maximum horizontal top-of-rock acceleration of 0.18 g for the SSE and a maximum vertical top-of-rock acceleration of 0.12 g.

Figures 2.5-236a and 2.5-236b show the site seismic design response spectra for the OBE and SSE, respectively, for all damping ratios used in the design of rock-supported structures. Vertical design response spectra are two-thirds (2/3) of the corresponding horizontal spectra.

The original seismic design basis for Watts Bar Nuclear Plant is 0.18 g as discussed above. However, in the course of their review for the operating license, NRC requested additional information concerning the seismic design basis. This culminated in the development of a site specific response spectrum. This spectrum represents the

84th percentile of 13 actual earthquake recordings and has a peak acceleration of 0.22 g. This site specific spectrum was used for evaluation of present designs and not as a design basis. The development of the site specific spectrum is presented in the following reports.

(1) Justification of the Seismic Design Criteria Used for the Sequoyah, Watts Bar, and Bellefonte Nuclear Plants - Phase I, TVA, April 1978.

(2) Justification of the Seismic Design Criteria Used for the Sequoyah, Watts Bar, and Bellefonte Nuclear Plants - Phase II, TVA, August 1978.

(3) Prediction of strong motions for Eastern North America on the Basis of Magnitude, Weston Geophysical Report for TVA, August 1978.

(4) Earthquake Ground Motion Study in the Vicinity of the Sequoyah Nuclear Power Plant, Weston Geophysical Report for TVA, February 1979.

(5) Justification of the Seismic Design Criteria Used for the Sequoyah, Watts Bar, and Bellefonte Nuclear Plants - Phase II - Responses to NRC Questions 1 through 6, TVA, June 1979.

In response to the NRC staff's review of the liquefaction potential of the soils along the ERCW pipeline and 1E conduit alignments (see Section 2.5.4.8), a site-specific study of the top-of-ground motion for the Watts Bar Nuclear Plant was made. The results of this study are contained in a report entitled, 'Site-Specific Top-of-Ground Motions for ERCW Pipeline' dated September 23, 1983. The result of this study was an 84th percentile peak ground acceleration of 0.22 g. This peak acceleration was used for the liquefaction evaluation in Section 2.5.4.8. As a result of NRC concerns about the above report, the report was resubmitted and supplemented with responses to the NRC concerns[158]. The result of this resubmitted report was an 84th percentile peak ground acceleration of 0.26 g.

During the review of these reports, the NRC staff specified that a peak ground acceleration of 0.40 g should be used. Therefore 0.40 g was used for the peak top-of-ground horizontal acceleration for the liquefaction evaluation along the ERCW pipeline and 1E conduit alignments.

In response to various seismic analysis and geotechnical concerns the original seismic design basis was supplemented with additional criteria. These criteria are termed "Evaluation Seismic Criteria" and "New Design/Modification Seismic Criteria."

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

As stated previously in Section 2.5.2.4, the design response spectrum is not site dependent. Soil properties for bedrock and soil strata under the plant site are detailed in Section 2.5.4.

2.5.2.6 Safe Shutdown Earthquake

The plant was designed for an SSE with a maximum rock acceleration of 0.18 g horizontally and 0.12 g vertically. Response spectra for horizontal motion are shown in Figure 2.5-236b.

2.5.2.7 Operating Basis Earthquake

Examination of the recorded seismic history of the Southern Appalachian Tectonic Province reveals that there have been six earthquakes of intensity MM VII, or MM VI - VII. As discussed in Section 2.5.2.1, the maximum intensity felt at the site from earthquakes outside the province was probably MM V or MM VI. Therefore, it is reasonable to expect an MM VII during the operating life of the plant.

Assuming the operating basis earthquake (OBE) is an MM VII and using the same empirical relationships discussed in Section 2.5.2.4, the accelerations are less than one-half those of the safe shutdown earthquake (SSE). According to 10 CFR 100, Appendix A, the OBE must be at least one half of the SSE. Therefore, the maximum horizontal and vertical ground accelerations for the OBE are 0.09 g and 0.06 g, respectively. The response spectra for the OBE are one-half of the response spectra for the SSE and are shown in Figure 2.5-236a.

2.5.3 Surface Faulting

2.5.3.1 Geologic Conditions of the Site

The lithologic, stratigraphic, and structural geologic conditions of the site and the area surrounding the site have been discussed in Sections 2.5.1.2.2 through 2.5.1.2.5. A regional geologic map is provided as Figure 2.5-1. A subregional geologic section is provided as Figure 2.5-3. A geologic map of the plant area and a geologic section through the plant area are provided as Figures 2.5-9, 2.5-10 and 2.5-11. Regional and subregional fault maps are provided as Figures 2.5-7 and 2.5-8, respectively.

2.5.3.2 Evidence of Fault Offset

Section 2.8.5, Geologic Structure, of the Watts Bar Nuclear Plant Preliminary Safety Analysis Report stated that the foundation strata were 'folded, contorted, crumpled, sheared, and broken by small faults.' These structural complexities were demonstrated to be confined to the Middle Cambrian Conasauga Formation, a weaker shale and limestone unit lying between more massive sandstones of the Rome Formation below, and massive overlying dolomite and limestone of the Knox Formation.

When foundation excavation and preparation for the plant began in July, 1973, daily geologic studies of the foundation areas were undertaken. These consisted of geologic mapping of the foundation, including floors and walls of all cuts, photographing the geologic conditions exposed, and preparing geologic plans and sections. Initial excavation began in the Turbine Building area and progressed to the Reactor Building area.

In mid-December 1973, when the foundation for the Reactor Building was partially excavated, AEC regulatory staff geologists were invited to visit the site to inspect the complex foundation conditions including the frequency and magnitude of faulting and shearing. On December 20, 1973, Mr. James Skrove of the AEC staff visited the site and inspected the foundation areas exposed.

On January 25, 1974, a request was received by TVA from the AEC Directorate of Licensing requesting an interim report documenting the geology of the foundation as it existed in December 1973. Because much of the foundation rock in the reactor areas was not exposed in December, 1973, this report was delayed until foundation excavation was completed so that the geologic conditions of the entire foundation area could be evaluated. The last major segment of the foundation was exposed in mid-April 1974 and the report was submitted to NRC shortly thereafter.

In discussing the length, attitude, and senses of movements of the faults exposed in the excavations, the larger thrust faults shown on the plan view geologic maps (Figures 2.5-110 and 2.5-111) extend completely across the powerhouse foundation in a N35-40° E direction and dip 30-45° southeast. One fault trace was mapped over a horizontal distance of 450 feet across the powerhouse foundation, and others were projected across distances of 250 feet to 300 feet. Because the traces of these faults were indistinguishable on horizontal surfaces where they paralleled bedding in the shale, their continuity was determined by following them downward along a vertical excavation surface and projecting a bearing of N35-40° E across the intervening horizontal surface into the next vertical cut where the faults were normally found to recur. This fault detection technique is illustrated in Figure 2.5-123 where approximately 175 feet of fault trace can be observed cutting across the Auxiliary Building foundation, into the vertical wall at an approximate location of A5-9 feet and 6 feet south of the east-west reactor centerline, across the southeast of Unit 1 perimeter, and into the vertical cut wall in the background. A closeup of the inset area of Figure 2.5-123 is shown in Figure 2.5-124.

Although the faults at the site cut entirely across the powerhouse foundation and are expected to continue northeastward and southwestward for an unknown distance, these faults are confined to the Conasauga formation and do not intersect or displace any other stratigraphic formation. They have neither the horizontal nor stratigraphic displacement of the Kingston fault, located one mile northwest of the site, or of the Chattanooga and Whiteoak Mountain fault systems farther to the northwest and southeast, respectively. As shown in the geologic section along A-A' in Figure 2.5-11, the weaker Conasauga shale was confined, during thrust movement, between the more massive Rome sandstone below, and the Knox limestones and dolomites above. As a result of the tremendous disparity in strength, the Conasauga was intensely contorted, crumpled, and faulted. These characteristics were predicted and outlined in the Preliminary Safety Analysis Report of the Watts Bar Nuclear Plant and were known from the data collected during foundation investigations of the Watts Bar Steam Plant and Watts Bar Dam 3/4 mile to the northeast. The same strike belt of Conasauga Formation provides the foundation for all of these projects.

The folded sequences at the site are all homaxial in that they, as well as the major Valley and Ridge fault slices, are related genetically to only one longitudinal axial direction. Superimposed or cross-folding or faulting with longitudinal axes oriented in directions other than that of a regional trend, N35-40° E, were not encountered. This, in conjunction with data obtained from regional geologic and topographic maps, is evidence of a northwestward sense of movement.

Recurrence of compressional surges throughout the late Paleozoic orogenic episode, causing the structural deformation at the site, is evident. The direction of impingement of the compressional forces, however, was inherently from the southeast throughout this period.

In regard to the geometric relations of the faults to folded and warped beds which were truncated by the faults, movement of adjacent rock units across fault planes was not always measurable as one primary displacement. Distributive faulting is predominant throughout the area in which movement took place along bedding planes and closely spaced fractures as well as along the thrust faults and shears. Where bedding plane movement occurred along 'bedding thrusts' in limestones and competent shale horizons, slickensides are oriented in a northwest direction.

Bedding thrusts and minor thrust faults located in areas devoid of thick limestone beds are generally characterized by finely ground shale fragments and thin grey plastic clay seams along the fault as shown in Figure 2.5-125. The absence of competent strata in these areas allowed considerable sliding along low angle fault traces, developing a thin gouge or clay seam.

Disparity of stratigraphic lineation and the presence of drag folds, normally found where stronger beds slipped past weaker ones, was commonplace throughout the site area. Due to frictional resistance along discordant fault planes, thin limestone beds flanked by shale were commonly warped and buckled near their truncated ends (at the fault trace) in the direction of the adjacent block movement. The confining but weaker shales permitted a considerable amount of flexural buckling of the limestone beds prior to rupture. (See Figures 2.5-126, and 2.5-127, 2.5-112, 2.5-113, and 2.5-114.)

Structural elements were considerably less complex in the southeastern and northwestern segments of the powerhouse foundation than in the belt of rock that extended generally N40 degrees E from the northwest corner of the Turbine Building foundation, diagonally across the center of the Control Building foundation, through the eastern segment of the Auxiliary Building and into the Unit 2 reactor foundation (see Figures 2.5-112, 2.5-113, 2.5-115, 2.5-116, 2.5-117, and 2.5-128). Although rock competency is measured on a relative basis, the thick limestone beds, which were found to predominate in this strike belt, were capable of transmitting the compressive forces much farther than the weaker shaly strata in the southeastern and northwestern segments of the powerhouse foundation. This resulted in massive folded sequences containing broadly undulating asymmetrical and recumbent folds which plunged both northeast and southwest along strike. Characteristically, when the region was subjected to compression, multitudes of folds of every possible size developed. The stronger strata were folded into broad synclines and anticlines, whereas weaker strata developed low angle thrusts drag folds and parasitic folds. The thin layers of weaker shale interlaminated between the stronger limestone units abound with parasitic folds. Digitations were also found along some of the larger folds, adding to the structural complexity of the foundation rock. The presence of these features made detailed mapping of the horizontal excavation surfaces extremely difficult. Detection of fault traces across these areas was almost impossible unless clay seams were present along the fault planes.

As a result of the distributive faulting, shale horizons were crumpled and displaced as the more massive limestone horizons absorbed the brunt of the compressive forces and moved to the northwest. Where the massive limestones formed large drag-folded horizontal or plunging anticlines and synclines, the adjoining weaker shale was squeezed into tightly compressed folds, commonly ground into thin platy fragments and extruded into the cores of the tight limestone drag folds.

Except where plainly evident in deep vertical cuts, actual displacement of rock units along the faults could not be measured because of the absence of stratigraphic change on either side of the fault, structural complexity, and by the absence of key horizons. The only detectable continuous horizon on which displacement could be measured was a broadly folded, faulted fivefoot-thick massive limestone which was encountered in the N line vertical wall excavation (Figure 2.5-113), and which continued along strike through the Q-4 vertical wall (Figure 2.5-115), into the S-4 vertical wall (Figure 2.5-116), and into the Unit 2 reactor cavity (Figure 2.5-117). Though folded and faulted along its entire length, measured displacement did not exceed 6-8 feet.

Displacements ranging from one to three feet were measured in several vertical wall cuts as shown in the geologic sections, Figures 2.5-115, 2.5-120, 2.5-126 and 2.5-129. As shown in Figure 2.5-126, the left block of the normal fault in Unit 1 west wall has moved down approximately one foot with respect to the right block. Scale can be approximated by the 6 x 6 mesh wire at the base of the wall. True fault gouge has not developed along the fault plane, but a thin zone of finely ground shale particles delineates the trace of the fault.

The normal fault shown in Figure 2.5-129 had a measurable displacement of 1.5 feet. Such faults, which dip northward, were rarely observed. The fault could not be traced into the Unit 1 cavity area, nor into the vertical cut northeast of the reactor area. It was therefore considered to be a minor fault related to adjustments associated with thrusting.

Displacement shows considerable variation throughout the plant site area but is considered to be minor in both extent and consequence. Although several low angle thrusts and shears show that considerable horizontal movement may have occurred (see geologic Section C-B, Figure 2.5-117), no extreme vertical displacements were observed in the foundation strata.

In summarizing the existing data in an attempt to more clearly understand the fault-fold geometrical relationships, it is concluded that the faulting and folding developed contemporaneously, over a considerable length of time. The presence or absence of thick limestone beds apparently is the dominant factor in determining the type of structure. Long, low angle thrusts and shears developed along bedding planes in predominantly shale areas. In areas of thick limestone sequences broad undulating folded anticlines and synclines developed with their associated higher angle thrusts and subsidiary normal faults. Compressional forces acting on thick limestone strata were dissipated in forming broad folds until the rupture limit was reached and faulting occurred. The interlaminated, brittle, weaker shales were pinched into tight folds within the broadly undulating limestone and then were sheared along with the limestone.

In regard to the relationship of the foundation faults to through-going major faults, the controlling feature of the geologic structure at the Watts Bar site is the Kingston fault. The trace of this fault lies along the northwest side of the prominent ridge approximately one mile northwest of the site area as shown on the geologic map of the plant area (Figures 2.5-9 and 2.5-10). The Kingston fault is one of the major overthrusts characteristic of the Valley and Ridge province. As shown on the geologic map and section (Figures 2.5-9 through 2.5-11), the fault dips to the southeast approximately 30° or more and is considered to be approximately 2,000 feet deep at the site. Along the fault, steeply dipping beds of the Rome Formation have been thrust over gently dipping strata of the Chickamauga Limestone.

The Kingston fault is only one of the several lengthy thrust faults that characterizes the geologic structure of the Appalachian Valley, which is a part of the Valley and Ridge physiographic province. The Kingston fault is essentially parallel to the Chattanooga fault which lies approximately five miles northwest of the plant site; the White Oak Mountain fault, approximately 3.9 miles southeast of the plant site; and the Copper Creek fault, approximately 6.3 miles southeast of the plant site. The structural deformation resulting in thrust fault development is attributed to the Allegheny orogeny which culminated at the end of the Paleozoic Era. It is postulated that these major tectonic structures have been inactive since the cessation of the orogenic movement. The duration of this orogenic epoch has not been precisely determined in the region since Pennsylvanian strata are the youngest rocks known to have been affected. Some deformation is thought to have continued after the initial development of the major faults because some of the thrust sheets are folded. This late structural development represents the final phase of the orogeny. The only undeformed mappable units in the region are the unconsolidated materials: alluvial deposits, including high-level terrace deposits as well as recent floodplain alluvium, and residuum that nearly everywhere mantles bedrock. The alluvium along the Tennessee River channel and its tributaries ranges in age from less than a decade at the top to several tens of thousands of years at the base. The coarse terrace gravel deposits are much older and are considered to have been deposited during the Pleistocene.

The intensely deformed character of the Conasauga shale upon which the Watts Bar plant is being built is a function of its lithology and of its structural history. The Conasauga Formation is overlain stratigraphically by 2,500 feet to 3,000 feet of massive dolomite and limestone of the Knox Group and is underlain by 800 to 1,200 feet of sandstone and shale of the Rome Formation. During the course of the Allegheny orogeny, these formations were thrust northwestward on the Kingston thrust sheet which overrode the underlying rocks for an undetermined distance. During the thrust movement, the Conasauga Formation was compressed between the two massive blocks of the competent underlying Rome Formation and the overlying Knox Group. As a result of the marked difference in competency, the Conasauga shale was folded, contorted, crumpled, sheared, and broken by small faults as evidenced in the Watts Bar foundation.

The minor faulting in the plant foundation is therefore directly related to the major through-going fault systems. As indicated in Sections 2.5.1.1.2, 2.5.1.1.4, and 2.5.1.1.6, investigations in the region have revealed that the several named faults are merely branches of a single, nearly flat, sole fault developed in a relatively less competent formation just above the crystalline basement.

In regard to the bedrock-terrace deposit relationship, an investigation was conducted throughout the plant site to document specific occurrences where faulting was found to be terminated by the overlying, blanketing terrace gravel deposit. Several occurrences were found as shown in Figures 2.5-132 through 2.5-136. In Figure 2.5-132, the fault plane is seen to displace the thin limestone beds in the bottom left corner, and is truncated by a thin iron oxide crust at the base of the overlying terrace gravel in the top right corner. Figure 2.5-134 and associated Figures 2.5-135 and 2.5-136 show an occurrence where a fault plane, which extended diagonally across the entire Auxiliary Building foundation, is delineated by a 1/4 inch blue-grey clay seam. It can be seen that the fault plane is truncated by the thin iron oxide crust at the base of the overlying terrace gravel deposit. The upper three feet of shale has weathered to saprolite which retains the relict shale structure. The 1/4 inch blue-grey plastic clay seam extended downward along the fault plane for the entire 30 foot exposure of the vertical cut as shown in Figure 2.5-135. Discoloration and saprolitization of the near-surface shale from a blue-grey to buff color is common at the site, and is caused by normal weathering processes.

In the base of the terrace gravel, at the interface with the saprolitic residuum, a thin zone of iron oxide concentration was commonly observed. The origin of this deep reddish-brown indurated crust of iron oxide is attributed to iron saturated water movement along the contact between the overlying terrace gravel and the saprolitic shale residuum. The terrace gravel is highly permeable, offering very little impedance to ground water percolation. At the base of the terrace gravel, however, the downward moving ground water is met with a considerable decrease in permeability and moves laterally. This results in the accumulation of the iron oxide crust, which in some places blankets the upper surface of the saprolitic material and conforms to the micro-topography. Because of hematite cementation, the crust becomes indurated.

Though found dispersed through the entire thickness of the terrace deposits (see Figure 2.5-137), crusts that truncate fault planes provide additional proof that no movement has taken place along the faults subsequent to their formation.

The terrace gravel was deposited over the site area when the ancestral Tennessee River was flowing at a much higher elevation in the past. Except where cut by present stream drainage patterns, the terrace deposits are represented by broad, flat to gently rolling topography between approximate elevations of 700-740 feet. The terrace deposits have been relatively well preserved on the inside curve of the meanders of the Tennessee River, but have been removed where the Tennessee River impinges on knobby topography, for example, the River Knobs area southwest of the plant site. North of Watts Bar Dam, normal pool elevation is approximately 741 feet, covering the terrace deposits.

The appearance, thickness, color, size, and depositional characteristics of the basal gravel differs considerably around the walls of the excavation at the plant site. In general, trends were observed in the basal gravel deposit from southeast to northwest as follows:

(1) General thinning of the gravel-bearing material varying from a thickness of 18-20 feet of gravel mixed with sand and silt at the southeast to approximately six feet of clean, coarse gravel in the northwest corner of the excavation.

(2) The thick sandy matrix, prevalent in the southeast plant area, decreases considerably northwestward. Thick interlaminated sand sequences dispersed throughout the terrace gravel in the southeast are absent in the northwest plant area.

(3) Iron oxide staining, predominant through the entire thickness of the gravel deposit in the southeastern extremities of the plant site, is virtually nonexistent in the northwest section, which has a high content of clean, white terrace gravel in a clean washed, sandy matrix. Though the iron oxide crust was not always found at the contact between the Conasauga shale and the overlying terrace gravel deposit, concentrations of hematite-stained gravel, sand, and clay particles were everywhere present, because of lateral water movement along the contact. The sparse development of deep hematite staining in the northwest segment of the foundation is possibly due to protection by the overlying thick impermeable slough clay deposit which is not present in the southeastern segment of the site area.

(4) Gravel size increased from pea-gravel (1/2-3/4 inch diameter) in lenses dispersed throughout the entire 18-20 foot thickness of the deposit in the southeast, to 8-10 inch diameter cobbles and smaller pebbles in the thinner six-foot terrace gravel deposit in the northwest corner of the excavation.

(5) Crossbedding and preserved remnants of ancient stream channels found in the southeastern segment of the plant site area were absent in the northwest.

In the northwest corner of the powerhouse foundation the basal gravel is sharply delineated both at the top and bottom. Above the gravel is a three-to-four-foot thick layer of dark grey to black slough clay containing organic fragments (see Figures 2.5-122 and 2.5-138). This sequence is absent in the extreme southeastern area of the plant site and in its place is 18-25 feet of iron-stained sands interspersed throughout with well-defined one-eighth to one foot thick pebble layers. The pebbles in these layers are mostly less than two to three inches in diameter.

The presence in the basal gravel of large (8-10 inch diameter) pebbles and cobbles at the northwest margin of the plant site area, compared to the much smaller pebbles and gravels at the southeast margin subtly suggests that the main course of the old Tennessee River was once located near the northwest margin. Although this concept implies a long period of time for the main river channel to migrate some 2,000 to 3,000 feet southeastward to its present course, such migration is not improbable. The present configuration of the river channel indicates that in this stretch of the river southeastward migration would be progressing even today were it not for the artificial conditions created by man in impounding Chickamauga Lake. The lake essentially creates an artificial base level, which impedes lateral erosion processes in the river channel.

The sharp contact between the bedrock and the basal gravel deposit, with the absence of any major thickness of silts and sands except where interspersed within the voids of the basal gravel, strengthens the concept of a migrating river channel. The scouring action of the bedload in the main river channel, according to this concept, provided the necessary abrasive action to erode the bedrock surface to its present elevation. The gravel was deposited on this surface as velocity decreased when the river continued its southeastward migration.

In late February 1974, partially carbonized wood fragments were found in the north face of the main plant excavation embedded in a dark, tough, organic clay overlying the basal gravel layer of the terrace deposits. The occurrence is shown graphically on Figure 2.5-122 and pictorially on 2.5-138. Approximately two kilograms of material were collected, divided into three samples, and submitted to the Radioisotope Laboratory, Dicar Corporation, Cleveland, Ohio for carbon 14 dating. To preclude any unintentional bias on the part of the laboratory, the samples were submitted 'blind.' That is, no data on geographic location or stratigraphic position were furnished.

Upon receipt of the samples in the laboratory they were examined under 20x magnification, the obvious debris removed, and each sample fragmented into pieces one centimeter by one centimeter. The material was then subjected to three cycles of saturation in 2.0 normal NaOH, filtration, and drying. The treated samples were then pyrolized and the $^{14}CO_2$ collected for radioactive counting.

The median age of all samples is 32,400 years BP with median deviations of +2150 and -3000 years. We feel that these data represent a minimum rather than a maximum age. The samples when collected were completely saturated to the point of being 'spongy.' The samples occurred at elevation 717.5 and groundwater observations in the immediate area indicated that the permanent water table, prior to excavation, stood at or above elevation 721 even during the dry months of August, September, and October. This indicates that the material had been continually saturated. Even though percolation rates through the tough, organic clay surrounding the samples must have been very low, any migration of water through and around the material would tend to minimize the apparent age. It is probable that the true age of these wood fragments is in excess of 35,000 years.

Potassium-Argon (K-Ar) Dating

Samples of the Conasauga Formation with a high glauconite content were collected at two different locations in the main plant excavation and were submitted to the USGS Branch of Isotope Geology for K-Ar dating. One sample was obtained from strata showing minimum structural deformation, while the other was obtained from strata in and immediately adjacent to one of the small faults or shears cutting across the foundation.

Though realizing that glauconite is not an ideal geochronometer, it was felt that any significant divergence in apparent ages of the two samples might indicate (1) evidence of sufficient temperature and pressure increase associated with the shearing and faulting to have 'reset', either wholly or in part, the K-Ar 'clock', or (2) evidence of geologically recent readjustment of the K-Ar 'clock' in the material from the sheared area.

The analyses were made by Dr. John D. Obradovich at the USGS laboratory in Denver, Colorado. The apparent ages determined were $387 + 8 \times 10^6$ years for the samples from 'undisturbed' strata and $389 + 8 \times 10^6$ years for the sample from the shear zone. The two ages are essentially the same, approximately 390×10^6 years, for both samples.

The age of sedimentation for the Conasauga Formation is approximately 530×10^6 years - upper Middle Cambrian. The radiometric age of 390×10^6 years is approximately 26% younger. As Dr. Obradovich points out, this anomaly may reflect partial but not total loss of argon 40 during the Allegheny approximately 250×10^6 years ago or could result from burial at a depth of 5,000 to 7,000 feet since the Paleozoic. Either of these alternatives is viable for the Watts Bar area. However, the radiometric age determination falls within the scatter of other K-Ar data of comparable age and a major tectonic event is not necessary to explain the deviation between the radiometric and true ages.

As Dr. Obradovich further points out, the agreement in ages between the 'undisturbed' and 'sheared' samples indicates that any effects related to shearing were insufficient to cause any significant loss of radiogenic argon. This indicates that the time-temperature regime associated with the shear zone in recent geologic times was not enough to reduce the apparent age of the glauconite in the sheared area as compared to the age of the material from the undisturbed area.

Although the precise time when the shearing took place could not be determined by these analyses, the data obtained provide additional support for the conclusions reached from the other investigations that the small faults and shears in the foundation of the Watts Bar Plant are ancient structures which do not adversely affect the suitability of the foundation.

In summary, the bedrock at the Watts Bar Nuclear Plant site was faulted and folded contemporaneously during an orogenic epoch which culminated in the late Paleozoic approximately 250 million years ago. The faults and shears originated primarily along bedding planes at low angles in the incompetent shales. In the zone of competent limestone beds the faults are more discordant, occurring within broad undulating folds.

Since the end of the orogeny, the bedrock and its' contained shears and faults have been subjected to reannealing pressures, recementation, surficial erosion and weathering from some unknown preexisting elevation to the elevation at which it was subjected to the abrasive scour by coarse terrace gravel. This probably occurred in Pleistocene time. The bedrock was subsequently buried by thick gravel, sand, and clay deposits. There is no present-day surficial expression of the subsurface faults at the site.

2.5.3.3 Earthquakes Associated With Capable Faults

There are no historically reported earthquakes that can be reasonably associated with surface faults, any parts of which are within 5 miles of the site. Furthermore, no earthquake-surface fault relationships have been determined anywhere within the region.

2.5.3.4 Investigations of Capable Faults

That faults of great linear extent are present in the Valley and Ridge Province of East Tennessee was recognized by the earliest geologists to study the area (Troost, 1837, 1840, 1841 and Safford, 1856, 1859, 1869). While they did not assign specific ages to the faulting, they pointed out that it occurred long before the cycle began that resulted in the 'Valley and Ridge' topography developed by differential erosion of harder and softer strata.

The second generation of geologic studies in the area was made in the late 1800s and early 1900s by geologists of the U.S. Geological Survey. This work resulted in a series of 18 geologic folios (Campbell, 1899; Hayes, 1894, 1894a, 1894b, 1895, 1895a, 1895b; Keith, 1895, 1896, 1896a, 1896b, 1901, 1903, 1904, 1905, 1905a, 1907, 1907a) each covering a 30-minute quadrangle. This series provided the first detailed mapping of the area and basically interpreted the relationship and extent of the various fault sheets. In all of these folios the point is stressed that the folding and faulting culminated at the end of the Carboniferous and subsequently the area has been subjected only to broad epierogenic warping. No mention or inference of geologically recent faulting is made in any of these folios. These investigations, in addition to the folios, produced other professional papers and reports (Hayes, 1891, 1892, 1895, 1899; Hayes and Campbell, 1894; Keith, 1896, 1902, 1902a, 1923; Willis, 1893). These as well presented the thesis that faulting had culminated in the Late Paleozoic and had not recurred subsequently.

The third generation of studies, which has resulted in a mass of detailed data began in the 1940s and has continued to the present. The scope of these studies has ranged from textbooks and USGS professional papers to master's theses submitted to various universities in the area. The more significant publications resulting from this work are Bridge, 1950; Colton, 1970; Cooper, 1961, 1964; Ferguson and Jewell, 1951; Dietz, 1972; Gwinn, 1964; Hack, 1965, 1966; Harris, 1965, 1970; King, 1949, 1950, 1955, 1964, 1964a, 1969; King and Ferguson, 1960; Milici, 1962, 1963, 1967, 1968, 1968a, 1970; Miller, 1962; Neuman and Nelson, 1965; Owens, 1970; Rodgers, 1949, 1950, 1953, 1953a, 1963, 1964, 1967, 1970; Stearns, 1954, 1955; Swingle, 1961; and Wilson and Stearns, 1958. In none of these publications is any active faulting since the Paleozoic described, implied, or inferred. A search of all available data has failed to disclose any report of post-Paleozoic movement along any fault in the Valley and Ridge Province.

The early geologists, Troost and Safford, recognized that faults existed in the Valley and Ridge Province in Tennessee, but made no inference as to their attitudes at depth. The first geologist to recognize that these faults possibly could flatten out with depth and also have been folded and faulted subsequent to initial formation was Keith (1902a, 1905a, 1907, 1907a, 1923, 1927). However, major credit for the derivation of the 'thin-skinned' tectonic theory as applied to the southern Appalachian area rests with Rodgers (1949, 1950, 1953, 1953a, 1963. 1964, 1967, 1970). Although most of the geologic community working in the area sided with Rodgers (Gwinn, 1964; Harris, 1965, 1970; King, 1950, 1955, 1964, 1964a, 1969; King and Ferguson, 1960; Milici, 1962, 1963, 1970; Miller, 1962; Stearns, 1954, 1955; Swingle, 1961; Wilson and Stearns, 1958) some were unconvinced (Cooper, 1961, 1964).

It was not until early 1974 that definitive evidence was released to support the 'thin-skinned' hypotheses. At that time, Geophysical Services Incorporated published an advertising brochure describing reflection seismic data they had available for sale. The example of a reflection profile used in their brochure was made along U.S. Highway 70 from near Kingston, Tennessee, to the vicinity of Knoxville, Tennessee. This profile essentially at right angles to the regional strike is reproduced in Figure 2.5-160.

The vertical scale of this profile is represented in seconds. This indicates the double travel time necessary for the shock wave to descend to the reflector and return to the surface. Assuming a wave velocity of 20,000 feet per second, the times indicated equate to depths in thousands of feet. The 'thin-skinned' tectonic structure of the upper strata, above the 1.5 second (15,000 foot) line, is clearly indicated. The depth of approximately 15,000 feet to basement strata in this area is confirmed by gravity and magnetic data[128].

The significance of the confirmation of 'thin-skinned' tectonics in the area in relation to the geologic and seismic considerations of the Watts Bar plant lies in the fact that data now exist to show the separation of faults cropping out at the surface from geologic structures in the basement at a depth of approximately 15,000 feet or 4.5 kilometers. This means that earthquakes with hypocenters at depths of five or more kilometers cannot be associated with faults cropping out at the surface even though the epicenter (surface projection of the hypocenter) falls on or near the trace of the fault.

During investigations for the Clinch River Breeder Reactor Plant[98], samples of faulted material were collected and radiometrically dated. These samples were from the Copper Creek fault and were collected about 15 miles west of Knoxville and about 30 miles from the Watts Bar Nuclear Plant. This fault is of the Whiteoak Mountain family. The results of these age determinations (280-290 + 10 million years) indicate the movement of these faults occurred during the Late Paleozoic[102,113,125]. Furthermore, as indicated in Section 2.5.3.2, the bedrock fault - terrace relationship dates and the glauconite dates at the site indicate no recent movements along the bedrock faults.

TVA has drilled holes through some of the major faults in eastern Tennessee. Diamond core borings at Chickamauga Dam went through the Missionary Ridge fault and the cores through the fault zone came out unbroken. Upon being hammered, however, the core did break along the fault. The fault was not simply 'healed' or recemented with secondary deposits of calcite or dolomite, but was a very tight contact along which apparently pulverized material had recrystallized.

Core borings have been made through the Knoxville fault at the Tellico Project in eastern Tennessee. Here again the core through the fault was recovered unbroken.

Therefore, the evidence available from all of the geologic studies that have been made suggests that all of our Appalachian Valley faults, including the Kingston and Whiteoak Mountain faults, are inactive. Regional and subregional fault maps are provided as Figures 2.5-7 and 2.5-8.

2.5.3.5 Correlation of Epicenters With Capable Faults

The relationships between regional faulting and regional tectonics is discussed in Sections 2.5.1.1.5, 2.5.1.1.6, and 2.5.3.4.

No capable faults have been identified within the site region.

2.5.3.6 Description of Capable Faults

No capable faults have been identified with the site region.

2.5.3.7 Zone Requiring Detailed Faulting Investigation

Not pertinent to the site as no capable faults have been identified within the site region.

2.5.3.8 Results of Faulting Investigations

Details of regional, site, and foundation faulting, dating techniques used, and results, are provided in Sections 2.5.3.2 and 2.5.3.4.

2.5.4 Stability of Subsurface Materials

2.5.4.1 Geologic Features

The Conasauga formation of the Middle Cambrian age is the principal foundation rock found at the site. This formation is discussed in Sections 2.5.1.1.6, 2.5.1.2.7, and 2.5.1.2.9.

2.5.4.2 Properties of Subsurface Materials

2.5.4.2.1 In Situ Soils

2.5.4.2.1.1 General Description

The unconsolidated deposits overlying bedrock are composed primarily of alluvial deposits on the elevated flood plain near the lake shore and terrace materials, deposited by the Tennessee River when flowing at a higher level, over the bench that covers most of the site area. The alluvium is composed of fine-grained, finely sorted silts and clays, with micaceous sand and some quartz gravel. The thickness of the unit varies, but drilling showed an average thickness of approximately 25 feet. Near the base of the terrace bench the alluvial deposits thin out to a feather edge. Included in the alluvial material are some fairly well defined beds of tough, blue-gray clay, containing carbonized fragments of wood. These are interpreted as old slough fillings.

The terrace deposits are much older than the recent flood plain deposits and their edge is marked by a distinct topographic bench some 30 feet high which lies from 200 to 1,000 feet northwest of the edge of Chickamauga Lake. Recent drillings show the thickness of the terrace deposits to vary from a minimum of 31 feet to a maximum of 46 feet. The average thickness is 40 feet.

Approximately the upper half of the unit is composed of sandy, silty clay and the lower half is much coarser, consisting of pebbles, cobbles, and small boulders of quartz or quartzitic sandstone embedded in a sandy clay matrix.

In contrast to the conditions at the Sequoyah site, very little residual material derived from weathering of the underlying shale is present under the terrace deposits at the Watts Bar site. In a few holes a foot or two of residual clay was encountered, but in most instances the terrace deposits are immediately underlain by a few feet of soft but unweathered shale.

2.5.4.2.1.2 Investigations

Field Investigations and Testing

Soil investigations were conducted at the site for the major features. Figures 2.5-185 and 2.5-185a shows the locations of borings for the soil investigations for the various features at the site.

The field exploration for in situ soils consisted of split-spoon borings, using standard penetration test procedures for all features, and borings for undisturbed sampling for most features, auger borings to determine the top of rock, and test pits to obtain undisturbed samples. Although most borings were made using dry procedures, some borings, specifically for the liquefaction study, were made using drilling mud with a fishtail bit to advance the boring.

The split-spoon borings were made at the plant site using the methods specified in ASTM D 1586. The purpose of these borings was to obtain disturbed soil samples for laboratory testing and to determine the standard penetration resistance, N, of the in situ soils. Disturbed samples from the split-spoon borings were sealed in glass jars after removal from the soil sampler and taken to TVA's Materials Testing Laboratory for tests.

Undisturbed soil sample borings were made to obtain undisturbed samples for laboratory testing. The soil samples were obtained using various types of samplers. The ends of each tube were sealed immediately after removal from the boring to preserve the natural moisture content of the sample.

The types of borings made for any feature were based on the design requirements for each feature. In addition to the borings made for SPT samples and undisturbed samples, a number of locations were tested using a cone penetrometer. The results of this testing are described in Reference [167].

Field Investigation and Sampling Techniques Along the ERCW Pipeline and 1E Conduit Alignments

As a result of the NRC's interest in the techniques used in the field for the investigations of the soils along the ERCW pipeline and 1E conduit alignments, the following specific information is furnished.

The initial field investigation was completed between July 24 and August 19, 1975 with two Mobile model B-50 drills. The standard penetration test (SPT) borings were advanced by dry methods using 3-3/8 inch inside diameter hollow stem augers. Standard-2 inch split-barrel samplers complying with specification ASTM D 1586 and equipped with light duty spring retainers were used for sampling. The string of tools was exclusively AW drill rods. Table 2.5-28 provides information on the weight of various lengths of drill rod. Table 2.5-29 provides the depth of each split-spoon boring from which the drill rod weight may be obtained. Safety-type 140-lb drive hammers were used. One wrap of rope was used on the cathead. Blow counts were recorded for each 0.5' interval driven and sample recovery recorded. Drilling and sampling were in accordance with ASTM D 1586 procedures. Sample descriptions were recorded on both the drilling log and sample tags. Samples were immediately sealed in glass pint jars and temporarily stored in an onsite building to avoid extreme temperatures.

The undisturbed sampling borings were also advanced by dry methods, but using 6-inch inside diameter hollow stem augers. Samples were taken with 5-inch diameter thin-walled tubas attached to a piston-type sampler conforming to specifications in ASTM D 1587. Samples were sealed on both ends with at least 1-inch of beeswax-paraffin sealing wax. Depths of sample recovery were recorded on drill logs and sample tags. Samples were transported on rubberpadded racks for temporary storage to an onsite building to avoid extreme temperatures. A covered vehicle with rubber-padded racks was used to transport the samples from temporary storage to TVA's Singleton Materials Engineering Laboratory. Certified soils technicians performed all handling, moving, and transportation of specimens.

A subsequent field exploration was completed between May 30 and July 3, 1979. Equipment used was a CME-55 drill and a Mobile B-50 drill. The methods and sampling equipment used on the SPT borings exactly match those described above for the report of March 17, 1976. Tables 2.5-28 and 2.5-30 provide information about the drilling equipment used for each boring.

Rotary drilling methods were used between sampling elevations in the undisturbed sample borings. Bentonite drilling fluid was used. The 5-1/2 inch wide drag bit was equipped with baffles which deflected the drilling fluid upward. Samples were obtained with 5-inch diameter thin-walled tubes attached to a piston sampler.

Samples were sealed on both ends with a beeswax-paraffin mixture and temporarily stored onsite to protect them from extreme temperatures. They were transported to the laboratory on rubber-padded racks in a vehicle driven by a soils technician.

No engineering testing was required on these samples. However, following standard practice, the tube samples were extracted and unit weights and general classification tests conducted and recorded.

Additional SPT borings were completed between November 4 and 24, 1981. All borings were drilled with a Mobile B-61 drill. Procedures followed the recommendations in Table 2.5-31. Tables 2.5-32 and 2.5-33 provide information about the drill rig and equipment used for each boring.

On all Watts Bar Nuclear Plant ERCW assignments, one drill operator was assigned to, and stayed with, a specific drill. Exceptions would normally occur only in case of illness or other personal emergencies. Such situations are not documented.

Ropes used in drilling standard penetration test borings are normally replaced when noticeably worn on the initiative of either the driller or inspector. There are no specific guidelines or documentation. During the 1975 and 1979 investigations, it is judged that the ropes were used and somewhat limp. During the 1981 investigations, the ropes were new and stiff in accordance with specific instructions.

During all investigations, a 140-lb Mobile safety-type drive hammer, Model 006981, was used.

Test pits were excavated by a Gradall excavator equipped with a 3 yd^3 smooth bucket. Side walls were excavated to about a 1 to 1 slope. Dewatering was facilitated by installing a section of perforated 18-inch diameter pipe surrounded by a + 3/4 inch crushed stone filter. Undisturbed samples were obtained by benching into the side wall and hand trimming 1 ft^3 blocks with handtools. The trimmed top and sides were covered with three alternating layers of cheesecloth and paraffin. The sample was then cut at the bottom which was covered in a similar manner. Samples were placed in a wooden box surrounded with damp sawdust padding. A soil technician immediately transported the blocks on styrofoam pads to the laboratory.

Laboratory Testing

The following laboratory tests were made on all split-spoon samples:

(1) Moisture content

(2) Atterberg limits (ASTM D 423 and D 424)

(3) Grain size tests (ASTM D 422)

(4) Classification (ASTM D 2487)

For features where undisturbed borings were located based on information obtained from the split-spoon borings, the results obtained from these tests were used in the assessment of the existing soil characteristics. In order to assure continuity between the split-spoon borings and the companion undisturbed boring, undisturbed soil samples were subjected to moisture content, Atterberg limits, grain size and classification tests.

Soil strength tests were conducted on the soils. The particular soil tests conducted on any given feature were dependent on design requirements. Following is a list of the soil tests that were conducted in the laboratory to determine the soil characteristics necessary for design.

(1) Unconfined compression (UC)--This test is used in defining the allowable bearing of a foundation on clay. Also, the sensitivity of a soil can be determined by using the UC test on remolded samples.

(2) Unconsolidated-undrained (Q)--This test is used to determine representative soil conditions during and immediately after construction.

(3) Consolidated-undrained (R)--This test is used in the analysis of clay foundations or embankments where the rate of construction permits partial consolidation, or on natural slopes, or cuts in clay which are subject to rapid drawdown.

(4) Consolidated-drained (S) direct shear--This test is representative of conditions where complete dissipation of pore pressures occurs, such as in soil foundation subject to long-term loads.

(5) Cyclic Q--This test was conducted on soils that had a sensitivity of 4 or greater, as determined by the UC tests on undisturbed and remolded samples.

(6) Cyclic R--This test was conducted on soils that exhibited a potential for liquefaction based on parameters reported by D'Appolonia (Journal of the Soil Mechanics and Foundations Division, ASCE, January 1970).

(7) Consolidation--This test was used to provide parameters for determining the settlement below a structure that was constructed on or above a fine grained soil layer.

Table 2.5-1 indicates the soil strength tests that were conducted on representative samples for any given feature. Laboratory testing was carried out by TVA's Materials Testing Laboratory and the U.S. Army Engineers Waterways Experiment Station. The Waterways Experiment Station conducted the Cyclic R tests used to determine the liquefaction potential of soils in the intake channel. All laboratory tests were performed in accordance with ASTM standards where applicable. Procedures that are generally and widely accepted were employed for those laboratory tests that have not been standardized by ASTM.

2.5.4.2.1.3 Test Results and Selection of Design Properties

The classification of the soil a found at this site is displayed on graphic logs (see Figures 2.5-186 to 2.5-202 and 2.5-282 through 2.5-338). The symbols used for classification of soils are in accordance with ASTM D 2487. In addition, it is standard TVA practice to add the prefix 'G-' to the symbol of soils containing 12% or more gravel particles but do not classify as gravel. The following prefixes are used in boring 150 designations:

(1) SS - Split-spoon boring

(2) US - Undisturbed boring

(3) PAH - Auger boring

5OOkV Transformer Yard (Non-Category I Feature)

Borings US-1 and US-2, as shown on Figure 2.5-186, has ground elevations of 743.2 and 742.1, respectively. Sampling started near the final grade at elevation 730 in silty or clayey sand. The lean or nonplastic sands extend to about elevation 717, at which elevation the water table was encountered. Below this elevation, terrace deposits consisting of poorly graded, silty, sandy gravel extend to about elevation 706, which is top of the weathered shale. Undisturbed sampling was impaired by the nonplastic, sandy soil structure of high gravel content. Dry soil densities are from about 95 to 113 pcf with corresponding void ratios from 0.48 to 0.78. A saturated, lean silt, ML, in US-1 at elevation 714.7 to 714.1 had the low density of 87.9 pcf and a void ratio of 0.92; but the standard penetration resistance at this elevation resulted in 50 blows. One sample each in US-1 and US-2, being fine grained, allowed unconfined compression testing which resulted in 2.2 and 1.5 tsf. These soils exhibit low sensitivity. The results of the laboratory testing are summarized in Table 2.5-2.

Overall, the favorable laboratory test results were confirmed by high standard penetration resistance as shown in companion borings 2 feet apart from the undisturbed borings. An increase of penetration resistance with depth, primarily due to increased gravel content, is apparent. However, in both borings above elevation 718, strata with less than 10 blows per foot were determined. A correlation of these low N-values with laboratory tests does not bear out a particular instability.

In the transformer yard, subsoils are generally of high density and medium to high soil penetration resistance. The non-uniformity in bearing capacity of soils above elevation 715 should not effect differential settlement as long as loads do not exceed 1.5 tsf.

5OOkV Switchyard (Non-category I Feature)

Boring US-3 (see Figure 2.5-186) in the area of the switchyard shows similar subsoil characteristics to soils in the transformer yard. Silty sands grade into gravelly, silty sands below elevation 727 and into well-graded, silty gravel below the water table at elevation 714. A 2-foot gravel layer overlies highly weathered shale which classified CL, SM, G-CL, and SC. Standard penetration tests show very hard consistency below elevation 715, but immediately above this elevation an isolated, relatively weak layer exists from elevations 716 to 719. This G-SM layer contains about 30% fines, is of low plasticity, and has relatively low dry density, averaging 106 pcf. Immediately above it, soils of similar texture show high penetration resistance.

With 10 feet of very firm to hard subsoils between elevations 730 and 720, it will largely depend on the size of the superimposed load and its influence on the low-blow soil layer above the water table in order to predict settlement of switchyard foundations.

Switchyard subsoils based on only one boring, are not unlike those in the adjacent transformer yard, except for a weak layer above the water table, elevation 715. Soils 10 feet thick overlying this loose layer, owing to hard consistencies, will provide some bridging. Consolidation of the sandy soils will be rapid. Barring localized unstable conditions, allowable soil bearing capacities are in excess of 1.5 tsf. The results of the laboratory testing are summarized in Table 2.5-3.

Cooling Towers (Non-Category I Feature)

Borings SS-4, SS-5, SS-6, US-7 and US-8 were located in the area of the north tower, as shown on Figure 2.5-185. Graphic logs are shown in Figure 2.5-187. Fine-grained subsoils are evident to about elevation 725 and are underlain by silty sands open interspersed with cherty gravel. The weathered shale in this area was determined at about elevation 709, and in two borings, SS-4 and US-8, the top of shale coincides with the water table. In borings SS-5, SS-6, and US-7 the water table was located between elevations 716 and 718. A marked increased in penetration resistance was generally found below elevation 713 when gravel content increased above 30%. The center of the tower at boring SS-6 shows marginal blow counts ranging from six to nine to elevation 715. The four borings along the perimeter indicate greater firmness and penetration resistance. Unconfined compressive strength tests performed on soils above elevation 721 indicate allowable bearing capacities from 1 to 6 tsf with a low sensitivity ratio. Consolidation tests resulted in C_c values from 0.07 to 0.22, indicative of low to moderate soil compressibility. Only one SM soil from boring US-7, elevations 723.9 to 722.6, had a C_c of 0.31, revealing medium compressibility. Preconsolidation indices, Pc, were from 1 to about 4 tsf. High preconsolidation loads, especially in the surficial soils, appear to be the results of desiccation. The results of the laboratory testing are summarized in Table 2.5-4.

Borings SS-9, SS-10, SS-11, US-12 and US-13 were located in the area of the south tower, as shown in Figure 2.5-49. Graphic logs are shown in Figure 2.5-187. Cohesive subsoils generally above the water table are of the CL, Ch, gL, and MB types, and unconfined compressive strengths were found to be appreciably lower than that of soils in the area of the north tower. It will be noted that plasticity indices of the subsoils are higher than those established for soils in the north tower foundation.

At the time of the investigation the water table in these five borings was established between elevations 719 and 714. UC values range from 0.4 to 3.6 tsf, but the average unconfined compressive strength is 1.3 tsf. These are not sensitive soils. Foundation soils generally exhibit lover dry densities, higher void ratios, and somewhat lower preconsolidation loads, ranging as indicated in Table 2.5-5.

The two locations of the cooling towers indicate a marked difference in subsoil conditions. The soils in the area of the north tower founded on insensitive alluvial soils of fairly high preconsolidation, especially along the perimeter of the tower. The center boring shows reduced bearing capacity in the top 20 feet. This weakened condition exists along a line running from boring SS-5 to SS-10. The soils in the area of the south tower, as shown by various soils parameters, do not have the same overall stability, mainly due to saturated soil strata at or immediately above the water table. Bearing capacities as low as 0.4 tsf were determined on these nonsensitive subsoils. Below elevation 715 soil stability is greatly increased. Due to the weaker soil strata for the south tower, both towers were constructed on pile foundations that bear in bedrock.

CCW Pumping Station (Non-Category I Feature)

Borings SS-14 through SS-18, as shown on Figure 2.5-188, had ground elevations ranging from 733.3 to 736.0. The water table was established at about El. 720. Sampling was required only below El. 715, and the upper 20 feet of overburden was augered without obtaining samples. Visually, these soils were classified fine-grained, cohesive silts and clays.

In borings SS-14 and SS-15, about two feet of fine-grained, cohesive soils are present between elevations 715 and 713. Materials between 715 and 705 are terrace sands and gravels. The rounded gravel and sand-size particles are made up of quartz, quartzite, and a small amount of chert. Gravel contents in these materials are variable and as high as 58%. Beneath the terrace deposit, weathered shale is overlying the sound shale which was encountered at about El. 702. Shaly soils classified lean clay (CL), silt (ML), and silty sand (SM), with shaly gravel reflecting the varying degrees of weathering.

Natural moisture contents are generally moderate except for a layer of silty clay (ML-CL) encountered in boring SS-15. This material shows a moisture content exceeding the liquid limit, thus indicating weakness. The soft consistency was also confirmed by standard penetration testing and a low number of blows. High penetration resistance of subsoils, however, is indicated throughout the foundation except for the above-mentioned isolated weak layer.

In summary, subsoils at this site appear to be stable, with the exception of a small pocket of cohesive soil located between El. 713 and El. 715 in boring SS-15. This material was excavated during the preparation of the foundation. The underlying granular terrace deposit is considered dense. Standard penetration blow counts indicate the weathered shale to be hard.

The granular material could be tested for sensitivity, but the probability of liquefaction for soils of this texture and high density is remote. Below El. 715, fine-grained soils are either of shaly configuration or present in minimal quantity. The recommended allowable safe bearing capacity as determined by inplace testing is estimated to be in excess of 2.0 tsf.

Service and Office Buildings (Non-Category I Feature)

Borings SS-19 through SS-24, as shown on Figure 2.5-189, had ground surface elevations ranging from 733.8 to 744.5. The overburden depth varied from 38.5 to 46.5 feet, with shaly bedrock encountered between El. 695 to El. 700. The water table was established at about El. 725.

Sampling and in-place standard penetration testing were carried out from footing grade to bedrock. Above the proposed grade elevation, no sampling or testing was done. In three of the borings, SS-19, SS-20, and SS-24, sampling below El. 727 shows a relatively uniform soil profile consisting of silty sand and gravelly, silty sand, SM and G-SM, extending to about El. 712, underlain by 4 to 10 feed of poorly graded gravel, GP. Beneath these alluvial materials 5 to 11 feet of hard, weathered shale, ML, CL, and G-SM-SC, overlie bedrock.

Standard penetration resistance in these borings reveals a pronounced weakness, N<10, in the silty sand, SM strata to El. 715 in borings SS-20 and SS-24 and to El. 717 in SS-19. In boring SS-24, above El. 720, a layer of stiff, lean clay, 3 feet thick, has a relatively high bearing capacity, but is not considered of sufficient thickness to effectively bridge over the weak underlying silty sands.

Borings SS-21, SS-22 and SS-23 indicate that footings at El. 709 will be founded on 3 to 6 feet of clean, poorly graded gravel, GP, which is underlain by 3 to 8 feet of residual, weathered shale, ML and G-SM-SC. These soils are of high relative density or hard consistency, as determined by standard penetration resistance.

In summary, this foundation exploration at the proposed site for the Service and Office buildings discloses nonuniform soil bearing capacities. In the area of borings SS-21, SS-22, and SS-23, footings located at El. 709 will be founded on poorly graded gravel of bearing capacities in excess of 4,000 psf. In the area of borings SS-19, SS-20, and SS-24, footings to be founded between El. 718 and El. 727 will be in weak, silty sands with bearing capacities ranging from 500 to 1,500 psf. Under these conditions, settlement, even under light loads, could be significant.

The foundation for the service building consists of piles driven to bedrock and the foundation for the office building consists of spread footings on undisturbed soil or Class A backfill.

Diesel Generator Building (Category I Feature)

Borings SS-25 through SS-28, as shown on Figure 2.5-190, had ground elevations ranging from 734.1 to 735.0. Undisturbed borings were made to sample typical representative soil types identified in the split-spoon borings. The water table was established at about elevation 727.

The graphic logs reveal firm silty gravel under the entire building area uniformly below elevation 713, with lean clay, silt of low plasticity, and sandy silt above the building foundation.

The Diesel Generator building is rectangular in plan with dimensions approximately 120 feet by 95 feet. The grade floor slab consists of diesel fuel storage tanks encased in concrete and is approximately 10 feet thick with the bottom at approximately elevation 732.

The bearing pressure under the grade slab will be approximately 2,500 psf. As can be seen from the summary of the test data given in Table 2.5-6 and the standard-penetration data shown on Figure 2.5-190, it cannot be assured that the material between elevation 713 and the grade slab is capable of safely supporting 2,500 psf. The silty gravel from elevation 713 to the top of rock is capable of supporting the imposed load as shown by the standard-penetration data given in Figure 2.5-190. The results of the laboratory testing are summarized in Table 2.5-6.

In order to assure a safe foundation for the building, the fine grained soils above the in situ gravel were removed and replaced with granular fill as illustrated in Figure 2.5-226. The criteria for the granular fill is discussed In Section 2.5.4.5.2.

Intake Station and Channel (Category I Feature)

The intake channel is a man-made feature extending approximately 800 feet from the edge of the reservoir through the flood plain to the intake pumping station. The bottom of the channel is elevation 660, and is 50 feet wide. Channel earth side slopes are one vertical on four horizontal. The nominal ground surface is elevation 695. Groundwater is near elevation 685. The location of the channel with respect to the plant layout is shown in Figure 2.1-5. The channel is illustrated in Figure 2.4-99.

The layout of holes is as shown on Figure 2.5-185 and 2.5-185a. Three lines of borings at about 200 feet spacing (borings 30-34, 35-39, and 41-45) were laid out parallel to the intake channel with additional borings near the river bank (40 and 46) and in the slough on the flood plain (47 and 48). Boring 29 was drilled during early site investigation. The layout included 19 (30-48) standard-penetration split-spoon borings with sufficient undisturbed borings (adjacent to split-spoon borings) to sample all types of soils in the profile.

Initial alternate split-spoon borings were taken to the top of rock to obtain general rock elevations and to confirm the presence of firm gravel in the lower part of the soil profile as was indicated in the earlier site exploration. Successive split-spoon borings were made into the firm gravel between these initial borings in order to confirm the general uniformity of soils above the gravel. Borings 33, 36, 38, and 44 were not made since the uniformity of the profile was disclosed by the other borings. Graphic logs of all borings are shown on Figures 2.5-191 through 2.5-195.

The graphic logs reveal firm silty and sandy gravel under the entire intake channel area, below about elevation 665 across the flood plain, and below elevation 675 at the intake structure. Above the gravel are lean clays, silts of low plasticity, and silty sands.

Index tests for soil classification, moisture, mechanical analysis, and Atterberg limits on the split-spoon samples were used to reflect locations for undisturbed borings for sampling all types of soils in the profile. Since the split-spoon borings confirmed gravel in the lower part of the soil profile, the undisturbed samples were taken only in soils above the gravel. Five continuous undisturbed borings were made beside split-spoon borings at these locations. Sampling and testing of the basal gravel is described below.

Laboratory tests on undisturbed samples are recorded in Tables 2.5-7 through 2.5-9. Included in these tables is standard-penetration data in the split-spoon boring adjacent to each undisturbed boring. On both the logs and the tables, samples are identified by capital letters and lowercase letters.

The identification system of using uppercase and lowercase letters can be described as follows. Capital letters identify the representative undisturbed samples that are subjected to strength tests. Lowercase letters identify other undisturbed samples and disturbed samples considered to be the same soil as a corresponding capital letter sample. Letter designations on graphic logs and data tabulations permit easier reading of the soil profile and provide a record of the adequacy of the selective sampling and testing. The letter designations are completely arbitrary and apply only to the one project, or even to a single project feature.

The process for selecting test samples is described as follows. Split-spoon standard penetration borings are made to explore the area. The disturbed samples are examined for index properties. This data, with penetration records, is used to determine soil types distribution in the profile in order to select specific locations at which to obtain inclusive representative undisturbed samples for strength testing. Letters are assigned to the indicated separate soil types. The undisturbed samples are taken and tested also for index properties, and density and void ratio. Letter designations are then finalized, with some changes in previous designations, and with possible variations in some "same" samples' properties, because of judgment designation based on all properties. Representative undisturbed samples are given capital letter identifications and are tested for strength.

The silty sand is deposited on top of the firm gravel from approximate elevation 665 to 680, and the lean clay (or silt) from approximate elevation 680 to 695. Strength properties of these soils in situ were obtained from the test results shown on Tables 2.5-7 through 2.5-9. The results of the shear tests are plotted in graphical form (Figures 2.5-247 through 2.5-250) and a value of c and φ was selected for design.

The soils exploration disclosed a possible weak layer of lean clay soil at approximate elevation 690 to 685 in borings US-30 and US-36, which are on opposite sides of the channel near the reservoir. The test results indicate the minimum strength properties of this material as $\varphi = 3°$ and c = 500 psf.

Cohesive soil samples were tested for sensitivity. Of the many samples, the four with sensitivity greater than 2 were remolded to in situ density and moisture content and had unconsolidated-undrained (Q) shear tests run. The results are shown in Tables 2.5-8 and 2.5-9. The tests do not indicate serious strength loss.

The liquefaction potential of the site soil deposits are discussed in Section 2.5.4.8.

The basal gravel is located on top of rock at approximate elevation 650, and extends to approximate elevation 665. A trench was made in the flood plain for access to the basal gravel for undisturbed sampling. Gravel sizes up to 6-inches and water conditions in wet weather prevented useful undisturbed sampling. Successive essentially saturated grab samples were taken to a depth of 4-feet with a reasonably tight clamshell bucket. Fines contents in the samples so obtained were about 3%, compared with about 5 to 10% in previous boring sampling. Samples were scalped to maximum 2-inch size, scooped into a 12-inch-cube direct shear box, consolidated in submerged condition under equivalent overburden pressure of 3000 psf, and sheared under submerged conditions. Figures 2.5-203, 2.5-204, and 2.5-205 show gradation and shear test results. The gravel strength used in design is $\varphi = 42°$ and c = 0. The shear test results show an 0-load intercept of 0.4 to 0.6 tsf, representing interlock of particles in the shear box. Since the magnitude of this effect in the gravel mass cannot be assured, it is ignored in the basic stability analyses.

Due to unexpected soil conditions encountered during the excavation of the intake channel, an additional investigation was made and this information is provided in Section 2.5.5.2.2.

Class 1E Electrical Conduits Alignment (Category I Feature)

The Class 1E conduits furnish electrical power and control for the pumps, valves, screens, control boards, etc., at the intake pumping station. The soils investigation for the conduit alignment was to establish the dynamic soil properties along the alignment and to provide soil strength information for any slopes that would have to be qualified if the conduits were constructed in the slopes.

The layout for the soils investigation is shown on Figure 2.5-273. The graphic logs for borings 49 through 63 are shown on Figures 2.5-196 and 2.5-197, and for borings 171 through 177 are shown on Figures 2.5-274 through 2.5-280. The graphic logs indicate that the over-burden varies from 24 to 60 feet thick. Weathered shale is encountered at depths of 10 to 32 feet. The water table was established between El. 690 and El. 710. Figure 2.5-281 shows a profile along the 1E conduit bank from the ERCW pump station to the main plant with borings spaced along the a1ignment.

The overburden consists primarily of lean clay (CL) and silt (ML and MH) with small quantities of silty and gravelly sand (SM and G-SM), and silty gravel (GM and GP-GM). Below the top of the weathered shale the laminated shaly materials are classified as sand-sized soil. The granular portion of the soils above the shale is made up of silicious and micaceous sand and subangular to rounded sandstone and cherty gravel.

The standard penetration test results shown on the graphic logs indicate soils of a medium to stiff consistency with blow counts usually between 10 and 30 blows per foot. In a few instances, usually near the water table, the penetration results indicate a loose or soft consistency. The liquefaction potential of the samples with a loose consistency was evaluated and is discussed in Section 2.5.4.8.

The results of the laboratory testing are summarized in Tables 2.5-10 and 2.5-11. The strength values used in design are represented in Table 2.5-12. The values used for design (Table 2.5-12) are low averages for all of the strength data shown in Tables 2.5-10, 2.5-11 and 2.5-24. The results for each type shear test are plotted in graphical form (Figures 2.5-206 through 2.5-208), and a value below the average for c and φ is selected to be a conservative value to use in the design. There were no sensitive soils encountered in the investigation. The dynamic soil properties are discussed in Section 2.5.4.4.2.

ERCW Piping Alignment (Category I Feature)

The Essential Raw Cooling Water piping furnishes water for cooling the reactor during emergency condition. Additional piping along the same alignment furnishes water for extinguishing fires (High Pressure Fire Protection (HPFP) piping). The soils investigation for the piping alignment was to establish the dynamic soil properties along the alignment and to provide soil strength information for any slopes that would have to be qualified if the piping were constructed in the slopes. The results of the investigation of the dynamic soil properties are provided in Section 2.5.4.4. The location of the borings for the soils investigation is shown on Figure 2.5-185. The graphic logs for all soil borings are shown on Figure 2.5-198 through 2.5-202, 2.5-282 through 2.5-330, 2.5-332 and 2.5-333. The graphic logs indicate an overburden that varies from 10 to 66 feet, and averages 37 feet. Weathered shale was encountered at the surface at one boring location and at depths up to 37, feet over the remaining portion of the site. Bedrock ranges from El. 668 to El. 699 with an average elevation of 685.6.

Alluvial soils consist of lean to fat clay (CL and CH), lean to highly plastic silt (ML and MH), along with smaller amounts of silty and clayey sand (SM and SC), and silty and clayey gravel (GM and GC). The coarse-grained portion of these soils includes silicious and micaceous sands and rounded to subangular gravel of 1-inch maximum-recovered size. Below the top of weathered shale the laminated residual materials classify lean clay and silt (CL and ML), silty and clayey sand (SM and SC), and silty or clayey gravel (GC and GM).

In situ standard penetration testing disclosed the alluvium to be of medium to stiff consistency. In some borings, usually near the water table, the penetration results indicate a soil with a loose or soft consistency. The liquefaction potential of the samples with a loose consistency was evaluated and is discussed in Section 2.5.4.8.

The residual shaly soils are generally of stiff to hard consistency. The only weakness established in these subsoils was near the contact with the overlying aluvium.

The results of the laboratory testing are summarized in Table 2.5-24. The strength values used in design are represented in Table 2.5-12. The values used for design are low averages for the strength data shown in Tables 2.5-10, 2.5-11, and 2.5-24 except for the strengths of organic samples. The results of each type shear test are plotted in graphical form (Figures 2.5-207, 2.5-241, and 2.5-242) and a conservative value below the average for c and φ is selected for use in the design. There were no sensitive soils encountered in the investigation. The dynamic soil properties are discussed in Section 2.5.4.4.2.

The shear strengths of the organic samples were not considered in the selection of design values, because the organic samples were not representative of the soils through which the piping will be constructed. The organic samples appear in only boring SS-107 and that boring is located approximately 100-feet from the piping alignment.

Boring SS-107 is located in an intermittent stream and the graphic log (Figure 2.5-202) shows that bedrock is shallow. Boring SS-94, similar to boring SS-107, is located in an intermittent stream, but located along the piping alignment and also has a shallow depth to bedrock. This indicates that during construction, organic deposits located along the piping alignment would be exposed and removed.

Cyclic Testing - ERCW Piping and 1E Conduit Alignments

In the process of reviewing the potential liquefaction of the soils along the ERCW piping and 1E conduit alignments (Section 2.5.4.8) several samplers were selected for cyclic testing. Two stages of testing were done. The initial stage consisted of cyclic triaxial (R) tests on undisturbed samples obtained from borings and is described as follows.

Silty sand (SM) and sand silts (ML) are present in borings 49, 50, 59, 60, 65, 67, 87, and 88. The location of these borings are shown in Figures 2.5-185 and 2.5-185a. Graphic logs are shown in Figures 2.5-196, 2.5-197, 2.5-198, and 2.5-200. In borings 49, 67, 87, and 88, the SM and ML material with standard penetration test blow counts of about 10 or less are either below the top of weathered shale or above the water table. Based on the information obtained from the borings of the in situ soils, the nonplastic SM material with low blow counts in boring SS-50 (approximate elevation 698.0) and SS-65 (approximate elevation 710.0) are judged to be the most susceptible to liquefaction. These two areas were investigated to obtain samples for cyclic testing.

Additional split-spoon borings were located as close as possible to the original locations of SS-50 and SS-65. The split-spoon borings were to locate the material desired for cyclic testing. Once located, undisturbed borings were drilled 5 feet from the split-spoon boring to retrieve the samples. For location SS-50, two undisturbed borings were made, one 5-feet and the other 10-feet from the additional split-spoon boring. Undisturbed samples were recovered from both borings. Figure 2.5-339 shows the graphic logs for the original split-spoon borings (SS-50 and SS65) along with the additional split-spoon (SS-50-1 and SS-65-1) and undisturbed (US-50-1 , US-50-1A, and US-65-1) borings. The laboratory test data for the undisturbed borings are given in Table 2.5-34. Grain size curves for these samples are given in Figures 2.5-340 through 2.5-352.

The soils selected for cyclic testing were from US-50-1 at elevations 698.9-696.6 (sample 2), 696.4-695.3 (sample 3) and 695.3-694.5 (sample 4). These samples contain 85%, 88%, and 53% sand, respectively, with the remainder being silt and clay in about a 3:1 ratio. Sample 4, with 53% sand, was the first sample selected for cyclic testing. It served both as a useful test and as a calibration sample. Samples 2 and 3 are nonplastic and contain 82% and 88% sand, respectively, and have the highest sand content and the lowest silt and clay content of all the samples. Sample 3 contains the least silt (9%) and clay (3%) and the second highest void ratio (1.002). The sample (sample 2 in boring US-50-IA) with the lowest dry density (79.2 lb/ft^3) and highest void ratio (1.148) contains 67% sand, 22% silt, and 11% clay. It was not selected for testing and is not considered critical based on the test results for sample 4 and the high (33%) fines content.

The results of the cyclic tests are presented in Table 2.5-36 and Figure 2.5-353. These tests were all performed with an effective confining pressure of 1,000 lb/ft^2 which represents a vertical pressure of 2,000 lb/ft^2 (approximately 15-20-feet of overburden soil). These test conditions approximate field conditions. The cyclic stress ratio $(\sigma_d / 2\sigma_3)$ was conservatively limited to 0.5. Cyclic tests for samples 2 and 4 were performed on undisturbed specimens. For sample 3, a reconstituted specimen was used because of a large gravel in the tube sample. For the reconstituted specimens, a moist tamping method was adopted in which the material was placed in five layers with each layer compacted to a prescribed dry unit weight. The final density of the test specimen was the same as the in situ density.

The second stage of cyclic testing consisted of cyclic triaxial (R) tests on silty sands and cyclic simple shear tests on clayey silts and silty clays. The samples for this testing were obtained from block samples from two test pits along the piping alignment. The test pits and the results of the cyclic testings are described in a report entitled, 'Watts Bar Nuclear Plant Liquefaction Evaluation of the ERCW Pipeline Route' (Reference 167).

The test pit samples are considered to be representative of actual field conditions based on a comparison of the soil classification, grain size distribution, and densities of the test pit samples, with samples from the soil borings.

Tables 2.5-37 and 2.5-38 are comparisons of the classification data for the samples from test pits 1 and 2, respectively, with the classification data for SM soils from the split-spoon borings closest to each test pit respectively. Figure 2.5-354 is a plot of the gradation of the samples from test pit 1 compared with the range of gradations for the split-spoon samples given in Table 2.5-37. Figure 2.5-355 is a plot of the gradation of the samples from test pit 2 compared with the range of gradations for the split-spoon samples given in Table 2.5-38. The information contained in these tables and figures shows that the data on the undisturbed block samples correlates very well with the data from the split-spoon borings nearest the test pits.

Tables 2.5-39 and 2.5-40 are tabulations of the classification data for the split-spoon samples from the borings along the ERCW pipeline in the area south of the cooling towers and in the main plant area, respectively, and have a factor of safety less than 1.05 as calculated and presented by our consultant in the report, 'Liquefaction Evaluation of the ERCW Pipeline Route - Watts Bar Nuclear Plant'[168]. These factors of safety are calculated on the basis of standard-penetration test blow counts and are summarized in Table 1 of the referenced report. Figure 2.5-356 is a plot showing the mean gradation for the test pit samples in comparison with the maximum, minimum, and mean gradations of the split-spoon samples in Table 2.5-39. Figure 2.5-357 shows the same information, but for the split-spoon samples In Table 2.5-40. These two figures show reasonably good correlation between the gradation of the test pit samples and the gradations of the split-spoon samples that have the lowest factors of safety in the liquefaction analysis.

Table 2.5-41 is a comparison of the classification and density data on the test pit samples and the undistributed SM samples taken along the ERCW pipeline. The average dry density for the undisturbed samples from the soil borings was 90.4 lb/ft^3 and for the undisturbed samples from the test pits was 86.4 lb/ft^3. This is reasonably good agreement. Since the test pit samples had a lower density than the samples from the undisturbed borings, this indicates that the results from the test pit samples are not only valid but are representative of the worst field conditions at the site.

In Situ Basal Gravel

As a result of the NRC staff's concerns about the properties of the basal gravel at the site, an additional soil investigation was conducted in the vicinity of Category I soil-supported structures between June 4 and July 6, 1979. The soil exploration and testing program was designed to determine the properties of the in situ basal gravel and the weathered shale.

Twenty-six borings were drilled at the six locations shown on Figure 2.5-358. Samplers used included 2 inch outside diameter split-spoon sampler; a 7-3/4-inch outside diameter Sprague and Henwood soil-sampling core barrel; a 5 inch outside diameter Shelby tube sampler; and a 6 inch outside diameter heavy-duty flat spiral slit sampler.

The split-spoon borings were first made and the soil stratification was identified. The split-spoon borings were drilled approximately to the top of bedrock (auger refusal). After completion of the split-spoon borings, the Sprague and Henwood sampler was used in an attempt to take undisturbed samples of the basal gravel at location 125. Hollow stem augers (with 6 inch inside diameter stem) were removed after being used to advance the boring to the top of the basal gravel. A casing was then placed in the boring to stabilize the hole and allow entrance of the Sprague and Henwood sampler. Using drilling water, the sampler was then rotated and advanced slowly to refusal or until the sampling interval was completed. The coarser fraction of the basal gravel was recovered, but the fines were washed away by the drilling water. Three unsuccessful sampling attempts were made in this manner at locations 125 and 128 with the same results. The Sprague and Henwood sampler was abandoned after the cutting edge of the sampler had been worn.

Attempts were also made to obtain undisturbed samples of basal gravel using 5 inch outside diameter Shelby tube samplers. Relatively undisturbed samples were obtained by rotating Shelby tubes through the basal gravel and penetrating the weathered shale (clay residuum) below it to form a plug. Samples taken by this method were disturbed to some extent but represent the best among the possibilities to obtain samples of the basal gravel suitable for in situ density determination. Some of the basal gravel samples increased in volume showing more material recovered than the sampler penetrated. This volume increase was apparently due to shifting and realignment of the gravel as the sampler was rotated into the material. Several attempts were made to obtain the basal gravel samples with Shelby tubes because the gravel layer containing material larger than 5 inches in diameter caused refusal or displacement of the sampler.

Additionally, undisturbed Shelby tube samples were also obtained from the upper portion of the weathered shale residuum having an N (standard penetration number of the blow counts) value below 30.

The final phase of the sampling program consisted of drilling six heavy-duty sampler borings to supplement the representative basal gravel material obtained during the undisturbed phase of sampling. Due to the design limitations of the heavy-duty sampler, the maximum particle size recovered was 3 inches to 4 inches in diameter.

Graphic logs shown in Figures 2.5-359 through 2.5-364 indicate the soil profile; number of blow counts; natural moisture content; soil classification; Atterberg limits; groundwater elevation; sampling elevation; and shear strength and consolidation test data. Profiles were not plotted at locations where only representative samples were obtained.

The borings indicated that the top of in situ basal gravel stratum varied from elevation 710.5 to 714.0 and extended to elevations ranging from elevation 706.0 to 709.0. The materials encountered between the existing (finished) grade elevation to the top of the in situ basal gravel consisted of surficial crushed stone or sod underlain by a lean clay fill and/or fine-grained material (mostly in situ alluvium). The investigation of these materials was not included in this program since all Category I soil-supported structures are founded either on in situ basal gravel or compacted granular fill after excavating all the material overlying the in situ basal gravel or bedrock.

Furthermore, the backfill and the in situ alluvium above the foundation elevation will not have a significant effect on the bearing capacity of the subject foundations.

The in situ basal gravel classified from a poorly graded silty sand to a well graded sandy, silty gravel. The N values for the in situ basal gravel ranged from 16 to 50+.

Weathered shale was encountered directly below the basal gravel. The thickness of weathered shale residuum ranged from approximately 7.4 feet to 23 feet. When soil classification terminology is used, the weathered shale varied from a lean clay to lean silt to a gravelly silty sand. The N values for the weathered shale are mainly in the 50+ range which indicates that, in general, it is a hard material. The top uppermost portion of weathered shale (generally the interface between the basal gravel and the hard weathered shale) revealed somewhat lower N values ranging from 20 to 32. This indicates that even the relatively weaker layer of weathered shale has a very stiff consistency. Boring SS-130 revealed a local spot in the weathered shale with N value equal to 3. In order to investigate this soft spot, a confirmatory boring SS-130A was drilled within 15 feet of SS-130. The standard penetration test was conducted and the N values at the location were 24 and 27 and there was no evidence of any soft material. Therefore, it was concluded that the soft spot was either a localized small picket or the standard penetration test was conducted on a disturbed material (before cleaning the hole).

Bedrock (auger refusal) was encountered between elevations 687.9 and 701.6.

The water table ranged from elevations 711.5 to 724.0.

All split-spoon samples were tested for natural moisture content, Atterberg limits, and/or grain size distribution. The in situ densities of the relatively undisturbed basal gravel samples were determined. All stones larger than 2 inches were removed from the representative disturbed samples of basal gravel. The remaining material (minus 2-inch material) was loosely placed at natural moisture content in a 0.5 ft^3 (12 inch by 12 inch by 6 inch) specimen box of the direct shear machine. The specimen was inundated with water and consolidated under a load of 2,000 lb/ft^2 which is approximately equal to the effective overburden pressure on the in situ basal gravel (computed from the original ground contours). Consolidated densities were then computed for each sample. Test specimens were remolded to approximately the consolidated and the in situ densities and tested for direct shear S (consolidated-drained) strength. The basal gravel material from boring location 125 could not be remolded to the low dry density value of 97 ft^3. Therefore, the test was performed at a dry density of 113.8 ft^3, the lowest density attainable. The in situ and consolidated densities and the direct shear test results are presented in Table 2.5-42. The strength envelopes are shown in Figure 2.5-365. The grain size distribution curves of the representative basal gravel are shown in Figures 2.5-366 through 2.5-371. Photos of the representative basal gravel material are presented in Figures 2.5-372 and 2.5-373.

Undisturbed samples of the top portion of the weathered shale (interface between the basal gravel and hard weathered shale) were tested for natural moisture content; Atterberg limits; density; triaxial Q (unconsolidated-undrained); triaxial R (consolidated-undrained); and consolidation tests. The test results are presented in Table 2.5-43. The Q, R, and R strength envelopes are shown in Figures 2.5-374 through 2.5-376.

The soil properties adopted for determining the bearing capacities of the foundations for soil-supported Category I structures are presented in Table 2.5-44. Most of the adopted properties were evaluated on the basis of the soil investigation program discussed above. The properties which are assumed in the absence of test data are conservative.

2.5.4.2.2 Rock

2.5.4.2.2.1 Engineering Description of Bedrock

The geology at the Watts Bar Nuclear Plant site consists of interbedded shales and limestones of the lower Conasauga Formation, Middle Cambrain age, overlain by alluvial material averaging 40 feet in thickness. The structural geology of the area is extremely complex, and can best be described as a system of small tight folds and crinkles superimposed on an average strike of N30°E and an average dip of 30°-45°E.

During the borehole investigation it was not possible to correlate individual shale and limestone beds between holes.

Except for some soft areas at the bedrock surface, both the shale and the limestone appear fresh and unweathered, and there is no evidence of solution in the limestone horizons.

In order to make an engineering analysis of the rock, and to provide a tenable means of logging the complex geology in the core, the following classification system was developed:

Rock Type	Description
0	Core Loss--Zones of core not recovered either because of grinding between harder overlying and underlying strata or because they were too soft or fragmented to be recovered by conventional hard rock drilling methods. No cavities were observed with the borehole TV apparatus. For engineering purposes, all zones of core loss are assumed to be Type I rock.
1	Soft Shale--Material which, when removed from the core barrel, can be easily scratched with a fingernail, or is in such small, although sound and unweathered pieces, that it resembles an agglomerate rather than solid rock. The physical character of the soft shale is generally a function of intense folding and crinkling, and is not generally attributed to weathering. In most instances the individual shale particles are bounded by slicken-sided surfaces.
2	Hard Shale--Shale which, when removed from the core barrel, cannot be easily scratched with a fingernail. It is recovered in relatively large discrete pieces which may break down upon exposure to discs of shale (also called 'poker chips'). Slickensides are present but are not as plentiful as in the soft shale, and are generally concentrated along bedding surfaces.

3 Limestone--Pieces of core that are either entirely composed of limestone or are composed predominantly of limestone with a few thin shale stringers.

Photographs showing the general nature of the three rock types are presented in Figure 2.5-209. Watts Bar Nuclear Plant shale specimens tested in the University of Illinois Rock Mechanics Laboratory[142] demonstrate the following additional physical properties:

Rock Type	Unit Weight pcf	Natural Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index
1	157 to 165	4.6	18.8	19.6	Nonplastic
2	160 to 169	1.3 to 2.5	--	--	Nonplastic

The average core contains about 16% limestone. Soft shale, including core loss, averages about 37% in the upper 50-feet of rock and 19% for the interval from 50-feet to 150-feet into rock. The increase in percent soft shale near the rock surface is most likely the result of the combined effects of stress relief and superficial weathering.

The water table is in the alluvium, 20 to 30 feet above the bedrock surface throughout the main plant area. Of the six boles in the plant area which were water pressure tested, none revealed water takes in the bedrock. The bedrock surface varies from elevation 690 to 700, averaging 697.

2.5.4.2.2.2 Test Program

Figure 2.5-210 shows the layout of holes tested and the tests performed in each. Both Menard Pressuremeter tests and Birdwell 3D sonic logs were made.

In all, 72 Pressuremeter Tests were performed in six holes. The pressuremeter testing program was designed to determine the static compressibility of the shales at the site.

The Birdwell sonic logs were obtained to determine dynamic moduli for use in the earthquake analysis. Continuous logs were made in seven holes, four of which were also tested with the pressuremeter.

2.5.4.2.2.3 Description of Testing Techniques

2.5.4.2.2.3.1 Menard Pressuremeter

The Menard Pressuremeter is a relatively simple device designed to stress the walls of a borehole to obtain the compressibility of soil and low strength rock in situ.

Dixon[139] presents the following description of the operation of the Menard Pressuremeter:

The Pressuremeter equipment consists of a combination volumeter-manometer connected to a cylindrical borehole expansion device, or probe. The probe consists of a steel tube surrounded by two flexible rubber membranes. The interior membrane forms the measuring cell, and the exterior membrane provides guard cells at the two ends of the probe. The guard cell is activated by gas pressure and is used to reduce end effects on the measuring cell to provide an essentially two-dimension, cylindrical stress condition. The measuring cell is pressurized with water and kept at a slightly higher pressure than the guard cell to ensure that it is always pressing against the borebole wall. Two concentric tubes connect the volumeter to the probe. An adapter connects the probe to a standard drill rod for lowering and raising the probe within the borehole.

After lowering the probe to the desired depth, pressure is applied to the borehole wall by inflating the rubber membranes. In addition to providing a means for applying increments of pressure to the measuring cell, the volumeter-manometer measures the radial expansion under each pressure increment by measurement of water flow into the probe.

The Pressuremeter Test is performed with each pressure increment being held steady for one minute.

For the six holes tested at Watts Bar, the upper 20 feet of rock was drilled in 5-foot increments, and one pressuremeter test was performed in each increment. The remainder of each hole was drilled in 10-foot increments, again with one test in each increment. Probes 36-inches and 18-inches long were available, but most tests were performed with the short probe so that the probe could be positioned entirely within one of the three rock types. Emphasis was placed on obtaining test values for each rock as a function of depth. Testing was done immediately after drilling each increment to minimize rock disturbance around the borehole resulting from relaxation and exposure to drilling fluid.

The deformation modulus is calculated from Pressuremeter data using the following equation (Menard[144], Dixon[139], Hendron[143], et al):

$$E = \frac{\Delta P - (\Delta q_{iv})\,(K_v)}{\Delta V - (\Delta P_a)}$$

where:

E	=	deformation modulus in situ
ΔP	=	pressure change
ΔV	=	volume change
Δq_{iv}	=	term correcting for pressure required to expand rubber membrane in air
ΔP_a	=	term correcting for compressibility of the measuring system
K_v	=	constant relating borehole volume deformation to linear radial information.

Because the Pressuremeter system is comparatively flexible, its accuracy becomes questionable when the rock modulus is greater than 0.75 to 1.0 x 10 psi. The Pressuremeter tends to underestimate the modulus for high modulus materials. For this reason, Pressuremeter testing at Watts Bar was confined chiefly to the shale.

2.5.4.2.2.3.2 Birdwell Borehole Logging

Details of the Birdwell testing program are presented in Section 2.5.1.2.11.

2.5.4.2.2.4 Test Results for Menard Pressuremeter

Deformation moduli from individual tests are shown in Figures 2.5-33, 2.5-42, 2.5-52, 2.5-54, 2.5-56, and 2.5-65. These values are plotted in Figure 2.5-211 as a function of depth below the bedrock surface for rock types 1 and 2. Results of tests performed in zones of no core recovery, or of tests performed in mixed rock types, are not included in Figure 2.5-211.

The solid lines in Figure 2.5-211 show the assumed average measured E as a function of depth for rock types 1 and 2. These average lines are drawn to be conservative for both shales, but reflect the trend toward increasing modulus with depth. On the basis of a comparison between Pressuremeter moduli and moduli determined from laboratory consolidation and laboratory triaxial tests on shale, Hendron[143] recommends dividing moduli obtained by use of the Pressuremeter by 3 to estimate the actual modulus which governs the settlement of a structure on shale. This correction is designed to account for three uncertainties: (1) vertical loading by a building versus horizontal loading by the pressuremeter, (2) a drained condition during building erection versus an undrained condition during the fast Pressuremeter test, and (3) uncertainties in the exactness of Pressuremeter values. Hendron developed his correction factor of 3 from tests on horizontally bedded shale where the horizontal deformation modulus parallel to the bedding was measured by the Pressuremeter.

Commonly, this modulus is several times greater than that perpendicular to the bedding. At Watts Bar, where beds have an average dip of 30°-45° and are also highly contorted on a small scale, it is expected that vertical and horizontal moduli will be approximately equal. Thus, the first consideration for applying the correction factor is nearly eliminated. Accordingly, a correction factor of 2 is used for the shales at Watts Bar.

In Figure 2.5-211, the dashed lines are derived by dividing the curve for average measured values of E by 2. The dashed lines are assumed to be the actual in situ deformation moduli under drained conditions, and are used for the ensuing settlement calculations. For rock type 1, soft shale, the modulus increases from 5,000 to 50,000 psi in the first 60 feet of rock, and is assumed to continue at 40,000 psi below 60 feet. The modulus for rock type 2, hard shale, increases from 25,000 psi to 300,000 psi in the first 140 feet of rock and is assumed to continue at 300,000 psi below 140 feet.

2.5.4.2.2.5 Comparison of Results from Menard Pressuremeter and Birdwell 3D Sonic Logger

The dynamic moduli obtained with sonic apparatus such as the Birdwell 3D sonic logger are generally higher than static moduli such as those obtained with the Menard Pressuremeter. Dynamic moduli cannot generally be used for foundation settlement analyses without first being reduced by some empirical correction factor. For this reason, the static moduli from the pressuremeter are considered more reliable for settlement analysis, and are used exclusively to determine the settlement characteristics of the foundation at Watts Bar.

In Figure 2.5-212, the dynamic moduli and static moduli are compared at the pressuremeter test locations. As shown in the figure, the ratio (E dynamic/E static) decreases from over 50 to less than 10 with increasing modulus. Deere, Merritt, and Coon[137] show a similar decrease in (E dynamic/E static) with increasing modulus. There are at least three factors contributing to the higher dynamic moduli: (1) dynamic moduli are calculated at much lower stress levels than static moduli, (2) dynamic stresses are applied for an extremely short period of time compared to static stresses, and (3) dynamic stresses are imparted to a smaller volume of rock than static stresses.

This correlation shows that not only must dynamic moduli be reduced for use in a static settlement analysis, but also that this reduction is not a constant.

2.5.4.2.2.6 Settlement Analysis

2.5.4.2.2.6.1 Introduction

In order to estimate probable building settlements based on the Menard Pressuremeter data, Figure 2.5-211, and on data showing the distribution of rock types with depth, Figures 2.5-33, 2.5-42, 2.5-52, 2.5-54, 2.5-56, and 2.5-65, the following procedure has been uti1ized:

(1) Divide the rock into layers

(2) Calculate the increase in vertical stress at the center of each layer using charts from Newmark[147].

(3) Estimate the average deformation modulus for each layer using the borehole logs and pressuremeter data.

(4) Using the calculated stresses and moduli, determine vertical strains and deformations for each layer.

(5) Sum the deformations for all layers to obtain surface settlement.

This procedure is described in detail in the following sections, and results are presented for the six holes tested with the pressuremeter.

In addition to settlement calculations made for the six core holes tested with the pressuremeter, the following calculations are made for comparison:

(1) Settlement assuming all rock types to be type 1, soft shale, with modulus as shown by the dashed line in Figure 2.5-211.

(2) Settlement assuming all rock to be type 2, hard shale, with modulus as shown by dashed line in Figure 2.5-211.

(3) Settlement assuming all rock to be type 3, limestone, with modulus of 1 x 10^6 psi for all depth intervals.

(4) Settlement based on an average borebole with 40% type 1, 50% type 2, and 10% type 3 rock in the first 50 feet of rock; and 20% type 1, 50% type 2, and 10% type 3 rock below 50 feet into rock.

All calculations are based on the assumption that the entire semi-infinite mass under the foundation is made up of material distributed in the same fashion as that under the center of the foundation. No correction can be made for the contingency that different material may appear under the corner of the footing than at the center. However, by using this technique, settlements can be bracketed by determining the boreboles whose average moduli would produce the maximum and minimum settlements, and an estimate can be made of the maximum probable differential settlements.

2.5.4.2.2.6.2 Determination of Stresses at Depth

The distribution of stresses below a flexible circular footing can be obtained using the influence factors presented in Figure 2.5-213. These influences are derived from the Boussinesq stress equations through the use of Newmark's charts[147]. To obtain the stress increase at any depth imposed by a flexible circular foundation, the influence factor for the depth and footing size in question is multiplied by the average stress on the foundation. For example, for a 100-foot-diameter foundation loaded with 5 ksf, the vertical stress increase 100-feet below the footing is (5 ksf) (0.3) = 1.5 ksf. Calculations to determine surface settlement are carried to a depth where the stress increase is less than 10% of the applied surface stress, or approximately two times the footing diameter, as shown in Figure 2.5-213. Since large footings stress deeper material, and since the moduli for the shales increase with depth, the effective stiffness of the foundation will increase with footing size. For this reason, calculations are made for footings 10-, 50-, 100-, and 200-feet in diameter.

The average stress on the 132-foot-diameter mats under the Reactor Buildings will be approximately 5 ksf. The Auxiliary Building, on a mat detached from the Reactor Building mats, will impart an average stress of 4 to 5 ksf to the foundations, and the turbine mat foundations will be loaded to between 6 and 7 ksf. The remainder of the Turbine Building outside the turbine mats will be founded on individual column footings with foundation pressures approaching 10 ksf.

Assuming the Reactor Building to be founded below 10-feet of overburden at 120 pcf and 5-feet of shale at 160 pcf, and assuming the water table to be 20-feet below the surface, the effective stress removed from the foundation surface by excavation of overburden will be approximately:

(0.12) (20) + (0.12-0.06) (20) + (0.16-0.06) (5) = 4.1 ksf.

Since the weight of the reactor is not appreciably larger than the effective weight of removed overburden, settlement under the Reactor Building will be limited chiefly to the recovery of heave experienced during excavation. After construction, as the water table is allowed to rise to its original elevation, the effective stress applied by the reactor will be decreased to 3.4 ksf, less than the effective weight of the overburden, and settlement should cease, followed by rebound potentially as great as the previous settlement.

In order to estimate settlements during construction, a footing pressure of 5 ksf is used. These settlements can be extrapolated linearly to footings with different average pressures.

2.5.4.2.2.6.3 Computation of Average Modulus of Deformation for Each Layer

To calculate average moduli, the bedrock is divided into 5-foot layers to 60- feet, 10-foot layers to 200-feet, and 50-foot layers to 400-feet. Where boring logs are available (the upper 50-feet or rock in holes 29, 41, 43, and 52, and the upper 150-feet of rock in holes 20 and 39), actual percentages of each rock type are used to calculate the average modulus for each 5- or 10-foot layer. For holes where no core was taken for the interval between 50- and 150-feet into rock, average percentages for this depth interval in holes 20 and 39 are used; 20% type 1, 70% type 2, and 10% type 3 rock. These percentages are assumed to continue below 150-feet into rock for all holes.

The average modulus for each depth interval is found by converting the equation for adding springs in series to:

$$E_{ia} = \frac{100}{\frac{P_1}{E_i} + \frac{P_2}{E_2} + \frac{P_3}{E_3}}$$

where:

E_{ia} = average E for 5-, 10-, or 50-foot-depth interval in question

P_1 = actual or estimated percent type 1 rock in interval

P_2 = actual or estimated percent type 2 rock in interval

P_3 = actual or estimated percent type 3 rock in interval

E_1 = modulus for type 1 rock at center of interval, taken from dashed line for type 1 rock in Figure 2.5-211

E_2 = modulus for type 2 rock at center of interval, taken from dashed line for type 2 rock in Figure 2.5-211.

E_3 = assumed modulus for limestone, 1 x 10^6 psi for all depth intervals.

The average moduli for each depth interval are shown in Figure 2.5-214 for zones where core was taken and actual percentages of each rock type are known.

2.5.4.2.2.6.4 Determination of Surface Settlement

The settlement for each depth interval is obtained using:

$$S_i = \frac{\sigma_i t_i}{E_{ia}}$$

where:

σ_i = stress at the center of depth interval i, calculated using Figure 2.5-213
t_i = thickness of depth interval i

Total surface settlement, S_s, at the center of each flexible circular footing is obtained by summing all s below the footing.

For comparison, the edge of a flexible footing will settle approximately two-thirds of the center settlement[152]. A rigid footing will settle approximately 0.8 times the center settlement of a flexible circular footing[145].

Whether or not a slab is rigid or flexible, according to De Simone[138], is a function of the moment of inertia of the slab, the relative stiffnesses of the slab material and foundation rock, and width of the slab. Using De Simone's relationship and moduli for the average hole, and assuming that the structure itself adds no rigidity to the foundation slab, the following slab thicknesses are required in order to be considered rigid:

Slab Diameter Feet	Thickness to be Considered Rigid Feet
10	1.5
50	5.7
100	10.3
200	18.3

These thicknesses are conservative since the walls within a structure such as nuclear plant will increase rigidity appreciably.

Once the total settlement is obtained, an average effective deformation modulus can be calculated for each hole using:

$$E = \frac{2qr\,(1 - v^2)}{S_s}$$

where:

E = average deformation modulus which would produce calculated surface settlement
q = foundation pressure
r = foundation radius
v = Poisson's ratio, assumed 0.20
S_s = total surface settlement at center of flexible footing

Surface settlements and deformation moduli are presented for each set of calculations in Table 2.5-13 and settlements are compared in Figure 2.5-215.

For the six holes for which settlements have been calculated, settlements are between the values calculated by assuming that all rock is either type 1 or type 2. S ranges from 0.11-inches for a 10-foot-diameter footing on hole 43, to 1.18-inches for a 200-foot-diameter footing on hole 29. E for the average hole ranged from 17,000 psi for a 10-foot-diameter footing to 83,000 psi for a 200-foot-diameter footing.

2.5.4.2.2.6.5 Determination of Site Uniformity

The same method of settlement analysis could be employed to calculate settlements and average moduli for the 28 other core holes in the immediate plant area. However, Figure 2.5-216 allows easy empirical extrapolation of information from the six pressuremeter holes to the remaining holes in the plant area. This extrapolation has been developed for the 10-foot-diameter footings only, because the small footings are most sensitive to the presence of soft shale near the surface of the bedrock. A similar extrapolation could be developed for larger footings.

In Figure 2.5-216, it is assumed that type 2 rock, hard shale, is the average rock type at the plant site. It is further assumed that equal parts of type 1 rock and type 3 rock do not change the modulus from that if all rock were type 2. It follows then that the average modulus should be a function of

(% type 1 rock) - (% type 3 rock)

for the depth interval receiving the most stress from the footing. In Figure 2.5-216, this function is plotted versus deformation modulus for the 20-foot-depth interval below 10-foot diameter footings. All holes tested with the pressuremeter except N-65 follow the well-established trend. The curve showing all type 1, type 2, or type 3 rock is also presented for comparison.

Using the boring legs and the relationship shown in Figure 2.5-216, the average deformation moduli for the 10-foot-diameter footings for all holes in the plant ares have been estimated and are plotted in Figure 2.5-217. Except for a higher modulus zone of rock delineated by the dashed lines through the Auxiliary Building, conditions appear to be uniform across the site.

2.5.4.2.2.6.6 Improvement of Foundation Uniformity by Removing to 10-Feet of Surface Rock

Because a large part of the variation between holes shown in Figure 2.5-217 arises from soft shale near the bedrock surface where it has the most influence on settlement, it is expected that with the removal of 5- to 10- feet of surface rock the moduli in Figure 2.5-217 will become even more uniform. To partially verify this effect, detailed settlement calculations were made for the pressuremeter boles assuming 10-foot-diameter footings founded 10-feet into rock. Table 2.5-14 shows that: (1) the magnitude of average settlements is halved, and that (2) differential settlement due to varying rock quality is decreased by a factor of 5 with removal of the upper 10-feet of rock.

Since a major part of the plant is to be founded 5- to 10-feet into rock, the advantages of removing the upper rock should be fully realized.

2.5.4.2.2.6.7 Bearing Capacity

An estimate of the minimum bearing capacity of the shale can be made using the following assumptions:

(1) All rock is type 1.

(2) Footing has no surcharge (center of excavation).

(3) Factor of safety of 3 against bearing capacity failure.

(4) 0 = 25° for the shale.

(5) E = 10,000 psi.

(6) Previous studies[136] show the unconfined compressive strength of shale to be approximately E/100. E/200 = 50 psi is assumed to account for the slicken-sided and contorted nature of the shale.

With these assumptions, the allowable bearing capacity for a footing 10-feet in diameter using the relationship shown by Terazghi and Peck[151] is 26 ksf. This value is greater than any anticipated loads at the Watts Bar Nuclear Plant. Bearing capacity will be governed by tolerable differential settlements.

2.5.4.2.2.7 Behavior of Watts Bar Lock

2.5.4.2.2.7.1 Moduli Calculated from Lock Settlement

Watts Bar Dam, constructed in 1939, is situated less than 2 miles upstream from the present nuclear plant site. The left end of the dam and the lock are founded on the same material which will form the foundation for the nuclear plant. During construction, a number of settlement points were placed on various blocks within the lock, and settlements were monitored for the period between April 1940 and June 1942. Most of the lock was placed within an original water course, and less than 5-feet of alluvium and 5-feet of rock were excavated to foundation grade. A simplified plan of the lock foundation is shown in Figure 2.5-218, and a typical settlement curve is shown in Figure 2.5-219.

With known settlements and foundation load distributions, the deformation moduli shown in Figure 2.5-218 have been calculated using Newmark's chart for vertical displacements[146] which assumes a perfectly flexible foundation. This assumption is most valid for point E, for which a large part of the settlement can be attributed to the adjacent fill. The effective modules at point E should be compared with that of a footing 100- to 200-feet in diameter at the nuclear plant. The modulus of deformation at point

E, 99,000 psi, is equal to the highest modulus hole under a 200-foot-diameter footing at the nuclear plant, indicating that the assumptions made concerning the modulus of deformation for deep rock at the nuclear plant are slightly conservative.

For points A through D on the upper guard well, moduli range from 23,500 psi to 89,000 psi, with the two higher moduli on the lock side of each block. Since the blocks are high with respect to their base size, and since they are triangular in shape with the heaviest load on the lock side of the foundation, additional calculations have been made assuming them to be rigid with rotation about their centers of gravity. The resulting moduli, 43,000 and 49,000 psi, should be compared with moduli calculated for footings at the nuclear plant approximately 25-feet in diameter and founded 5-feet into shale. From Table 2.5-14, the average E for a 10-foot footing founded 10-feet into rock is 33,000 psi. Since the modulus for a larger footing would be greater, the calculated moduli at the lock and at the nuclear plant appear to agree perfectly.

The calculations based on lock settlement verify the validity of assumptions and techniques used to estimate deformation moduli at the nuclear plant, settlement of the lock serves as a large scale foundation test for the nuclear plant foundation.

2.5.4.2.2.7.2 Settlement of Lock and Nuclear Plant as a Function of Time

Figure 2.5-219 shows settlement of block R-10 (point F in Figure 2.5-218) with respect to three important events: (1) completion of construction of the block, (2) flooding of the cofferdam, and (3) the start of reservoir filling. By August 1940, when the block was completed, approximately 65% of the total settlement had taken place. Eight months later, settlements were nearly complete. As the cofferdam was flooded, reducing the effective stress on the block foundation, settlement ceased, and one-third of the previous settlement was recovered as heave during the next six months. Between April 1941 and January 1942, the cyclic nature of the settlement curve is most likely response to the lake level behind Chickamauga Dam.

The average foundation stress at block R-10 was approximately 6.5 ksf before the cofferdam was flooded, a stress similar to those expected at the nuclear plant. However, at the lock the effective weight of removed overburden was only 0.8 ksf as opposed to 4.1 ksf at the nuclear plant. Because of this difference, it is expected that an even greater percentage of total settlement will be realized during construction of the nuclear plant.

Settlement should cease as the water table is allowed to retain its original elevation around the nuclear plant, and as shown at the lock, heave can be expected at this time as effective stresses on the foundation are reduced.

Since two periods of differential movement are expected during construction, and during raising of the water table around the plant installation of utility lines passing between buildings was delayed as long as possible during the plant construction to allow the water table to almost return to the original ground water level around the plant.

2.5.4.2.2.8 Excavation Experience in the Rutledge Shale at Watts Bar Lock

Exploratory holes were drilled in the area of the Watts Bar Dam navigation lock by TVA and Corps of Engineers personnel. The following remarks[150] relate the experience during drilling and subsequent excavation.

> . . . Most of these holes penetrated dark gray sandy fissile shale with thin layers of interbedded dense gray sandstone. The core recovery was poor, leaving only the hard sandy shale and thin layer of sandstone after the soft fissile shale was washed away by the drilling operation. The cores indicated a rather level rock formation with little weathering at the surface.
>
> After the foundation excavation was started the true nature of the rock surface was revealed. The shale was soft and weathered for a foot or two below the surface, and below this limit the shale was consistently uniform in character although the dip of the beds varied between wide limits. The shale was somewhat harder than was at first expected and disintegrated very little on exposure to the weather. It proved practical to dig the shale with a power shovel without blasting.

At the powerhouse where blasting was required, the perimeter of the area was line drilled prior to blasting to minimize damage to adjacent rock.

Core drilling experiences at the lock have been duplicated at the nuclear plant site except that core loss decreased with depth and the 'sandstone' at the lock site has been reinterpreted to be a glauconitic limestone. It is expected that excavation conditions at the nuclear plant will duplicate those at the lock.

During excavation, it was found that weathered shale could be identified as being brown, rusty, and 'rotten,' while fresh shale was dark gray to black. The rock under the lock is described as having hundreds of small, sharp, overtured folds plunging to the northeast. It was found that the small-scale folds 'strengthened the shale against sliding, but they always made hand scaling difficult for it was impossible to follow any one bed.'

Information from the 'Final Geologic Report of the Watts Bar Project' by P. P. Fox[140] states:

> To obtain a satisfactory surface on the shales under the lock, a saw-toothed surface was cut into the shale, after an attempt to scale blocks R-10, L-2, and R-2 as a flat surface. This method proved to be much faster and better than any other tried. On the flat surfaces innumerable small, partially detached, loose, and fragile particles of shale existed in spite of all care in scaling, but by the notched method the hard beds could be exposed on the top of the benches and the softer shale left undisturbed in the nearly vertical faces.

TVA Technical Report No. 9[150] describes the excavation operation as follows:

> . . . Power shovels excavated to within 6 inches of the neat line and grade for the lock walls. The final excavation was performed by hand picks and pneumatic tools approximately 24 hours before the placing of concrete. The final scaling left the bedrock stepped or with a saw-tooth relief, the more horizontal areas cut to a plant along the more durable sandstone strata. Experience proved that the rock did not disintegrate as rapidly as expected and that final scaling of only 3 inches from walls and floors, where no truck traffic was expected, would be sufficient . . .

Several specimens of shale from the clear plant site were tested with a newly developed slaking durability test in the University of Illinois Rock Mechanics Laboratory[142]. The test is based on the percent of an oven-dry specimen retained in a 2-mm mesh drum after 10 minutes of rotation. This value is compared with the plasticity index of the shale. The test is sensitive to the shale's ability to withstand stress relief and cyclic wetting and drying. Since the shales at Watts Bar are nonplastic, cyclic wetting and drying is not of chief concern because it does not cause appreciable swelling and shrinkage. However, the percent retained in the durability test was 30% for type 1 shale and 80% for type 2 shale. From past experience, durability becomes a matter of concern when the percent retained falls below 95%. The low durability of the Watts Bar shale is attributed to its slicken-sided and contorted nature, which makes it susceptible to deterioration upon stress relief.

The results of the durability test are borne out in observations made by Fox[141] during construction of Watts Bar lock. Upon examination of the lock foundation, Mr. Fox expressed the opinion that wetting and drying was not the chief source of deterioration in the shale, but that stress relief causing parting along preexisting planes of weakness, such as joints and slickensided surfaces, was most responsible for deterioration. Mr. Fox thought that final scaling should be followed in less than two hours by placement of the first lift of concrete. As experience was gained, it was found that stress relief was not a problem if the first pour came within 24 hours of scaling.

2.5.4.2.2.9 Evaluation of Settlement

2.5.4.2.2.9.1 Initial Settlement Monitoring

The initial program to monitor settlement began in October 1973 to record structural movements during construction. The location of the settlement stations are provided in Figures 3.8.4-66 and 3.8.4-67. Figure 2.5-585 shows details of a typical settlement monument. In general, the monuments were read monthly for selected monuments and terminated for the rest. The recorded settlement data are given in Tables 2.5-67 through 2.5-70. The differential settlement readings between rock-supported structures are provided in Table 2.5-71. During the course of construction several settlement stations became inaccessible (and are so labeled) either because they were physically buried or were impossible or extremely difficult to reach. All accessible settlement stations were last read during December 1981 and January 1982 when three additional surveys were run. During each of these surveys all accessible stations were read except for locations SS-4, -5, -6, -7, -13, -14, -15, and -16. These eight were read once in January 1982 and again in a special survey during February 1982.

All eight are in highly congested areas and were previously labeled as inaccessible. These stations were only accessible using specially modified level tripods and survey rods. Six of these eight stations are in the annulus.

During construction, several monuments were relocated for various reasons. Monuments 1 and 2 had associated relocated Monuments 1A, 1B, and 2A. These and their corresponding old monuments were all read for a sufficiently long period that they constitute essentially separate data. The Diesel Generator Building monuments were relocated from the inside to the outside of the building in April 1980. In excess of 7 years' data are available for the interior monuments. Other monuments were relocated several months after original installation and were subsequently read for periods of more than 7 years. These are not identified by a letter (1A, 2B, etc), but the relocation dates are given in Tables 2.5-69 and 2.5-70 and the monuments are identified as reset. Their settlements prior to reset are small and are not carried forward; in effect truncating several months of earlier records. This is done for two reasons. First, the total duration of record (over 7 years) is large compared to the several months truncated. Second, the magnitudes of the "measured settlements" at the time of reset are small and generally less than the apparent random fluctuations of the data. Absolute magnitude of error allowed in typical surveys of these monuments is on the order of from 0.01- to 0.06-feet depending on the monument and length of run. However, the actual error of closure for the surveys was generally less than 0.01-feet.

2.5.4.2.2.9.2 Evaluation of the Program

All rock supported Category I structures were designed for total settlement of 1-to 2-inches and differential settlements of 1-inch. The settlement of rock supported structures was not deemed to control the design of the building. Table 2.5-72 provides the design settlement (total and differential), the maximum recorded settlement, and the current settlement measurement for each rock supported structure. The maximum recorded settlement shown in Table 2.5-72 is for a single monthly reading with both preceding and following readings indicating smaller settlement. The current settlement reading is the average of the three readings from December 1981 to January 1982 unless otherwise noted.

The time versus settlement plots of Unit 1 and 2 Reactor Buildings, shown in Figures 2.5-586 and 2.5-587, respectively, reflect the latest reliable data available. Readings were discontinued June 1978 because settlement stations became inaccessible. The survey data of January and February 1982 were not used for the plots of the Units 1 and 2 Reactor Building because the surveyors experienced considerable difficulty in reaching the settlement stations in the annulus. However, the current data is presented in Table 2.5-70. For the Unit 1 Reactor Building, the maximum and minimum settlement stations were inaccessible after March 1978, and October 1977, respectively. The Unit 2 Reactor Building settlement was discontinued in June 1978 because the settlement stations became inaccessible.

Updated time versus settlement plots are provided in Figures 2.5-588 and 2.5-589 for the Auxiliary Control Building, the Diesel Generator Building, and the Intake Pumping Station.

The measured settlements have not approached the design criteria of 1-inch of differential settlement between buildings or 1- to 2-inches of total settlement with respect to the surrounding area. In general the maximum settlement of rock-supported structures had occurred by 1977, and thereafter the settlements have been stable. The maximum settlement of 0.056-feet (0.67-inches) was recorded April 1980. The maximum differential settlement of 0.038-feet between the Reactor Building Unit 1 and the Auxiliary Building was recorded August 3, 1977. The measured differential settlement of 0.060-feet, August 1977, between settlement stations (SS) 18 and 23 was judged to be a measurement error for three reasons. First, the differential settlements one month before and after were recorded to be 0.018-feet and 0.024-feet, respectively. Second, the latest reading between SS18 and SS23 was recorded to be 0.023-feet of differential settlement. Third, the maximum settlement recorded a year before and after the error was 0.036-feet between SS18 and SS23.

TVA has fulfilled the commitment of monitoring rock-supported structures since the structure loading is essentially complete on all rock-supported buildings, and all the total and differential settlements are well within the design criteria allowables. Settlement readings will no longer be reported for rock-supported structures.

The settlement monitoring of the soil-supported Category I structures include the Diesel Generator Building and the waste packaging area. Table 2.5-72 provides the design settlement (total and differential), the maximum recorded settlement, and the current settlement measurement for these structures. The Diesel Generator Building is founded on compacted 1032 crushed stone (an engineered granular fill) underlain by basal gravel and bedrock. This foundation was deemed to have negligible settlement or differential settlement. The Diesel Generator Building as laid out and designed is not controlled by settlement. The connections were designed to allow for a differential settlement of 3-inches for the ERCW piping, 3/4-inch for expansion/deflection fittings on electrical conduits, and 1-inch between the cable trays and rigid conduits in conduit banks (electrical conduits).

The design of the waste packaging area was similar to that of the Diesel Generator Building. The settlement of the building was taken into account when designing the connections between the buildings. The waste packaging area does not have connecting ERCW pipes, other Category I pipes, or Category I electrical conduits. The waste packaging area was designed for 1-inch of differential settlement between the auxiliary building and itself.

Based on our evaluation, the total and differential settlements are not significant; there are no trends being exhibited; there has been no adverse structural performance; and there are no anticipated problems from the settlement of Category I structures.

2.5.4.2.2.9.3 Differential Settlement Not Incorporated in Design Criteria

The design 1-inch differential settlement between adjacent rock-supported structures was not incorporated into the design of piping and electrical components passing between adjacent rock-supported structures. The affected items pass between the Unit 1 Reactor Building and Auxiliary Building and between the Unit 2 Reactor Building and the Auxiliary Building. This design deficiency is covered in NCR WBNCEB8108.

The effect of the failure to include the 1-inch differential settlement between adjacent rock-supported structures would be limited to HVAC duct, cable trays, Category I piping, instrument lines, and conduit (plus their related supports) which pass between adjacent buildings. Through evaluation TVA has determined that all such HVAC duct, cable trays, and their supports can withstand a 1-inch settlement as is. TVA has also determined by analysis of settlement data on all Category I structures in the main plant area that the differential settlement of adjacent structures would not be 1-inch, but rather the maximum differential would be less than 1/2-inch. (This 1/2-inch figure is based on settlement which occurred in 1976 and early 1977 which is before the great majority of utility lines were installed.) The analysis also demonstrates that after 1982 additional settlement will be less than 1/4-inch.

By the engineering judgment of TVA design personnel, the conservatism inherent in the design of the plant is sufficient to accept the effects of this settlement on Category I piping, conduit, and instrumentation lines without causing line failure or adversely affecting safe operation of the plant.

To confirm this analysis, a system for monitoring future differential settlement was developed.

2.5.4.2.2.9.4 Monitoring Program for Differential Movement

Instrumentation for monitoring future differential settlement has been designed and installed. Details of the relative movement detectors (RMDS) are shown in Figure 2.5-590. Their locations are shown on Figure 3.8.4-66. Settlement was monitored until January 1984. At that time it was determined that future settlement would be insignificant. TVA memo on settlement stations dated February 6, 1984 stated that the settlement monitoring program was no longer needed. TVA calculation WCG-1-861 Settlement Monitoring provides further justification for this determination. On that basis, monitoring of differential settlement has been discontinued and is not required at WBN.

2.5.4.3 Exploration

The relationship between Category I foundations and the in situ soil or fill materials are described in the following sections:

In Situ Soil Investigations	Section 2.5.4.2.1
Borrow Investigations	Section 2.5.4.5
Excavation and Backfill	Section 2.5.4.5

The corresponding information with regard to rock is found in Section 2.5.1.2.6.

2.5.4.4 Geophysical Surveys

2.5.4.4.1 Rock Characteristics

The rock characteristics have been discussed in Sections 2.5.1.2.7 and 2.5.4.2.2. with regard to dynamic moduli.

2.5.4.4.2 Soil Characteristics

In situ soil dynamic studies were made at the Watts Bar site to obtain data for computation of elastic moduli for earthquake design criteria. Tests consisted of the following:

(1) Down-hole seismic surveys for 4 stations in the intake channel, for 1 station for the Diesel Generator Building, and for 25 stations for the Class 1E conduit and ERCW piping alignments.

(2) Seismic refraction surveys along two lines in the Diesel Generator Building and along four lines in the intake channel.

2.5.4.4.2.1 Equipment

(1) Intake Channel and Diesel Generator Building

The equipment used to record the time arrivals for compressional and shear wave velocities was a Bison signal enhancement seismograph, Model 1570B, a Bison strip chart recorder, a Hall-Sears MP-4 pressure type geophone, and a 8-hz Mark Products geophone.

(2) Class 1E Conduits and ERCW Piping Alignments

The equipment used for recording seismic waveforms through the soil consisted of a Bison Instruments signal enhancement seismograph, model 1575, strip chart seismic recorder and blaster, and a Hall-Sears MP-4 hydrophone.

2.5.4.4.2.2 Velocity Measurement Procedures

(1) Intake Channel and Diesel Generator Building

The seismic refraction surveys were made by placing a geophone at the end of a traverse line and generating a signal by hitting a steel plate with a sledge hammer at various measured distances from the geophone. The refraction lines were surveyed in both directions and the results averaged. The seismic down-hole surveys were made by placing a geophone down a cased hole near the top of rock and the energy (usually a cap plus a few inches of primacord) was detonated on the surface 20 feet from the top of the hole. Down-hole seismic measurements were made in four directions from the hole.

(2) Class 1E Conduits and ERCW Piping Alignments

Figure 2.5-227 shows a plan view and vertical section of a typical test configuration for an in situ soil dynamic property measurement. The soil test borehole was cased with a vinyl tube, capped on the lower end to prevent water from leaking into the surrounding soil. The hydrophone was lowered into the test hole to a depth such that it was both immersed in the water and not touching the bottom of the hole. The depth (z) in Figure 2.5-227 is the difference in elevation between the hydrophone center and the point of sonic energy application. Seismic waves were generated either by striking a steel plate with a sledgehammer or by exploding two feet of primacord about one foot below ground. For each borehole, either explosives or sledgehammer was used as the single type of energy source for the survey as soil sonic attenuation conditions required. That is, a single type of energy was used for the entire sequence of measurements at each hole.

The detection of shear waves by a hydrophone in a water-filled borehole has been shown to be most effective when the vertical shear wave component is incident at about 45 degrees to the hole axis (Reference 171). Therefore, the point of energy application on the surface was a distance (x) from the hole axis equal to the hydrophone depth (z) below surface. Four independent seismic waveforms were recorded for each borehole with the energy source locations placed in a 90-degree array at a horizontal distance (x) from the hole axis and oriented north, south, east, and west of the hole, as shown in Figure 2.5-227. Where this orientation was not possible, the whole array was rotated about the borehole axis by the angle φ, also shown in Figure 2.5-227. In all cases the time trace of the seismic wave form was recorded for later careful analysis.

2.5.4.4.2.3 Data Analysis and Results

(1) Intake Channel and Diesel Generator Building

In Situ Soil Dynamic Results--Compressional wave velocities were obtained from down-hole and refraction surveys. Shear wave velocities were measured from a few of the seismic refraction traverses, but there was no attempt to measure them by the down-hole survey. When no shear wave measurements were made, an assumed Poisson's ratio was used to calculate shear wave velocities.

An assumed Poisson's ratio of 0.45 was used for all downhole measurements, and Poisson's ratios of 0.35 to 0.46 were used for the various compressional velocity zones along the six seismic refraction traverses. The saturated soils gave higher compressional velocity values than those above the water table. Therefore, it was assumed that a high water content was the primary reason for the higher values. A Poisson's ratio of 0.45 or greater was used below the water table to offset the high compressional velocities so that more realistic shear velocities could be calculated.

The average results from the in situ dynamic program are given in Tables 2.5-15 and 2.5-16. Details pertinent to the program and detailed results are given in Figures 2.5-228 through 2.5-233.

(2) Class 1E Conduits and ERCW Piping Alignments

Data analysis was performed by measuring time intervals from detonation to geophone arrival for the compression wave (P-wave) and, where possible, for the shear wave (S-wave). The P-wave arrival is identified as the first deviation of the linear time trace. The S-wave is identified by any change in amplitude, frequency or both of the sinusoidal compressional wave at approximately twice (1.99 - 3.67) the first arrival time of the compressional wave. Shear wave first arrival could not be identified on all records. Only those records identified were used in the analysis. Straight line propagation paths were assumed for all analyses. The density value used for the analysis was the mean of those obtained by laboratory testing of soil samples. Converting travel time measurements to dynamic moduli values was performed on a Hewlett-Packard programmable calculator using programs specifically written for this purpose. The equations relating seismic velocities to dynamic moduli may be found in any standard text on geophysics. Statistical analysis of the data was performed in a similar manner. Data acquisition and reduction accuracy, though performed by standard practices, are very near 'state-of-the-art.' Efforts are still continuing toward developing improved methods of shear wave generation and detection.

In situ soil dynamic moduli and statistical analysis results are presented in Table 2.5-17 for the Watts Bar site.

The fact that the mean hydrophone depth (z) is the difference in elevation between the center of the hydrophone and the elevation of the points of energy application should be noted. This fact explains why in some cases the depth (z) could be greater than the depth of refusal. The cases where it is several feet less may be attributed to obstructions in the vinyl borehole casing.

Large standard deviations in the compressional and shear velocities may be attributed either to soil velocity anisotropism or to difficulty in recognizing wave arrival times either because of noise or wave attenuation.

2.5.4.4.2.4 Data Analysis and Results - Evaluation Seismic Criteria and New Design/Modification Seismic Criteria (Set B and Set B + C)

The dynamic soil property results used in the seismic analyses were reevaluated using current technology to more precisely reflect their elevation and plant variations. Separate layered soil profile were established for WBN soil-supported structures (Diesel Generator Building, Additional Diesel Generator Building and Refueling Water Storage Tank) and for the North Steam Valve Room. The dynamic soil properties used in the soil-structure interaction analyses of these structures are summarized in Tables

2.5-17A through 2.5-17D. Figures 2.5-233A through 2.5-233K referenced in these tables define the strain-dependent soil shear modulus and strain-dependent soil damping ratio.

2.5.4.5 Excavations and Backfill

2.5.4.5.1 Earthfill

The term 'earthfill' refers to soil which is obtained from onsite borrow areas and compacted in multiple lifts to form a fill meeting specified standards.

2.5.4.5.1.1 Investigation

Investigations for borrow centered in two major areas; (1) the area that required excavations for the structures in the main plant area, and (2) onsite borrow areas. The areas investigated for borrow are shown on Figures 2.5-220 for the main plant area, and Figures 2.5-221 and 2.5-221a for the onsite borrow areas. The onsite borrow areas are located on a topographic map in the upper right corner of the figure, with each individual borrow area shown in more detail elsewhere on the figure.

The usual method of sampling consisted of continuous augering to plant finish grade or to top of bedrock or to the ground water level, depending on the design requirements. The soil recovered from the auger borings for laboratory testing was placed in plastic bags. In some areas test pits were excavated to obtain bag samples. Representative samples of each borrow soil type were sealed in glass jars immediately upon removal from the auger boring or test pit for laboratory index testing.

The soils material from the borrow investigation was tested by family type groups. Soil classes were selected for each area upon completion of the compaction tests. To determine the borrow soil characteristics and to aid in establishing the soil classes in the borrow areas, the following index tests were performed on each typical borrow soil:

(1) Atterberg limits (ASTM D 423 and D 424)

(2) Grain size (ASTM D 422)

(3) Classification (ASTM D 2487)

(4) Standard compaction tests (ASTM D 698)

On earthfill selected to be used for construction of qualified fills, the following additional tests were performed as needed on each soil class in order to establish design properties.

(1) Unconsolidated - Undrained (Q) Shear Strength (ASTM D 2850).

(2) Consolidated - Undrained (R) Shear Strength (Standardized TVA Procedure).

(3) Consolidated - Drained (S) Shear Strength (ASTM D 3080).

The Q test specimens were remolded at 3% above optimum moisture content. The R and S test specimens were remolded at 3% below optimum moisture content. S shear tests were conducted using a direct shear machine.

2.5.4.5.1.2 Test Results

The results of the soil classification are shown on the graphic logs in Figure 2.5-222 for the main plant areas, and in Figures 2.5-223, 2.5-224, 2.5-260 through 2.5-270 and 2.5-377 through 2.5-519 for the onsite borrow areas. Auger borings for borrow are identified with the prefix 'PAH.' Tables 2.5-18, 2.5-19, 2.5-19a and 2.5-45 through 2.5-53 summarize the results obtained from the borrow investigations.

Main Plant Area

Initially, all samples obtained in this investigation were visually grouped and classified according to the Unified Soil Classification System. Six major soil types, MH, ML, CL, SM, SC, SM-SC, are present in cut areas. These materials were then regrouped according to their index properties and subjected to standard compaction testing. As shown on the family of curves (see Figure 2.5-235), three main classes were established:

(1) Class I, representing 11% of the total, classified clayey sand (SC) with an optimum moisture of 13.6% and a maximum density of 116.3 pcf. Soils represented by this class had an average moisture content of 22.3, or 8.7% above optimum.

(2) Class II accounted for 54% and classified sandy, lean clay (CL) with an optimum moisture content of 17.9% and a maximum density of 101.1 pcf. Soils represented by this class had an average moisture content of 23.3, or 54% above optimum.

(3) Class III amounted to 35% and classified sandy, plastic silt (MH) with an optimum moisture content of 21.8% and a maximum density of 101.1 pcf. Soils represented by this class had an average moisture content of 24.5, or 2.7% above optimum.

In summary, this investigation in the main plant area has established that borrow soils of a satisfactory type, but of relatively high moisture content, are present in cut areas at the site. Materials of acceptable moisture content were present in the transformer yard, the 500-kV switchyard, and cooling tower areas. Materials in the Reactor Building, Plant Building, and intake channel areas required drying before using these soils as fill.

The borrow soil types identified in the intake channel were remolded and tested for shear strength. The results are shown in Table 2.5-21. The results are shown in graphical form (Figure 2.5-251). As indicated by the graphical plot (Figure 2.5-251) the value selected for design is conservative.

Onsite Borrow Areas

The onsite borrow areas which are shown on Figures 2.5-221 and 2.5-221a were completed after the completion of the borrow investigation in the main plant area described above. The results of the borrow investigation are described in relationship to the results obtained from the borrow investigation in the main plant area.

Area 1, which covers approximately 13 acres, lies about 2,500 feet east of the main plant area. The soil profile, as established by the eight auger borings drilled in this area, consists of 8 to 13 feet of reddish-brown silt (ML and MH), underlain by 7 to 16 feet of brown lean clay (CL) (see Figure 2.5-223). The majority of these fine-grained essentially impervious soils have natural moisture contents near the plastic limit. This area will supply about 280,000 cubic yards of borrow material.

Area 2 is located about 3,700 feet west-northwest of the main plant site and covers about 8 acres. The predominant soil type encountered in this area is a lean clay (CL) (see Figure 2.5-223). Some silt of medium plasticity (MH) and a small amount of fat clay (CH) are also present. As in area 1, the upper soils are reddish brown to a depth of 4 to 7 feet with the underlying soils colored brown. This area will supply about 170,000 cubic yards of borrow.

In area 3, two borings were drilled and reddish brown to brown lean clay (CL) was encountered. (See Figure 2.5-223). Secondary soils are lean silt (ML) and clayey gravel (GC) with the gravel consisting of subrounded quartizite. This area is located about 3,600 feet west-southwest of the power plant and extends over 3 acres. About 50,000 cubic yards of fill will be available from area 3.

In area 4, located southwest of the plant and covering approximately 6 acres, seven borings were drilled and alluvial sandy lean clay (CL) was encountered (see Figure 2.5-224). Secondary soils are silty sand (SM) and sandy silt (ML) which are slightly micaceous. This area, located southwest of the power plant, covers about 100,000 cubic yards of borrow.

Area 7 covers approximately 12 acres and is borings located southwest of the main plant area. Eleven borings were drilled and the predominant soil encountered was a lean clay (CL). Secondary soils encountered were a lean clayey-silt (CL-ML) and a lean silt (ML). Minor quantities of a fat clayey-silt (CH-MH) and a fat silt (MH) were also encountered. Approximately 145,000 cubic yards of material is available from the area.

Subsoils in the areas are similar in texture and plasticity to the borrow soils determined in the soil investigation previously reported. Soil properties are listed in Table 2.5-19. The natural moisture contents are from 2 to 8% above optimum. Close moisture control during placement will be required to assure adequate compaction. In summarizing, these additional borrow sources at Watts Bar Nuclear Plant will yield approximately 0.7 million cubic yards of impervious fill with satisfactory characteristics.

Borrow area number 4 was selected as a source for any soil necessary for the construction of qualified fills. The area was selected based on; (1) the quantity of borrow material available, and (2) the information provided on the graphic logs. Table

2.5-25 presents laboratory test data on the borrow classes available in borrow area 4. The strength values used for design are shown in Table 2.5-12. The results for each type shear test are plotted in graphical form (Figures 2.5-244 through 2.5-246), and a conservative value below the average for c and φ is selected for use in the design. The values used for design (Table 2.5-12) are low averages for the strength data shown in Table 2.5-25.

Due to the need to construct the underground barrier trenches to resolve the issue of potentially liquefiable soils along portions of the ERCW piping and 1E conduit alignments, several additional onsite borrow areas were investigated for use as safety-related fill. The additional areas are shown on Figures 2.5-220, 2.5-221, and 2.5-221a. These areas are identified as Trench A, Trench B, Areas 9, 10, 11, 12, 13, and 2c, and the future 161-kV switchyard. The central laboratory investigated each of these areas and developed moisture-density compaction curves (ASTM D 698) for each area. The testing identified several soil classes for each area. The laboratory strength testing consisted of consolidated undrained (R) shear tests on each soil class. Samples were molded to 95% of maximum dry density (ASTM D 698) and 3% below optimum moisture content. All samples were subsequently saturated prior to shearing. Due to the desire for a higher design cohesion, borrow classes with a cohesion intercept (c) less than 0.2 tons/ft^2 were retested at a higher density. These samples were remolded to 100% of maximum dry density (ASTM D 698) and 3% below optimum moisture content. All samples were saturated prior to shearing. The test results for each borrow area are shown on Tables 2.5-45 through 2.5-53. The results of this testing were evaluated to provide soil properties to use in the design and analysis of the underground barrier trenches.

The backfill used for Trench A came from borrow areas Trench A, 9, 10, 2c, and the future 161-kV switchyard. Thus, materials from these areas were evaluated for the Trench A design soil properties. Since two different degrees of compaction were used in Trench A, separate evaluations were made. The first evaluation, shown on Figure 2.5-520, was for Earthfill A which was placed at 95% of maximum dry density, and the second evaluation, shown on Figure 2.5-521, was for Earthfill A1 which was placed at 100% of maximum dry density. In the second evaluation, the data for sands was deleted from the evaluation, since only fine-grained soils were used for Earthfill A1.

The backfill used for Trench B came from borrow areas Trench B, 12, 2c, 13, and the future 161-kV switchyard. Thus, materials from those areas were evaluated for the Trench B design soil properties. Since two different degrees of compaction were also used in Trench B, separate evaluations were made. The first evaluation, shown on Figure 2.5-522, was for Earthfill A which was placed at 95% of maximum dry density, and the second evaluation, shown on Figure 2.5-523, was for Earthfill A1 which was placed at 100% of maximum dry density. In the second evaluation, the data for sands was deleted from the evaluation since only fine-grained soils were used for Earthfill A1. Figure 2.5-583 provides a summary of the above borrow evaluations.

2.5.4.5.1.3 Field Work

Prior to construction, the central laboratory prepared a family of compaction curves for all soil classes are the site (see Figures 2.5-235, 2.5-271, 2.5-524 through 2.5-533). The soil classes were further divided into subclasses for use by the inspectors of backfill placing and the project laboratory for construction control and day-to-day testing of fill compaction. These tests were performed by the project laboratory and were for dry density, moisture content, and degree of compaction. A minimum of at least one set of tests for each 2,000 cubic yards of fill placed was performed throughout the course of the work. Additional sampling and testing were done as required by the inspectors or engineers in charge.

The quality of the backfill was documented by measuring the inplace density. The inplace compaction was expressed as a percent of the maximum density at optimum moisture content for the backfill material being placed. A backfill log book was maintained containing all pertinent information concerning daily backfill operation.

In addition, a penetrometer was used, correlated with penetration charts prepared by the central laboratory (see Figures 2.5-234, 2.5-272, and 2.5-534 through 2.5-543) to maintain a continual check on the compaction of the backfill. At Watts Bar Nuclear Plant, Class A backfill was placed around all Category I structures. This material, which was selected earth placed in not more than 6-inch layers, had a minimum required compaction of 95% of the maximum standard density at optimum moisture content. Class A1 backfill used in portions of the underground barrier trenches had the same requirements except it had a minimum required compaction of 100% of maximum density optimum moisture content.

The limits of excavation and the backfill placed around the Category I structures are shown in Figures 2.5-225, 2.5-226, and 2.5-226a.

Class B backfill is placed around non-Category I structures. This material, which was selected earth placed in not more than 9-inch layers, had a minimum required compaction of 90% of the maximum standard density at optimum moisture content.

A third class of fill was also used, Class C, using unclassified fills to be placed in approximately 12-inch layers and was compacted with hauling equipment. This fill class was used in areas not requiring Class A or B fills, or highway and railroad fills, such as spoil areas.

The fill used to form the channel slopes in the intake channel was composed of material originally excavated from the intake channel. The material was compacted to 95% of maximum density at optimum moisture content.

Earthfill borrow areas were worked in a manner which ensured a suitable material for compaction. They were excavated in layers so that widely varying soil classes were not mixed during placement and compaction. Any conditioning which the soil requires is normally accomplished in the borrow areas prior to hauling it to the earthfill site. This conditioning included control of moisture content and removal of deleterious materials. All borrow areas were maintained such that adequate drainage of ground water and surface runoff was provided. Drainage was accomplished by sloping excavations, crowning, channels, dikes, sumps, and pumping, as necessary.

Compaction of large areas of earthfill was accomplished using crawler-drawn or self-propelled sheep-foot rollers. Soils in areas of limited access were compacted with small power tampers or rollers. Compaction and all other earthwork was suspended during periods of inclement weather.

In areas where earthfills with differing compaction requirements adjoin, the compacted fill with the higher degree of compaction was placed prior to the placement of fill of lower density requirements.

2.5.4.5.1.4 Construction Control

All earthfills including engineered granular fills, were placed, tested and controlled in general accordance with applicable ASTM standards.[172]

All fill operations were accomplished in the presence of a trained inspector. The inspector had the authority to suspend fill operations whenever weather or material conditions were judged unsuitable. His responsibilities included material quality, selection, excavation, hauling, placement, and compaction control. Placement was controlled either through the use of compaction control in-place density tests or by a procedural specification supplied by the engineer. This testing determined soil classification, moisture content, inplace density, relative density (granular fill only), and degree of compaction (earthfill only). The frequency of testing is in accordance with ASTM requirements.[172] The inspector may have required additional testing to conclusively identify material or check compaction. A project laboratory was established at the plant site to perform the necessary testing. Project drawings and a series of construction control procedures relayed unique construction requirements to the construction personnel.

2.5.4.5.2 Granular Fill

2.5.4.5.2.1 General

Granular fill materials were used at the site for several purposes, such as structural fill, backfill, to establish a working surface, and for road foundations. The material was obtained from offsite commercial sources. The location and use of any type of material was determined by the engineer for any safety-related feature.

2.5.4.5.2.2 Section 1032 Material

A granular fill material, consisting of crushed stone or sand and gravel, was placed around and below safety-related features in lieu of earthfill in certain locations. The granular fill material was suitable for compaction to a dense, stable mass and consists of sound, durable particles which were graded within the following limits:

Passing	Percent by Weight Minimum	Maximum
1-1/4-inch Sieve	100	
1-inch Sieve	95	100
3/4-inch Sieve	70	100
3/8-inch Sieve	50	85
No. 4 Sieve	33	65
No. 10 Sieve	20	45
No. 40 Sieve	8	25
No. 200 Sieve	0	10

The material was free of soft friable particles, salt, alkali, organic matter or an adherent coating and reasonably free of thin, flat, or elongated pieces.

Laboratory shear strength tests were performed on the granular material to establish design properties. The testing consisted of triaxial (Q&R) and direct (S) shear tests. The tests were made on samples compacted to 70% and 80% of maximum relative density (ASTM D 2049). The samples composition were varied to provide three separate gradations for testing.

The three gradations tested were as follow:

	Percent (by Weight) Passing		
Sieve Size	Maximum Fines	Average Fines	Minimum Fines
1-1/4 inch	100	100	100
1-inch	100	100	95
3/4-inch	100	88	70
3/8-inch	85	67	51
No. 4	65	49	33
No. 10	45	32	20
No. 40	25	17	8
No. 200	10	5	0

Minimum and maximum densities were determined in accordance with ASTM D 2049.

The triaxial shear tests (Q&R) were made in a 4-inch diameter testing machine on particles passing the 3/4-inch sieve. The direct shear tests (S) were made using a 12-inch square shear box on particles passing the 1-1/4-inch sieve. The results of the shear testing are shown on Table 2.5-54, and the values to use for design are shown on Table 2.5-55. Figures 2.5-544 through 2.5-547 are graphical plots of the test results with the adopted design values for each type of shear test.

The apparent shear strength values for the R test were not presented because the test results were determined to be inconsistent. On tests at 80% relative density, two of the three sets of the R tests showed significant negative pore water pressures during the tests. It is unrealistic for a saturated fill of this granular material to develop negative pore pressures. During earthquakes, pool drawdowns, or conditions of steady seepage, a crushed stone fill would more likely develop positive pore pressures rather than negative pore pressures. Thus as indicated on Table 2.5-55, pore pressures were incremented during analysis to check the effect of pore pressure buildup.

The test results indicated that the coarse particle-size distribution (minimum fine distribution) produced a slightly higher friction angle along with a marked increased in cohesion intercept. Part of the 'cohesion' appeared to be the result of interlocking of the angular particles. Overall, the shear strength increased as particle size increased.

Consolidation tests were not made on the granular material, since consolidation would be negligible at the densities the fill is placed and because any connections between adjacent structures would not be made until after any minor consolidation had occurred.

In areas where this granular material was placed adjacent to an earthfill, the granular fill was placed and compacted prior to the placement of the earthfill. Granular fill was placed and compacted to a relative density as specified on drawings or in construction specifications and as determined by ASTM D 2049. The moisture content of the material was adjusted as necessary to obtain the required relative density. The construction control program for granular fill was discussed in Section 2.5.4.5.1.4.

As a result of inquiries by NRC about the granular material used to support the Diesel Generator Building, the following tables and figures are provided:

(1) Table 2.5-56 showing the compaction results;

(2) Figure 2.5-548 showing a statistical summary of the compaction test results; and

(3) Table 2.5-57 showing sieve analysis results on the material stockpile during the period which the granular fill material was placed for the Diesel Generator Building.

2.5.4.5.2.3 Section 1075 Material

This was a free-draining granular fill material, consisting of crushed stone or sand and gravel; it was frequently used to establish a working surface on top of soil or weathered rock, or to develop a good interface between earthfill and weathered rock, or to act as a surface cover for an area such as a switchyard. It was also used as a structural fill.

The granular fill material was graded within the following limits:

Percent (by Weight) Passing

Sieve Size	Bottom Layer	Alternate Bottom Layer	Top 2" Layer
1-1/2-inch	100	100	--
1-inch	90-100	--	--
3/4-inch	40-75	30-75	100
1/2-inch	15-35	--	90-100
3/8-inch	0-15	5-15	40-75
No. 4	0-5	0-5	5-25
No. 8	--	--	0-10
No. 16	--	--	0-5

The material is free of soft friable particles, salt, alkali, organic matter, or an adherent coating and reasonably free of thin, flat, or elongated pieces.

In areas where the material is used, it is placed and compacted using a procedural specification given on drawings or in construction specifications.

2.5.4.6 Groundwater Conditions

The normal ground water level for the main plant is at elevation 726, which is 2-feet below plant grade. This level was determined when making soil borings in the main plant area. Each structure in the main plant area was designed for the normal ground water level at elevation 710 for service load conditions, although most structural design for hydrostatic forces were controlled by the PMF.

In order to control groundwater seepage into any structure, each construction joint up to grade had a seal embedded across the joint to prevent the passage of water. Dewatering during construction was controlled by routing any seepage into the excavation to sumps in the excavation and pumping the seepage away from the excavation. No significant difference in the groundwater conditions were noticed from that described in the PSAR as a result of the construction. Refer to Section 2.4.13 for a detailed discussion of the groundwater conditions at the site.

The design groundwater for the ERCW pipeline and 1E conduit alignments was determined in conjunction with a study of the potential liquefaction of the soils along the alignments. The program to monitor the groundwater and the results of that program are presented in Appendices A and B of 'Watts Bar Nuclear Plant - Liquefaction Evaluation of the ERCW Pipeline Route'[167]. Since that report was issued, additional monitoring of the groundwater was done and the data is reported in Table 2.5-58. Based on the review by the NRC staff of the groundwater study in Reference [167], it was determined the seasonal groundwater did not adequately meet the criteria of a 25-year groundwater. On the basis of discussions with the NRC staff, the groundwater data was reanalyzed to arrive at a more representative 25-year groundwater level. The results of that reanalysis are discussed below.

The methods used in estimating the 25-year high groundwater level along the ERCW pipeline were essentially the same as those used previously to predict the normal seasonal high water table (see Reference [167] Appendix B for description of methods). Basically these methods involved extrapolating the historic high water levels recorded in site observation (control) wells located outside the ERCW pipeline area for which long-term water level records were available, to the ERCW piezometers for which only short-term records exist. The primary difference between the current and previous evaluations lay in the selection of the historic high water levels recorded in site control wells (B2, B3, B4, and B5). Since only 11 years of groundwater level records existed for the site control wells, it became necessary to assume a direct relationship between groundwater levels and rainfall in estimating the 25-year high water table. That is, we assumed that the 25-year high water table was associated with the 25-year high rainfall during the normal wet season of the year (November through March). Monthly rainfall data for the period 1940-1983 recorded at a gauging station at Watts Bar Dam located near WBNP were used in the analysis. A probability distribution of November-March rainfall is presented in Figure 2.5-591. As indicated in this figure, the rainfall recorded for the 1973-74 period approximately corresponds to a 25-year event. Since groundwater level measurements at WBNP are also available for this period, the maximum water table measured on February 8, 1974, is assumed to approximately represent the 25-year high water table. The February 8, 1974, water levels for control wells B2, B3, B4, and B5 were subsequently used to estimate the 25-year water levels in the ERCW piezometers. Results are presented in Table 2.5-73 along with previous estimates for comparison. A contour map of the predicted 25-year water table in the ERCW pipeline vicinity is shown in Figure 2.5-592. A profile comparing the normal and 25-year water tables is presented in Figure 2.5-593.

In general, the 25-year water table is one to two feet higher than the previously estimated seasonal high water table. The only significant exception is piezometer P5. Note that no groundwater was ever detected in P5 from the time it was constructed (October 26, 1981) until it was destroyed (November 15, 1981). Water level in P5 was assumed to be one foot below the bottom of the piezometer, and the seasonal high level was predicted using the same methods used for the other ERCW piezometers. However, as requested by the NRC Staff, the P5 data have been disregarded in the current analysis, and the 25-year level at the P5 location has been estimated by linear interpolation of the predicted levels at adjacent piezometers P4 and P6. Although we believe the method used previously to be a valid and conservative method for predicting water levels at P5, we have agreed to the more conservative NRC approach. The resulting predicted 25-year water table position does not significantly affect the liquefaction potential at the P5 location.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

The response of soil and rock to dynamic loading is discussed in Section 2.5.4.4. Soil-structure interaction and soil-embedded structures are discussed in Sections 3.7.2.4 and 3.7.2.1.1.

2.5.4.8 Liquefaction Potential

The liquefaction potential of all slopes and soil deposits were evaluated by using empirical rules based on observed performance and by comparing the soil conditions and earthquake characteristics at the site with similar sites that have liquefied[155].

The empirical rules used are based on the Japanese experience during the Niigata earthquake. It was observed that the following general coincident conditions could cause liquefaction:

(1) The percentage of silt and clay-size particles should be less than 10%.

(2) The particle diameter at 60% passing should be between 0.2 mm and 1.0 mm.

(3) The uniformity coefficient should be between 2 and 5.

(4) The blow count from Standard Penetration Tests should be less than 15.

Also, Reference [1] states that experience suggests liquefaction might occur for soils having a relative density less than 50% during ground motions with accelerations in excess of approximately 0.1 g; and that for relative densities greater than 75%, liquefaction for most earthquake loadings is unlikely.

Intake Channel

Using the rules outlined above, one layer of potentially liquefiable soil was found in the intake channel. This soil deposit was a layer of silty sand extending from elevation 665 to elevation 680 in the intake channel side slopes. The location of the channel with respect to the plant layout is shown in Figure 2.1-5. The channel is shown in Figure 2.4-99. The zone of potential liquefaction is shown in Sections A-A and B-B of Figure 2.4-99.

The Waterways Experiment Station of the Corps of Engineers performed cyclic triaxial shear tests on samples from this layer of silty sand. The results of the testing program are presented in Table 2.5-22. TVA performed parallel cyclic triaxial shear tests on similar samples, with the results presented in Table 2.5-23. The results from the parallel tests showed reasonable agreement, particularly for the isotropic loading cases.

A dynamic 2-dimensional finite element analysis was performed for the intake channel. The details of this analysis are discussed in Section 2.5.5.2.1. From this analysis the number of equivalent cycles for various levels of shear stress was determined using the procedures outlined by Lee and Chan[156]. Comparing the computed shear stress and number of cycles with the test results indicates that liquefaction would occur. Both sets of test results were used in the liquefaction evaluation and both indicated complete or partial liquefaction. Therefore, it was decided to excavate beyond the limits of the final channel to the top of firm gravel and compact the excavated material back in place to the final channel cross section (Figure 2.5-239) with controlled compaction density and moisture content. The compaction criteria are discussed in Section 2.5.4.5.

ERCW Pipeline and 1E Conduit Alignments

The plan of the ERCW piping and 1E conduit is shown on Figure 2.5-273. The plan shows (1) the location and routing of the piping and conduits, (2) boring locations, and (3) contours of the ground surface. The soils below the pipes and conduits were evaluated for liquefaction potential using the empirical rules outlined above in this section and the soils were judged to be nonliquifiable. Based on the NRC staff's review, the soils were subsequently reevaluated. The reevaluation included additional borings along the alignments (using the same as well as different drilling techniques); excavating test pits to obtain hand-carved undisturbed block samples; cyclic triaxial shear tests; monitoring the groundwater; and using different criteria for evaluating the soils for liquefaction. The additional borings along the piping and conduit alignments are discussed in Section 2.5.4.2.1.2. The purpose of the additional borings was to decrease the spacing of borings to approximately 100-feet along the piping and conduit alignments and to investigate apparent soft layers of fine sands. The majority of the additional borings were made with dry drilling procedures using a hollow stem auger to advance the borings. The borings added to investigate apparent soft layers were made with wet drilling procedures using drilling mud and fishtail bit to advance the boring.

Profiles were prepared to show the relationship of the safety-related piping and conduits to the soil deposits. A profile of the ERCW pipes is provided in Figures 2.5-549 through 2.5-553. The 1E electrical conduit profiles are provided in Figures 2.5-554 through 2.5-556. The profiles provide the following information:

(a) Pertinent boring logs along the routes showing blow counts and the classification of the in situ soil.

(b) The elevation of original grade, final grade, top of weathered shale, and the top of rock. (Note: Fill material was used to backfill around the pipes and to achieve final grade.)

(c) The electrical conduit and ERCW pipelines and their elevation to scale.

(d) The 24-hour water table and the design groundwater.

Figure 2.5-273 is a plan of the piping and conduit which shows the locations where the sections are drawn.

Test pits were excavated at two locations along the pipeline and are discussed in Section 2.5.4.2.1.2. The test pits were made to obtain undisturbed block samples to determine in situ densities and for cyclic triaxial testing.

A series of standpipe piezometers were installed along the pipeline to monitor the groundwater to establish a design groundwater level. The result of this monitoring is discussed in Section 2.5.4.6.

Several evaluations of potential soil liquefaction have been made. The criteria have varied from (1) evaluations based upon empirical rules derived from records of historical events, to (2) evaluations using laboratory cyclic shear testing, to (3) evaluations using empirical rules based on standard penetration testing, grain size analysis, and a correlation procedure developed to Seed and Idriss[159].

Liquefaction Evaluation - Based on Empirical Rules

The first approach to the evaluation of the liquefaction potential of soils was done using grain size distribution outlined empirical rules. Only those silty sands above the top of weathered shale were evaluated since all materials below that point are merely rock fragments which have been given a soil classification.

Figures 2.5-557 through 2.5-563 are grain-size distribution curves for SM material in borings SS-50, SS-60, and SS-63. These curves show that the silty sands are well-graded materials rather than the uniformly graded materials one associates with liquefiable soils. In addition, a comparison of these gradation curves with those for materials which are known to liquefy, including the potentially liquefiable sand encountered in the intake channel, shows that the gradation characteristics are not typical of those for liquefiable soils.

The electrical conduits follow the route defined by borings 49, 50, 51, 52, 53, 57, and 58. The graphic logs of the borings for the Class 1E electrical conduits show that the SM and G-SM materials are not present in extensive layers. These materials are present in isolated pockets. For instance, the SM and G-SM material in SS-50 extending from elevations 689.0 to 699.0 is not encountered in either SS-49, SS-51, or SS-59. Rather, thin layers of silty sand no more than 1 to 1-1/2 feet thick are found at elevations which do not correlate with boring 50. Approximately 5 feet of SM and G-SM material is found above the top of weathered shale in boring SS-53. However, the undisturbed boring SS-53 which was made only 5 feet away encountered on SM or G-SM material above top of weathered shale. One therefore concludes that the silty sand in SS-53 is not an extensive layer. The layers of silty sand in borings SS-60 and SS-63 are not extensions of the same layers since the silty sand of SS-53 is an isolated pocket. Similarly, the remainder of the borings show no evidence to suggest extensive layering of silty sands.

Borings 59 and 51 clearly establish that the silty sands encountered in the other borings for the electrical conduits are not extensions of the potentially liquefiable sands encountered in the intake channel since only a thin layer of silty sand is encountered in 51 and none in 59. The investigation of the intake channel revealed that the continuous layers of silty sand tapered out at the intake pumping station. This investigation substantiates that the silty sands shown in borings along the 1E conduits are not continuous layers. Finally, a comparison of the gradation characteristics of the potentially liquefiable sands of the intake channel (a typical gradation curve is presented in Figure 2.5-566) with those of the silty sands along the conduit and pipeline routes, shown in Figures 2.5-557 through 2.5-565, shows that the sands are not similar.

The criteria used to assess the potential for liquefaction along the ERCW pipeline route is similar to that used for the Class 1E conduits. The ERCW pipelines follow the route shown on Figure 2.5-273. Silty sand was encountered below the water table at various locations along the pipeline route. These silty sands were evaluated: (1) using the above empirical rules; (2) by comparing the gradation characteristics to those of materials known to liquefy; and (3) by the procedures described in References [160] and [161].

The only borings in which silty sands were encountered in quantities to warrant evaluation were SS-88, -90, and -92. The silty sands encountered between elevations 699.0 to 712.0 in SS-88, and 709 to 718 in SS-90, show blow counts as high as 50 and typically between 20 to 40. Evaluating these materials using the procedures of References [160] and [161] showed no potential for liquefaction.

Figures 2.5-564 and 2.5-565 are gradation curves for the silty sands found between elevations 714.0 to 722.0 in boring SS-92 (Figure 2.5-200). These figures show the silty sands to be well-graded materials whereas liquefaction is normally associated with uniformly graded materials. In addition, the percentage of silt and clay present in these samples (42%) is significantly above 50 the range of 10% or less specified in many criteria. A comparison of these materials with the potentially liquefiable silty sands of the intake channel shows that the materials are not similar in their gradation characteristics. The gradation characteristics of the silty sands of the intake channel are uniform which meet the empirical criteria for liquefaction potential, whereas the materials along the pipeline route do not meet the criteria. This dissimilarity of materials, in conjunction with the pattern in which silty sands were encountered along the pipeline route, establishes that these sands are not extensions of the potentially liquefiable sand found in the intake channel. The intake channel soils are discussed in Section 2.5.5. A distinct, continuous layer of silty sand is visible in the graphic logs of the borings in the intake channel. However, the borings show that this layer tapers out and the potentially liquefiable sand of the intake channel is confined to the floodplain.

The elevation of the water table is not shown on the graphic log for boring SS-92 (Figure 2.5-200). However, it is possible to infer the elevation of the water table from other borings around SS-92 and along the alternate route just east of the cooling towers (see Figure 2.5-273). Specifically, the water table elevation was determined in borings SS-65, -67, -87, -88, -93 through -95, -104, -105, -107, and -108. In those borings, the elevation of the water table varies from a minimum elevation of 693 to a maximum elevation of approximately 704. From Figure 2.5-273 it is apparent that the borings mentioned above surround SS-92. Furthermore, the area in which SS-92 was taken has no apparent topological features which could result in an unusually high water table. Therefore, it is reasonable to infer that the elevation of the water table in SS-92 is approximately 702 and certainly no more than elevation 705. If a water table elevation of 703 is assumed for boring SS-92 and with a 3 to 4 feet fluctuation, the SM material between elevations 714 and 722 is typically 10 to 11 feet above the water table. Therefore, the SM material in boring SS-92 will normally be in an unsaturated state and cannot liquefy.

In summary, the soils along the routes for the Class 1E conduits and ERCW pipelines have been evaluated for liquefaction potential. This evaluation has considered: (a) the graphic logs for the materials encountered; (b) the results of the laboratory tests on undisturbed samples of these materials; (c) the location of and fluctuations in the water table; (d) the location of top of weathered shale; (e) a determination of the extent of the liquefiable sands of the intake channel; (f) the simplified procedures of Seed and Idriss[160]; and (g) the information presented by Shannon & Wilson and Agbabian Associates[161]. Based on these criteria it is concluded that the soils along both routes will not liquefy.

Liquefaction Evaluation - Based on Cyclic Triaxial Shear Tests

The second approach to the evaluation of the liquefaction potential of soils was done using cyclic triaxial shear testing. This was in response to NRC and WES concerns with the initial liquefaction evaluation and the results of cyclic triaxial shear testing in the intake channel. The results of the cyclic triaxial shear testing on soils along the ERCW pipeline and 1E conduit alignments are given in Section 2.5.4.2.1.2 (Cyclic Triaxial (R) Testing - ERCW Pipeline and 1E Conduit Alignment).

The water table used in the analysis was based upon 24-hour readings in borings since a 25-year water table was not established for the site. Groundwater is discussed in detail in Section 2.5.4.6

The blow counts of the alluvial soils are given in Figures 2.5-549 through 2.5-556. The extent of the alluvial sand is provided in Figures 2.5-567 and 2.5-568. The cyclic triaxial shear test characteristics of alluvial soil susceptible to liquefaction are discussed in Section 2.5.4.2.1.2. The liquefaction analysis consisted of one-dimensional dynamic response computations performed for soil columns in the free field. These analyzes indicate an equivalent cyclic shear stress of 325 lb/in^2 at a depth of 15 to 20 feet. This depth corresponds to the location of SM material encountered in the borings. The resulting cyclic stress ratio is 0.32.

The profile selected and analyzed is based on boring SS-50-1 (Figure 2.5-339) which contained the most SM material. The surface elevation is 716.9. Around elevations 685.0 and 690.0 the blow count increases to +50 and is identified as 'top of weathered shale.' This is assumed as 'top of rock' for the liquefaction evaluation. Thus the depth of the profile is 30 feet. The water table is about 15 to 20 feet below the ground surface in borings SS-50, SS-50-1, SS-65, and SS-65-1. The profile analyzed is fairly typical of those along the ERCW route. This generalized soil profile is shown graphically in Figure 2.5-569.

The soil unit weight (moist) was assumed to be 120 lb/ft^3. The shear wave velocity of the soil is taken as 1,000 ft/s. This value is in agreement with data obtained from the intake channel and elsewhere on the site. The strain dependent shear modulus and damping ratio properties of these soils are assumed to conform with the relationships developed by Seed for sand. The coefficient of earth pressure at rest (K_o) is conservatively taken as 0.5. All soil properties are assumed to be constant with depth.

The rock has a unit weight of 165 lb/ft^3 and a shear wave velocity of 5,900 ft/s.

The postulated site SSE is based on an intensity MM VII-VIII or VIII event. Such an event would be approximately a magnitude 5.5 to perhaps a 6.0. The seismic input at the site is defined as a 0.18 g earthquake at top of rock. The liquefaction evaluation was performed using an artificial accelerogram which conforms to Regulatory Guide 1.60 requirements. Peak accelerations of 0.18 g, 0.225 g, and 0.25 g were considered.

The accelerogram was also high band pass filtered to eliminate frequencies greater than 5 Hz for three cases and 25 Hz for two cases. In all, five different analyses were performed and are listed below.

Case	Maximum Acceleration	Applied at Top of	Upper Frequency Cutoff
1	0.25 g	Ground	5 Hz
2	0.18 g	Ground	5 Hz
3	0.225 g	Ground	5 Hz
4	0.25 g	Ground	25 Hz
5	0.18 g	Rock	25 Hz

The most appropriate seismic loading is Case 1 where the 0.25 g accelerogram is applied at top of ground with a 5-Hz upper frequency cutoff. The results from Case 1 essentially envelop all other cases except for Case 5 where the input is at top of rock.

The dynamic response analysis was performed using the computer program SHAKE. Irregular shear stress time histories were not calculated. The equivalent uniform cyclic stress was taken as 65% of the maximum cyclic shear stress within each layer of the profile as calculated by SHAKE.

The results of the analyses are given in Table 2.5-59. The maximum and equivalent uniform stresses within each layer and the peak accelerations at the top of each layer are summarized in Table 2.5-59 for all five earthquake input conditions.

For material located about 17.5 feet below the surface (approximately the elevation of the samples which were tested cyclically), the maximum shear stress is:

$$\tau_{max} = 500 \text{ lb/ft}^2$$

The average shear stress is:

$$\tau_{avg} = 0.65, \quad \tau_{max} = 325 \text{ lb/ft}^2$$

The vertical pressure at 17.5 feet is:

$$\sigma_v = \delta h = (120 \text{ pcf})\ (17.5 \text{ ft}) = 2100 \text{ lb/ft}^2$$

Assuming $K_0 = 0.5$, the horizontal stress is:

$\sigma_h = 0.5\ \sigma_v = 1050\ lb/ft^2$, use $\sigma_h = \sigma_3 = 1000\ lb/ft^2$

The cyclic stress ratio is:

$$\frac{\sigma_d}{2\sigma_3} = \frac{\tau_{avg}}{\sigma_3} = \frac{325\ lb/ft^2}{1000\ lb/ft^2} = 0.32$$

Figure 2.5-353 shows the most susceptible sample will survive six load cycles with this stress ratio. Only five uniform load cycles should occur from a 0.18 to 0.25 g event. This event is an intensity VIII earthquake and is characterized as an m_{blg} 5.8. Extrapolating Seed and Idriss data:

Magnitude	Number Of Cycles	Equivalent Uniform Cyclic Stress
7	10	0.65 max
7-1/2	20	0.65 max
8	30	0.65 max

Five cycles of uniform load have been conservatively assumed for a magnitude 5.5 to 6.0 event. Factors of safety against the development of 5% strain are given in Tables 2.5-60 and 2.5-61 and Figure 2.5-570. These factors of safety were calculated only for seismic loading case 1. Results are presented for cases where the water table is not considered and where it is located 16.5 feet below the surface. The 16.5 feet groundwater depth is in the upper range as determined by the borings and the exact number 16.5 feet is chosen for convenience only. Factors of safety were calculated for both the reconstituted sample (sample 3) and for the in situ (undisturbed) sample (sample 2). The in situ sample is more representative of field conditions. It should be noted that these factors of safety are against the development of 5% strain and not against actual liquefaction which, if it occurs, occurs at strains in excess of 10% for the samples tested.

The scatter of the test results tabulated on Table 2.5-36 and plotted as shown on Figure 2.5-353 were anticipated due to the variations in the soil. These tests were conducted on in situ (undisturbed) soil samples. Only the soils judged most susceptible to liquefaction were selected for testing. Of the three samples selected for testing, all available specimens were tested. Sample 3 shows some scatter. All specimens from sample 3 were reconstituted due to the presence of a large gravel particle. Three of the four test points for US-50-1, sample 3, form the classical curve for cyclic test results. However, the fourth point (the lower point at 3 cycles) is off the curve. The curve was constructed by using the test results for the cyclic stress ratios of 0.44, 0.26, and 0.17. The remaining two curves were constructed essentially parallel to the first curve.

Comparisons of the stresses induced by the earthquake and the stresses required to cause a given strain level provide a means to determine the potential compressive strains in the soil. These potential compressive strains represent the strains that would develop in that part of the soil if it were not constrained by the surrounding soil (Reference 164). These strains may, therefore, be considered as the strain potential caused by the earthquake and can be used to assess the deformation (settlement and lateral movement) potential of the soil.

A criterion of 5% strain is generally used for evaluating the stability of embankments subjected to earthquake shaking. This criterion has been established on the basis of correlations between the results of seismic stability evaluations and the performance of earth dams which have been subject to significant earthquake loading (Reference 163 and 164). These case histories show that if the computed strains in the embankment are smaller than 5%, the earthquake has no significant effect on the stability and integrity of the dam. However, it should not be concluded that the stability and integrity of the embankment is impaired if the computed strains exceed 5% at some locations within the embankment. The effect of computed strains exceeding 5% depends on the zone of the embankment in which they may occur, and on the relative extent and location within a specific zone.

The cyclic strength stability may be evaluated by using Figure 2.5-353 and a cyclic stress ration of 0.32. This shows:

(1) US-50-1, sample 3, experiences 5% strain at 6 cycles of load. These 6 cycles are close enough to the postulated 5 cycles due to the SSE to indicate this layer may experience up to approximately 5% strain. This is a reconstituted test specimen; an undisturbed in situ specimen should perform even better.

(2) US-50-1, sample 2, experiences 5% strain until the loading greatly exceeds 10 cycles.

(3) US-50-1, sample 4, experiences less than 5% strain at 1000 cycles and is essentially unresponsive to cyclic loadings.

In general, only one sample is likely to experience 5% strain (actually strain potential). This implies the earthquake will have no significant effect on the stability and integrity of the soils along the route. Also, it should be noted that this zone is confined above and below by soils that are relatively less susceptible to liquefaction. Figure 2.5-353 clearly shows that samples 2 and 4 will not liquefy under the adopted earthquake loading conditions.

Liquefaction Evaluation - Based on SPT Data and Seed and Idriss Correlation

The third approach to the evaluation of the liquefaction potential of the soils at the site was made using standard penetration test (SPT) data, a correlation procedure developed by Seed and Idriss (1981) (Reference 159) and additional cyclic triaxial tests. Additional split-spoon borings were made adjacent to some existing borings. These additional borings were made with drilling mud and a fishtail bit. The results of this additional drilling are given in Section 2.5.4.2.1.1. The correlation procedure developed by Seed and Idriss (1981) differentiates between clean sands and dirty sands. The fine sands at the site are relatively dirty or silty with fines content (minus No. 200 sieve) ranging up to 50%. The cyclic performance of the silty sands is better than for clean sands when both materials have the same blow count or other similar characteristics.

This increased performance under cyclic loading is accounted for in the Seed and Idriss (1981) procedure. The additional cyclic tests were made on undisturbed block samples obtained from two test pits excavated along the ERCW piping alignment.

Several reports have been submitted to the NRC on this evaluation. The initial report, Reference 166, was issued February 8, 1982. The liquefaction evaluation was based on a seismic input of 0.18 g at the top of ground. The report also provided the results of a study to project the seasonal high groundwater levels for use in the liquefaction evaluation. These seasonal high groundwater levels were used in this and all subsequent liquefaction evaluations as the design groundwater. Additional water level readings from the groundwater monitoring program are given in Table 2.5-58. This evaluation concluded that although a few samples (3) had factors of safety against potential liquefaction less than one, the areas of concern were isolated and unlikely to cover a large lateral area and the potential settlement was insignificant. The second report Reference 167, was an update of the initial report, Reference 166. The second report, Reference 167, basically expressed the same conclusions as the initial report, Reference 166. However, this report, Reference 167, contained cyclic triaxial test results showing that the samples that had factors of safety less than one, using the Seed and Idriss (1981) procedures, would not liquefy.

Due to NRC concerns about the seismic input to the liquefaction evaluation, an additional study was made. The seismic input to the evaluation was 0.22 g at the top of ground. This revised seismic input was based on the results of a site-specific study and as discussed in Section 2.5.4.2. The third report, Reference 168, containing the results of this evaluation, was issued in November 1982. The report indicated that additional samples were susceptible to liquefaction, but the samples were localized and there were no indications that the liquefiable zones were continuous. In addition, the cyclic triaxial tests showed that these additional samples would not liquefy. The potential settlement due to the postulated liquefaction was also considered minimal.

As a result of several meetings with the NRC and the NRC's review References 166, 167, 168, the seismic input and the procedure for evaluating liquefaction were changed. The seismic input was changed from 0.22 g to 0.40 g at top of ground as discussed in Section 2.5.2.4. The procedure for evaluating liquefaction was changed from the Seed and Idriss (1981) procedure to the Seed and Idriss (1971) procedure. Both procedures are simplified methods for evaluating the liquefaction potential of sands, but the Seed and Idriss (1981) procedure provides a modification that accounts for presence of fines in the sand samples. In order to resolve the issue of potentially liquefiable soils at the site, TVA used the Seed and Idriss (1971) procedure.

A report has not been issued on the liquefaction evaluation based on a seismic input of 0.40 g at top-of-ground and the Seed and Idriss (1971) procedure. However, the results are presented as follows:

(1) Tables 2.5-62 through 2.5-64 tabulates the samples that would potentially liquefy, i.e., (FS$\leq$1.0).

(2) Figure 2.5-273 shows the layout of the ERCW piping and 1E conduits and the location of the sections that show the piping and conduit profiles.

(3) Figures 2.5-571 through 2.5-575 show profiles of the ERCW piping and the borings along the alignment. The borings have been marked to indicate the design groundwater, top of weathered shale, and the samples that will potentially liquefy.

(4) Figures 2.5-576 through 2.5-579 show profiles of the 1E conduit banks and the borings along the alignment. The borings have been marked to indicate the design groundwater, top of weathered shale, and the samples that will potentially liquefy.

The result of this evaluation is that the zones of potentially liquefiable materials are apparently continuous in some areas along the pipeline and conduit alignments and that some method of remedial treatment is needed. The method of remedial treatment to prevent the lateral flow of liquefied soils, the method of analysis, and the results are described in Sections 2.5.5.1.2 and 2.5.5.2.3.

As discussed in Section 2.5.4.6, the groundwater level was revised to reflect an estimated 25-year groundwater. The influence of this slightly higher groundwater on the liquefaction analysis and potential settlement due to liquefaction was discussed with the NRC staff. The staff indicated they concur with TVA's judgement that the higher groundwater will have negligible effects on the results of the liquefaction and settlement evaluations; therefore, no additional evaluations for the piping or conduits are needed.

The potential settlement of the soils along the ERCW pipeline and 1E conduit alignments, due to an earthquake sufficient to cause liquefaction, were evaluated for each report, References 166, 167, 168. All studies revealed that the potential settlement was insignificant or minimal and the performance of the piping or conduits would not be affected. When the peak ground acceleration was increased to 0.40 g (see Section 2.5.2.4) and the method of evaluating for potential liquefaction was changed to the Seed and Idriss (1971) procedure, the extent of the soils that would potentially liquefy increased, thereby significantly increasing the amount of potential settlement. The theoretical settlement at each boring location along the ERCW pipeline and 1E conduit alignments was calculated twice. The initial settlement evaluation was based on a paper by Lee and Albaise (1974) Reference 165). The second evaluation was based on a criteria provided by the NRC staff. The method and results of each evaluation are described below.

The evaluation based on Lee and Albaise's paper assumed the test data for a Monterey sand was applicable and the in situ relative density of the fine sands was 50%. Using test data for a Monterey sand is conservative, since the D_{50} for the fine sands at the Watts Bar site is in the range of 0.07 mm to 0.15 mm, and the test data shown in Figure 6 of the Lee and Albaise paper indicates that a finer sand will experience a lower volumetric strain. The use of an in situ relative density of 50% is also conservative, since the relative densities of the undisturbed block samples from the test pits ranged from 61% to 69% for two of the samples and above 70% for the other sample. The test data shown in Figure 7 of the Lee and Albaise paper indicates that a soil with a lower relative density will experience a higher volumetric strain. Based on Figure 7 of the Lee and Albaise paper, a Monterey sand sample with an initial relative density of 50% that subsequently liquefies will experience approximately 1.5% volumetric strain. For the initial settlement evaluation sand (SM or SP) samples that were theoretically susceptible to liquefaction were considered to experience 1.5% volumetric strain, and silt (ML) samples were considered to experience 0.75% volumetric strain. Figures 2.5-571 through 2.5-578 show the potential settlement calculated using the 1.5% strain (1.5%ξ) criteria at each boring along the pipeline and conduit alignments.

The criteria specified by the NRC staff is shown in Table 2.5-65 has a maximum volumetric strain of 6%. The criteria specifies a volumetric strain even for samples that will not liquefy. The results of the evaluation for potential settlement at each boring along the pipeline and conduits using the 6% strain (6%ξ) criteria are also shown on Figures 2.5-571 through 2.5-578. As can be noted, the potential settlement using the 6% criteria is significantly higher than the results using the 1.5% strain criteria. However, in order to resolve the issue of the potential settlement due to soil liquefaction, the results of the settlement evaluation based on the NRC staff's criteria (6%ξ) was used for evaluating the need for remedial treatment for the pipeline and conduits. The evaluation of the piping for the potential settlement along the ERCW piping alignment is described in Section 3.7.3.12. The evaluation of the conduits for the potential settlement along the 1E conduit alignment Is discussed in Section 3.7.2.1.3.

2.5.4.9 Earthquake Design Basis

For the Earthquake Design Basis, see Sections 2.5.2.6 and 2.5.2.7 and Section 3.7.

2.5.4.10 Static Analysis

2.5.4.10.1 Settlement

All Category I structures, except for small structures such as electrical manholes and handholes, are founded either on bedrock or engineered granular fill. Settlement computations, where made, were based on the Theory of Consolidation developed by Terzaghi with the equation for settlement of a layer of soil being:

$$S = H\left[\frac{e_1 - e_2}{1 + e_1}\right]$$

S = settlement In layer
H = initial thickness of layer
e_1 = initial void ratio of material before loading
e_2 = final void ratio after loading

The allowable settlement for any structure is dependent on the amount of deflection any associated piping and electrical connections will be able to withstand. Any potential differential settlement is accounted for in the design and analysis of the structural foundation.

Consolidation tests were made on the fine-grained soils which overlie the in situ gravel for the Diesel Generator Building (see Table 2.5-6 for a summary of these tests). Consolidation tests were not made for the fine-grained soils beneath any other Category I structures. The results of the investigation for the Diesel Generator Building indicated that settlement might be a problem and for this reason the fine-grained soils above the in situ gravel were removed and replaced with compacted granular fill. On other Category I structures where the structural load would cause a net increase in pressure on a fine-grained soil layer, the fine-grained soil layer has been removed and the structural foundation supported on granular material (see Figures 2.5-225, 2.5-226, and 2.5-226A for typical sections of these other Category I structures).

Some Category I structures are founded on fine-grained soils. However, none of these structures cause an increase in the net soil pressure in the supporting fine-grained soil layer. These structures are basically manholes and handholes for the conduit and piping systems. These structures are floating foundations, where the weight of material removed is equal to or greater than the structural weight added. Consolidation tests were not warranted and not made on soil layers below these structures.

2.5.4.10.2 Bearing Capacity

The ultimate bearing capacities for the Category I soil-supported structures were computed using Terzaghi's equations[169] and DeBeer's equations modified by Vesic[170]. Analysis for bearing capacity was made for the weakest soil layer encountered between the bottom of the foundations and the top of rock.

Soil profiles under each structure are based on the results of field investigations as reported in Section 2.5.4.2.1. Ultimate bearing capacities were computed using the appropriate shear test data. The theoretical minimum ultimate bearing capacities for any loading case were used to determine safety factors. The ultimate bearing capacities and the factors of safety against bearing type failures are reported in Table 2.5-66.

The remaining soil-supported structures (Class IE electrical system manholes and handholes for conduits and piping system) are supported on floating foundations, where the weight of material removed is equal to or greater than the structural weight added. There would be no net increase in pressure on the supporting soil layers.

Based on the foregoing soil investigation and the analysis, it is confirmed that all the Category I soil-supported structures are adequately founded and are safe against bearing type failure.

2.5.4.11 Safety-Related Criteria for Foundations

2.5.4.11.1 General

The foundation material beneath Category I features are either in situ soil, compacted granular fill, or in situ rock. Structural loads are transferred to this material through concrete foundations. The configurations of these foundations vary but all are some form of either a mat or spread footing. Some of these foundations rest on a mass concrete placed on bedrock. Detailed foundation descriptions are found in Section 3.8.5. The type of foundation beneath each Category I structure is shown on Figures 2.5-225, 2.5-226, and 2.5-226a.

2.5.4.11.2 Rock Strength

Refer to Sections 2.5.4.2.2.6 and 2.5.4.2.2.7.

2.5.4.11.3 Soil Strength

The allowable bearing pressure for soil-supported foundations was determined by methods outlined in Section 2.5.4.10.2 using a factor of safety of 3. Settlement analyses are made and, if necessary, the design bearing capacity is reduced to limit determination from stress relief and exposure.

2.5.4.12 Techniques to Improve Subsurface Conditions

Several techniques were used to improve subsurface conditions. The techniques included excavation and backfill, subsurface grouting, and placing concrete on freshly exposed shale to prevent further weathering from exposure.

The excavation and backfill technique was used in three locations: (1) in the intake channel, (2) below the Diesel Generator Building, and (3) below the refueling water storage tanks. In the intake channel, due to presence of possible liquefiable soils, it was decided to excavate beyond the limits of the final channel to the top of firm gravel and to compact the excavated material back in place to the final channel cross section with controlled compaction. The compaction density and moisture content criteria for Class A fill is described in Section 2.5.4.5.1.3. For the Diesel Generator Building, it could not be assured that the material directly below the structure could safely carry the load. Therefore, in order to assure a safe foundation for the building, the material between the top of firm gravel and the grade slab was removed and replaced with granular fill as illustrated in Figure 2.5-226. This is a sound durable stone well graded with a maximum size. The criteria for the granular fill is discussed in Section 2.5.4.5.2.

The refueling water storage tank is being treated in the same manner as the Diesel Generator Building except that the granular fill will be compacted to a density of 85% of maximum relative density.

The technique of subsurface grouting was used as follows. On March 28, 1974, while drilling 3-inch percussion holes for No. 9 J-Bar installation in the No. 2 reactor cavity south wall, a disintegrated shale pocket was intersected which allowed accumulated water and shale fragments to be blown out nearby drill holes. Suspecting possible solution or cavity development in the reactor wall, a geologic investigation was initiated to determine the conditions and extent of the zone and to develop an adequate treatment program.

On March 29, 1974, 10 holes were drilled, Nos. 6 through 15, into the wall at angles shown on the accompanying drill layout diagram for horizontal holes (Figure 2.5-139).

The disintegrated shale pocket was outlined as shown on Figure 2.5-140, with its centroid at an approximate elevation of 671.0, and extending in a generalized S 40° W direction along regional strike.

On April 1, 1974, 12 additional vertical holes were drilled, Nos. 16 through 27, as shown on the drill layout diagram for vertical holes (Figure 2.5-142). Of the 12 additional holes drilled, 7 holes intersected the disintegrated shale pocket. This information provided a basis for establishing an approximation of the extent and shape of the disintegrated zone.

The disintegrated zone developed along an area where a subsidiary fault intersected a through-going fault which strikes approximately N 35-40°E and dips 45° southeast.

The zone is complicated by a small anticlinal fold, in all probability a drag fold developed during movement along the larger fault. The fault and the folded structure were exposed at the southeast corner of the reactor cavity wall and continue northeastward. In this structurally complex area the less competent shale strata were ground into small fragments, making the material prone to disintegration upon exposure to water.

The disintegrated shale zone in reactor No. 2 foundation was grouted on April 19, 1974.

Water was pumped into hole Nos. 8 and 11. Water began to flow out of the top holes in the following order: Nos. 16, 17, 19, 20, 22, 23, 28, 18, 21.

The main flow of water was flowing from No. 16. The water was allowed to flow until it became clear, at which time the hole was plugged with a wood plug and the main flow shifted to hole Nos. 19 and 20. These were allowed to run until the water was clear. This method of plugging holes and shifting of main flow was as follows:

(1) Plugged No. 20--main flow shifted to No. 19.

(2) Plugged No. 19--main flow shifted to Nos. 17 and 22.

(3) Plugged No. 17--main flow shifted to No. 22.

(4) Plugged No. 22--main flow shifted to No. 23.

(5) Plugged No. 23--slight flow from Nos. 18 and 24.

Water pressure was held at approximately 5 to 10 psi through the flushing operation. After all holes were flushed, the hoses were disconnected from Nos. 8 and 11 and allowed to drain for one hour.

Grouting was begun by pouring a 1 to 1 mixture of grout into hole No. 19. When three and one-half bags were poured into this bole, grout began flowing out through hole No. 8. When the valve was closed on hole No. 8, hole No. 19 stopped receiving grout. Grouting continued according to the following sequence.

(1) When hole No. 19 stopped receiving grout, grout was poured through hole No. 16. Hole No. 16 stopped receiving grout after four bags were poured into it.

(2) Poured grout into hole No. 17. After one and one-half bags were poured, grout ran out of hole No. 11, which was then shut off.

(3) Alternating between holes Nos. 16, 17, 19, and 20, grout was poured until they would no longer receive grout. An additional four bags were poured into these holes.

(4) Poured grout into holes Nos. 18 and 23. Both received grout very slowly, and a total of one-half bag was placed into these holes.

(5) Holes Nos. 28, 27, 25, 26, 22, and 21 were filled with grout, but too, only that required to fill holes--approximately one-half bag of grout.

A total of 14 bags of grout was poured into this area. Grout did not flow from any of the top holes at any time while grout was being poured into another hole.

The following day, all holes in which grout had settled out were filled to the top. Some of these holes had settled as much as six feet.

In order to back up the 'disintegrated shale zone' treatment and to ensure against possible free passage of ground water around the south and east reactor cavity walls, 28 percussion holes were drilled on April 25 and 26 as indicated on Figure 2.5-144.

Grout application started on April 30, 1974, by gravity feed from the top of the hole at approximate elevation 690 for holes 1 through 10, and at approximate elevation 687 for holes 11 through 28. As each consecutive hole was initially filled, all prior holes were backfilled. Grouting proceeded from holes 1 through 28 in the following order.

(1) Holes 1 through 9 were started with a 3:1 water/cement ratio grout even though visible water was standing at various depths. Water was not blown from holes because of the potential for further agitation of the south wall zone. After an initial application of 3:1 grout, subsequent filling was with 1:1 or 3/4:1 grout.

(2) Holes 10 through 28 were initially blown free of water. First grout application was with a 1:1 mix. Near the end of the program a 3/4:1 mix was used. No between hole connections occurred in this series during the blowing operation.

The operation started at 8:25 a.m. with all initial fillings completed by 1:50 p.m. Backfill operations continued until 2:40 p.m., at which time all holes were considered to have stopped accepting grout. On May 1, 1974, all holes were 'topped out' with dry cement.

A total of 255 gallons of grout was mixed using 22 bags of cement. Of this total about 14 gallons were wasted. Because of the continuous refilling of holes, an exact distribution was impractical. Approximate total acceptance for each hole is indicated in Table 2.5-20. No excessive grout takes were apparent; therefore, no additional holes were drilled and the program was considered complete.

For the structures founded on bedrock, there was a need to protect the rock from stress relief and weathering after excavation to grade. Protection was provided by covering freshly exposed bedrock with concrete.

2.5.4.13 Construction Notes

No significant construction problems were encountered and no major design changes were initiated due to the foundation rock. In order to eliminate potential massive overbreak, initial construction plans precluded the use of explosives. Excavations were completed by drilling closely spaced percussion holes along all cut faces and removing the rock by use of rippers, backhoes, and paving breakers. Groundwater was easily handled after the terrace material was removed.

2.5.5 Stability of Slopes

2.5.5.1 Slope Characteristics

2.5.5.1.1 Essential Raw Cooling Water Intake Channel Slopes

The intake channel is a man-made feature extending approximately 800-feet from the edge of the reservoir through the flood plain to the intake pumping station. The results of the soils exploration and testing are presented in Section 2.5.4.2.1.3. Characteristics of the slopes and the underlying soil deposit are also presented in Section 2.5.4.2.1.3.

2.5.5.1.2 Underground Barrier for Protection Against Potential Soil Liquefaction

The underground barrier is a manmade feature extending along the ERCW pipeline and 1E conduit alignments in the area north of the intake pump station and south of the cooling towers and 500-kV switchyard. The purpose of the underground barrier is to prevent the lateral flow of soils should an earthquake occur that could liquefy some of the soils below the ERCW piping and 1E conduits. The underground barrier is located between the safety-related piping and conduits and the area towards which the material would attempt to flow should the soils liquefy. The liquefaction evaluation is presented in Section 2.5.4.8.

The underground barrier was constructed by excavating two trenches. The location of the underground barrier trenches is shown on Figure 2.5-580. The location was based on the extent of the potentially liquefiable soils along the piping and conduit alignments as shown on Figures 2.5-571 through 2.5-578. Figure 2.5-582 shows the layout of the underground barrier trenches in relation to the borings which indicate potentially liquefiable material.

The trenches were backfilled with soils excavated from the trenches, if acceptable, soil from approved onsite borrow areas and granular fill from off-site commercial sources. The method of construction and construction control was in accordance with the requirements and notes on Figures 2.5-580 and 2.5-581. The results of the soils investigation and testing of the borrow materials are described in Section 2.5.4.5.1. The design and analysis of the underground barrier is described in Section 2.5.5.2.3.

As can be seen on the layout (Figure 2.5-582) and on the profiles, some borings with potentially liquefiable material will not be included in the area encompassed by the underground barriers and no remedial treatment is being planned. Each of these areas is discussed in detail as follows:

(1) At boring SS-143 (Figure 2.5-571, sheet 2 of 4) and its associated borings (SS-143A, B, and C), the soil is localized; the liquefiable material is a thin layer which would produce small settlements. In three of the borings, it is unrealistic to expect the material to liquefy. The G-SP-SM (elevation 693.0) in boring SS-143 is part of the basal gravel that exists at the site (the "G" indicates the sample has greater than 12% gravel); the CL-ML (elevation 697.0) in boring SS-143C should not liquefy due to the high percentage of fines; and the SM (elevation 696.0) in boring SS-143B with a blow count of 21. The results of an extensive test program on the basal gravel is discussed in Section 2.5.4.2.1.3 (In Situ Basal Gravel).

(2) At borings SS-146 and SS-147 (Figure 2.5-571, sheet 2 of 4) both samples shown to be susceptible to liquefaction are in the basal gravel. Also, the blow counts (13 and 18) of the samples (13 and 18) indicate a fairly firm material.

(3) At boring SS-153 (Figure 2.5-571, sheet 3 of 4) the sample (G-SW-SM at elevation 707.0) represents a thin isolated pocket and the sample is in the basal gravel.

(4) In the main plant area (Figures 2.5-571, sheet 4 of 4, 2.5-572 through 2.5-575, and 2.5-577 and 2.5-578), there are no problems related to soil flow during liquefaction since there are no slopes in the area. Potential settlement in this area is discussed in Section 2.5.4.8.

(5) In the southern part of the switchyard, soils encountered in borings SS-53, SS-54, SS-55, SS-62, and SS-61, show some liquefaction potential. However, liquefaction does not appear to be realistic. In boring SS-53 (Figure 2.5-579) the two samples, an ML (elevation 711.0) and an SM (elevation 707.0) with apparent liquefaction potential have high blow counts (20 and 18) and one, the ML, has a high plasticity index (PI-18.4). In boring SS-54 (Figure 2.5-579) the two samples, an ML (elevation 703.0) and an SM (elevation 701.0) that apparently would liquefy have high blow counts (19 and 21) and have medium to high plasticity indices (PI = 10.4 and 16.8). At boring SS-55 (Figure 2.5-579) the two ML samples (elevations 714.0 and 709.0) have blow counts that are good to high (14 and 19) and the plasticity indices are high (PI 18.4 and 14.3). At boring SS-62 (Figure 2.5-579) the blow count of the potentially liquefiable material (elevation 687.0) is good (14) and the plasticity index is high for an SM (PI - 13.8). In addition, the layer is very thin and is probably weathered shale rather than alluvium. At boring SS-61 (Figure 2.5-579) the material is localized, located at the surface where it will not affect any soils overlying it; and it is a long distance from the conduit bank.

2.5.5.2 Design Criteria and Analysis

2.5.5.2.1 Design Criteria and Analyses for the Essential Raw Coolant Water

Intake Channel Slopes

The static design cases and the conditions and factors of safety associated with each are shown below:

Case	Factor of Safety
(1) Normal operating condition: reservoir elevation 675, ground-water elevation 685.	1.5
(2) Sudden drawdown due to loss of downstream dam: groundwater elevation 685; reservoir drawdown elevation 685 to 666.	1.1
(3) Construction condition: groundwater elevation 685, channel dry.	1.25

The earthquake design cases are the same as Case 1 and 2 above combined with a Safe Shutdown Earthquake. The minimum factor of safety must be equal to or greater than 1.0.

Static Analysis

Slip circle analysis using the Modified Swedish method were performed for the static design Case 2. The critical circle, which has a factor of safety of 2.5, is shown in Figure 2.5-238. The combination of events comprising design Cases 1 and 3 are less than those for Case 2. Since the factor of safety for Case 2 is 2.5, then the factor of safety for Cases 1 and 3 will be greater than that required for these cases.

The soils exploration in Section 2.5.4.2.1.3 disclosed a possible weak layer of lean clay soil at approximate elevation 680 to 685 in borings US-30 and US-36, which are on opposite sides of the channel near the reservoir. The test results indicate the minimum strength properties of this material as $\varphi = 3°$ and c = 500 psf. Wedge analyses were performed for design Case 2 assuming a failure plane at elevation 680, using these strength properties under the wedge. The minimum wedge, which has a factor of safety of 3.7, is shown in Figure 2.5-238. By inspection again, design Cases 1 and 3 are satisfied.

Earthquake Analysis

The soils exploration results presented in Section 2.5.4 revealed some silty sands that were possibly subject to liquefaction under earthquake excitation. Section 2.5.4.8 deals with the evaluation of the liquefaction potential following the performance of cyclic triaxial shear tests on these silty sands.

The cyclic testing program showed that this material would liquefy when subjected to earthquake motion. It was therefore decided to excavate this material and compact it back into place as described in Section 2.5.4.5. Section 2.5.4.2.1.3 presents the results of normal shear tests on the remolded channel area soils. The more important cohesion value is conservatively taken as 1,200 psf; friction angle is assigned an average value of 15 degrees. The two values are the same as the undisturbed values for the in situ clay above elevation 680.

A dynamic 2-dimensional finite element analysis for earthquake design Case 2 was performed on the intake slopes using in situ test results in Section 2.5.4.2.1.3. By inspection, design Case 2 controls for seismic analysis and is the only case considered. The finite element analysis considered the soil to behave as an elastic medium with a constant damping of 10% of critical. The soil deposit was modeled from the centerline of the intake channel to a distance of 350-feet beyond the crest of the intake channel side slope and from the top of the soil deposit to a fixed boundary at bedrock. Earthquake motion is input into the model at this fixed boundary at bedrock. The four artificial time histories discussed in Section 3.7 were each in turn used as input directly into the base of the model. Acceleration profiles for the intake channel were then prepared for the accelerations produced by each of the four records. An average acceleration was then calculated from the four profiles for use as seismic coefficients and taking into account the location of the various failure planes to be investigated by a pseudostatic approach (see Figure 2.5-237). Accordingly, for a failure plane at elevations 650.0 (at or directly above bedrock) to 665.0, a seismic coefficient.of 0.30 g was calculated for the SSE. Similarly, a seismic coefficient of 0.40 g was calculated for the SSE for a failure plane at elevation 680.0

Pseudostatic wedge analyses were performed for earthquake design Case 2 using seismic coefficients obtained from the finite element analysis, to determine the lateral extent of excavation required to obtain a minimum factor of safety equal to or greater than 1.0. The soil properties used in the pseudostatic wedge analyses are the same as in the static analyzes except the liquefiable sand is assumed to have no strength (Figure 2.5-237).

Section A-A, Figure 2.5-237 shows the wedge failure at elevation 650 which has a factor of safety equal to 1.12. The factor of safety increases for wedges considered further behind the crest of the slope.

Section B-B, Figure 2.5-237 shows the wedge failure at elevation 665. The factor of safety is 1.04 if the failure plane is in the firm gravel and is 1.57 if the failure plane occurs in the replacement material. The factor of safety increases for wedges considered further behind the crest of the slope. As a result of these analyses, excavation will be made down to firm gravel and laterally back to the point directly below the crest of the slope, and then to the surface with a slope not steeper than one vertical on 1.5 horizontal as shown in Figure 2.5-239.

Section C-C, Figure 2.5-237, shows a wedge analysis for failure at elevation 680 considering the replacement material in place and no strength in the in situ soil below elevation 680 behind the wedge. This wedge has a factor of safety of 2.77. If a larger wedge is considered at elevation 680, the factor of safety decreases since the driving force increases, but the resistance to sliding remains constant. If a wedge is considered approximately 160-feet back from the crest of the slope, the factor of safety would be 1.0. This wedge is shown in Figure 2.5-240. For this failure wedge to occur, the silty sand material must completely liquefy about 160 feet back from the crest of the slope. This is very unlikely, but using the method outlined by Newmark in Geotechnique Volume 15, 1965, the horizontal displacements for this wedge were evaluated. The following equation was used to evaluate the displacement:

$$U = \frac{6V^2}{2gN}$$

where:

U= displacement

V = velocity

g = acceleration of gravity

N = coefficient which when multiplied by weight of sliding mass results in resistance to sliding

As outlined previously, the seismic coefficient used for this wedge was 0.4 g. The velocity corresponding to this acceleration was evaluated by using Newmark's standard earthquake[94] where 0.5 g is related to a velocity of 24-inches per second. The horizontal displacement calculated in this manner is less than 1-foot for the Safe Shutdown Earthquake, which would not obstruct the intake channel.

Section B-B, Figure 2.5-237, also shows the results of investigating the sensitivity of the factor of safety through varying the strength properties of the firm gravel. By increasing the friction angle by 3 degrees or increasing cohesion by 100 psf, the factor of safety increases by about 0.1.

A dike will be left in place at the reservoir end of the channel while excavation and replacement in the channel is accomplished in the dry. When the dike is removed, some of the excavation and replacement will be under water. Rockfill will be used as the replacement material in this area, to the same one on four side slope. The strength of the rockfill is assumed to be $\varphi = 45°$ and c = 0.0. The factors of safety for all wedges considering the above strength are reported below in Section 2.5.5.2.2.

2.5.5.2.2 Additional Analyses Due to Unexpected Soil Conditions Encountered During Excavation of the Intake Channel

Description of Condition

Section 2.5.5.2.1 describes the design criteria and analyses performed for qualification of the intake channel side slopes. Figure 2.5-239 shows a typical cross-section to which the channel was to have been constructed, including a layer of basal gravel approximately 15-feet thick extending from elevation 665 to elevation 650.

During the excavation of the channel, TVA construction personnel encountered unexpected soil conditions in the layer of firm gravel. Therefore, test trenches and pits were excavated into the firm gravel to better define the soil conditions. On the upstream side of the channel, conditions were as expected except that from the pumping station to about halfway to the river, top of rock was determined to be at about elevation 663. Therefore, excavation in this area was made to top of rock, and about 18 inches of granular fill compacted to 85% maximum relative density was placed to provide a dry working base for placement of the compacted fill. The strength characteristics of the granular fill are better than the basal gravel and the compacted earthfill, and no additional design and analysis was required. On the downstream side of the channel, layers of sand and one layer of clay were found to exist in the firm gravel. From the pumping station to about halfway to the river, top of rock was found to be at about elevation 656. It was decided to excavate down to rock in this area and place the layer of granular fill (if needed to obtain a dry base) and then compacted earthfill as originally planned. Additional stability analyses have been made to verify the limits of excavation and are presented below. In the remainder of the downstream side, difficulties were encountered in excavating the trenches and test pits to top of rock due to the water table.

Samples of the sand and clay material in this area were collected by TVA's Singleton Materials Laboratory for evaluation. Preliminary examination of the sandy material by the soils laboratory and comparison of its characteristics with the empirical rules concerning evaluation of liquefaction potential outlined in Section 2.5.4.8 indicated a possibility for liquefaction during a seismic event.

Accordingly, a program of additional soils borings was formulated to determine the lateral and vertical extent of the sand and clay layers and to better define top of rock.

Figure 2.5-252 is a plan view of the channel which shows the locations of the additional soils borings. Figures 2.5-253, 2.5-254, and 2.5-255 show the soils profile obtained from the additional borings. The exploration program determined that the lowest bedrock elevation occurred near the mouth of the channel downstream side at elevation 650. In addition, a program of cyclic triaxial testing of the sandy material and static testing of the clay material, under R conditions (saturated, consolidated, undrained) in both cases, was instituted. Sandy material from two representative locations was tested at TVA's Singleton Materials Laboratory. The results of that testing are presented in Table 2.5-26. The results of static R testing on the clay material are shown in Table 2.5-27. The results of the exploration and testing program were evaluated to determine the need for additional analysis. These results indicated a probable liquefaction of the sand layer during a seismic event. In addition, the strength properties of the clay layer were too low to stabilize overlying slopes. Additional analyses have been made to determine new limits of excavation to top of rock for the downstream side of the channel extending from the reservoir to approximately halfway to the pumping station.

Additional Analyses

As outlined above, additional stability analyses were made for those portions of the downstream side of the channel with bedrock elevations ranging from 656 (approximately halfway to the reservoir) to 650 at the reservoir end of the channel. The analyses assumed that the excavated material would be compacted and placed as fill in the same manner as that used in other areas of the intake channel. The strength properties of the remolded material are $\varphi = 15°$ and c = 1200 psf, the same values used in the original analysis, as determined by tests on the remolded soil. The liquefiable material adjoining the remolded slopes is assumed to have no strength. The most critical design case has been established above to be that for sudden drawdown plus an SSE, for which the minimum factor of safety is 1.0. Therefore, the results presented below are for that case only.

From bedrock elevation 660 to elevation 656 the limits of excavation will be as shown in Figure 2.5-256. The factor of safety for a wedge failure along a plane at 656 is 1.12. The slope is therefore stable against failure by sliding.

Figure 2.5-257 shows the limits of excavation for a section with a bedrock elevation of 650. The factor of safety for a wedge failure along a plane at elevation 650 is 1.0. This factor of safety is considered adequate, since it was computed with the use of extremely conservative assumptions. As shown on Figure 2.5-257, the factor of safety was computed assuming that the entire zone of sandy material extending from elevation 680 to 650 liquefies completely during a seismic event. This is a very conservative assumption. Furthermore, the assumption has been made that no shear strength exists along the failure plane where it passes through the sandy zone; again, this is a very conservative assumption. Even a small amount of shear strength in the liquefiable zone along the failure plane would make the safety factor greater than 1.0.

The final configuration of the rockfill side slopes at the reservoir end of the intake channel, as discussed in Section 2.5.5.2.1, are also affected by the unexpected solid conditions encountered. On the upstream side of the mouth of the intake channel the firm gravel layer will be left in place and rockfill placed on top of it from elevation 665 to 695. On the downstream side the rockfill will be placed on bedrock down to elevation 650.

Figure 2.5-258 shows a typical cross section of the rockfill slopes on the upstream side of the channel. The factor of safety against sliding along a plane at elevation 665 is 1.5.

The downstream side of the channel with rockfill placed on a bedrock elevation of 650 is shown in Figure 2.5-259. The factor of safety for a wedge failure at 650 is 1.30, and the slope is therefore stable.

2.5.5.2.3 Design Criteria and Analysis for the Underground Barrier for the ERCW Pipeline and 1E Conduit Alignment

The location of the underground barrier is shown on Figure 2.5-580. The underground barrier was analyzed for the following cases:

Case	Required Factor of Safety
(1) During earthquake but prior to liquefaction (reduced passive pressure assumed to act)	1.0
(2) After earthquake and after liquefaction (no passive pressure assumed)	1.0

Figure 2.5-583 is a summary of the analysis of the underground barrier. The figure shows: a loading diagram of how the underground barrier was analyzed, a summary of the design parameter and criteria used in the stability analyses, and a summary of results of the stability analysis for each cross-section. Figure 2.5-584 is a plan of the area showing the locations of the as-built cross-sections.

As shown in the summary of the design parameters and criteria, the shear strengths of the alluvial sands (i.e., potentially liquefiable sands) have been assumed to be reduced during the earthquake. This was done to acknowledge the possibility that some strength loss in alluvial sands may occur during the earthquake. The magnitudes of the strength reduction, 50% of cohesion and 30% of angle of internal friction, was based on engineering judgement and is considered reasonable and conservative for the material.

The results of the stability analysis for each cross-section are provided for two sets of analyses representing "during earthquake" and "after earthquake" conditions for different potential failure planes. The "during earthquake" analyses show the stability of the barrier when the barrier mass is subjected to the peak acceleration, complete liquefaction of sands for the active earth pressure, and consider partial passive (reduced) earth resistance. The "after earthquake" analyses show the stability of the barrier after the earthquake and consider complete (postulated) liquefaction of the saturated alluvial sands for the active earth pressure and complete loss of downstream passive resistance. Depending on the section geometry and materials used as backfill, between one and five assumed failure planes were analyzed for each cross-section and design case.

Due to the need to complete the construction of the barrier prior to fuel load, the trench excavation was started prior to completion of the laboratory testing of the backfill soils. The barrier width was based on assumed design soil properties. The results of the evaluation of the initial laboratory shear strength test showed that the design cohesion was approximately half the needed cohesion to stabilize the barrier. To eliminate the need to widen the barrier, additional laboratory shear strength tests were made on backfill soils remolded to a higher level (100% Standard Compaction ASTM D 698) of compaction. The results of this testing showed that the cohesion was increased sufficiently to allow the barrier to be stable. The soil test results are presented in section 2.5.4.5.1. Since it was not necessary for the entire barrier to be constructed at the higher compaction level (100%), additional analyses were made to determine what elevation the lower compaction level (95%) could be used.

During excavation of the south end (near the pump station) of trench B, the depth of the trench was deeper than the exploratory borings had indicated. Stability analysis performed at the time the excavation was open indicated a need for a higher shear strength for the backfill material or to make the trench significantly wider. The option of using 1075 crushed stone with higher shear strength properties in lieu of earthfill A1 was selected. The properties of the 1075 crushed stone are shown in Figure 2.5-583. The use of the crushed stone met the higher shear strength requirements and provided the additional stability needed for actual field conditions. Also, since the construction period for the trench B barrier extended through a winter season, the option to use crushed stone facilitated completion of construction. Each change of backfill material in a cross-section presented a potential failure plane which was checked in the analysis.

Section 2.5.4.6 describes the study made to determine the design groundwater elevation for the piping and conduit alignments. As discussed in Section 2.5.4.6, the groundwater level was revised to reflect a 25-year groundwater. The influence of this higher groundwater on the analysis of the underground barrier was discussed with the NRC Staff. The Staff indicated they concur with TVA's judgment that the higher groundwater level will have a minimal effect on the results of the stability analysis, thus not requiring any additional evaluation of the stability of the underground barrier.

Figures 2.5-594 through 2.5-597 are representative cross-sections along the centerline of trench A. As shown on Figure 2.5-583, the results of a stability analysis for station 6+78 of trench A are not provided because the soil profile was not identified above the top of shale. This is not considered critical to the overall summary since the other 17 of the 18 cross-sections of trench A were analyzed and found to be adequate. The stability results are provided for two different potential failure planes, which are shown on the representative cross-sections (Figures 2.5-594 through 2.5-597) at (1) the top of weathered shale (A) and (2) the interface between the 95% and 100% maximum dry density fill (B). Figures 2.5-598 through 2.5-601 provide the summaries of in-place density and moisture-quality control tests conducted on the fill materials during construction of trench A.

Figures 2.5-602 through 2.5-605 are representative cross-sections along the centerline of trench B. Since trench B was backfilled with compacted crushed stone in addition to earthfill, additional potential failure planes were identified at the various material interfaces and analyzed. The stability summary on Figure 2.5-583 provides the results on two of these potential failure planes. The first, at the top of weathered shale (A), is provided for each section. The second, at one of the other potential failure planes, represents the lowest factor of safety for that cross-section other than at the top of weathered shale. Figure 2.5-606 through 2.5-610 provide the summaries of in-place density and moisture quality control tests conducted on the fill material during construction of trench B.

Figures 2.5-608 and 2.5-609 show that one quality control test had results that did not meet the required criteria for backfill in trench B compacted to 100% of maximum dry density. This failure to meet criteria was identified after trench construction was completed and an NCR (5804) was issued. The failure to meet required criteria resulted from the inadvertent use of the improper compaction control curve during construction. This resulted in the test sample being undercompacted by 1.3% and having too high a moisture content by 0.7% . The fill represented by the test sample was located near the top of the 100% maximum dry density backfill zone. This location is not critical to the analysis results. Therefore, the disposition of the NCR was use-as-is.

Figure 2.5-584 shows the final grading for the area of the underground barrier. Analyses of the underground barrier reveal that the as-built barrier meets or exceeds all design requirements.

2.5.5.3 Logs of Borings

Refer to Section 2.5.4.3 for the location of all in situ soil borings. Refer to Section 2.5.1.2.6 for the location of all rock borings.

2.5.5.4 Compaction Specifications

The compaction specification for earth and rock fills are discussed in Section 2.5.4.5.

2.5.6 Embankments

There are no embankments at the site which are used for plant flood protection or for impounding cooling water required for the operation of the nuclear power plant.

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Codes

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Standards

ASTM - American Society for Testing and Materials

ASTM C 33-90Concrete Aggregates

ASTM C 88-83Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate

ASTM C 131-89Standard Test Method for Resistance to Degradation of Small Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine

ASTM C 535-89Standard Test Method for Resistance to Degradation of Large Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine

ASTM D 422-63Standard Method for Particle-Size Analysis of Soils

ASTM D 653-90Standard Terminology Relating to Soil, Rock, and Contained Fluids

ASTM D 698-78Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 5.5-Pound (2.49-kg) Rammer and 12-Inch (305 mm) Drop

ASTM D 1556-82Standard Test Method for Density of Soils in Place by the Sand-Cone Method

ASTM D 2167-84Standard Test Method for Density and Unit Weight of Soil in Place by the Rubber-Balloon Method

ASTM D 2216-80Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures

ASTM D 2487-85Standard Test Method for Classification of Soils for Engineering Purposes

ASTM D 2922-81Standard Test Methods for Density of Soil and Soil Aggregate in Place By Nuclear Methods (shallow depth)

ASTM D 4253-83Standard Test Methods for Maximum Index Density of Soils Using a Vibratory Table

ASTM D 4254-83Standard Test Methods for Minimum Index Density of Soils and Calculation of Relative Density

ASTM D 4318-84Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

ASTM D 4318-84Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

Table 2.5-1 Soil Strength Tests

	UC		Q		Cyclic		R	S	Consolidation
Un*-Undistrubed									
Re*-Remolded	Un*	Re*	Un*	Re*	Q	R			
Transformer Yard Switchyard	X								
Cooling Tower Area	X								X
CCW Pumping Station									
Intake Channel	X	X	X	X	X	X	X		
Diesel Generator Building	X	X	X	X					X
AE Conduits Alignment	X	X	X		X[1]	X	X		
ERCW Piping Alignment	X	X	X		X[1]	X	X		
Office and Service Building									

X indicates this type of soil test was conducted on this feature

1. Q-cyclic to be run if some soils show liquefaction potential

Table 2.5-2 Watts Bar Nuclear Plant Soil Investigation 500-kv Transformer Yard Summary Of Laboratory Test Data

Elevation	Soil Symbol	Nat. Moist.	Grain Size Analysis: Gravel	Sand	Silt	Clay	D_{10}	Atterberg Limits: Liquid Limit	Plasticity Index	Dry Density	Void Ratio	Unconfirmed Compression: Undisturbed	Remolded	Sensitivity Ratio
			%	%	%	%	mm	%	%	pcf		tsf	tsf	
Boring US-1; station 66+34, P+17; surface elevation 743.2														
730.2 - 729.2	SM	16.0	0	57	24	19	--	25.9	3.3	106.2	0.59			
727.7 - 725.8	ML	19.6	1	48	27	24	--	29.6	5.2	105.1	0.61	2.2	1.3	1.7
725.0 - 722.8	SM	17.6	6	54	25	15	--	27.6	4.6	106.5	0.59			
721.6 - 720.7	G-SM	15.4	20	46	21	13	--	27.0	3.7	112.2	0.51			
720.7 - 719.8	SM	8.7	4	77	13	6	.025	NP	NP	108.9	0.55			
718.9 - 716.8	G-SM	12.8	12	69	14	5	.025	NP	NP	111.8	0.50			
716.2 - 714.7	SM	29.0	0	82	13	5	.026	NP	NP	94.5	0.78			
714.7 - 714.1	ML	33.3	0	40	40	20	--	29.2	3.0	87.9	0.92			
Boring US-2; station 66+84, M+82; surface elevation 742.1														
729.4 - 728.9	MH	23.2	0	21	35	44	--	53.1	23.1	98.2	0.75	1.5	1.4	1.1
728.9 - 728.1	SC	20.5	0	51	36	13	--	31.3	8.4	102.8	0.63			
726.3 - 725.5	SC	17.8	0	52	27	21	--	36.5	13.0	105.4	0.62			
725.5 - 724.8	SM	11.5	8	70	18	4	.025	NP	NP	113.0	0.48			
724.0 - 721.7	SM	16.8	0	66	26	8	--	NP	NP	96.5	0.74			
720.2 - 718.9	SM	14.8	6	60	22	12	--	NP	NP	107.3	0.56			
718.1 - 717.4	SM	16.5	9	62	21	8	--	NP	NP	107.9	0.59			

Table 2.5-3 WATTS BAR NUCLEAR PLANT SOIL INVESTIGATION 500-KV TRANSFORMER YARD SUMMARY OF LABORATORY TEST DATA

Elevation	Soil Symbol	Nat. Moist.	Grain Size Analysis Gravel	Grain Size Analysis Sand	Grain Size Analysis Silt	Grain Size Analysis Clay	D_{10}	Atterberg Limits Liquid Limit	Atterberg Limits Plasticity Index	Dry Density	Void Ratio
			%	%	%	%	mm	%	%	pcf	
Boring US-3; station 69+72, O+60; surface elevation 733.2											
729.2 - 728.0	SM	15.3	0	63	14	23	--	36.0	11.6	112.5	0.50
726.7 - 725.7	G-SM	16.6	13	43	24	20	--	32.4	8.6	112.7	0.50
724.2 - 723.3	G-SM	13.9	26	45	15	14	--	NP	NP	111.6	0.49
723.3 - 722.8	G-SM	13.2	28	49	13	10	--	NP	NP	93.8	0.78
721.1 - 720.6	G-SM	16.2	14	55	15	16	--	25.5	0.8	105.3	0.59
717.2 - 716.6	G-SM	15.8	17	54	17	12	--	23.8	2.7	107.8	0.56

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Table 2.5-4 WATTS BAR NUCLEAR PLANT SOIL INVESTIGATION NORTH COOLING TOWER SUMMARY OF LABORATORY TEST DATA

								Atterberg Limits								
	Soil	Nat.	Grain Size Analysis					Liquid	Plasticity	Dry	Void	Unconfirmed Compression		Sensitivity	Consolidation	
Elevation	Symbol	Moist.	Gravel	Sand	Silt	Clay	D_{10}	Limit	Index	Density	Ratio	Undisturbed	Remolded	Ratio	Cc	Pc
			%	%	%	%	mm	%	%	pcf		tsf	tsf			tsf
Boring US-7; station 60+98, D+16; surface elevation 735.92																
730.8-729.6	MH	27.3	0	30	29	41	--	53.3	20.0	97.1	0.79	4.1	4.8	0.9	0.08	2.68
727.4-726.0	ML	24.2	0	44	29	27	--	35.9	10.2	100.9	0.68	3.4	4.1	0.8	0.20	4.11
724.4-723.9	ML	26.6	0	35	36	29	--	37.6	11.5	97.8	0.72	1.1	1.0	1.1		
723.9-722.6	SM	21.1	3	67	22	8	.007	NP	NP	96.5	0.76				0.31	--
721.8-719.5	SM	25.9	0	63	24	13	--	NP	NP	94.2	0.80					
718.8-718.0	ML	29.4	0	49	31	20	--	30.1	5.6	91.1	0.80				0.22	1.01
718.0-716.6	SM	19.3	0	88	10	2	.06	NP	NP	95.8	0.78					
716.0-714.8	SM	27.6	0	82	13	5	.02	NP	NP	98.0	0.73				0.09	--
Boring US-8; station 61+86, A-51; surface elevation 733.7																
731.4-730.6	CL	17.0	0	25	47	28	--	26.8	10.8	107.3	0.55	2.5	2.4	1.0		
730.6-728.3	MH	26.2	0	28	21	51	--	53.9	19.6	99.4	0.70	6.1	6.1	1.0	0.09	3.5
728.0-727.3	MH	28.9	0	21	26	53	--	58.4	26.0	94.9	0.80	4.8	3.9	1.2	0.07	3.4
727.3-726.3	SM	18.2	0	56	22	22	--	32.4	6.2	108.3	0.57				0.14	--
724.7-723.3	ML	20.2	0	49	23	28	--	36.5	11.4	105.5	0.62	2.6	2.3	1.1	0.15	1.9
721.7-720.8	CL	23.1	0	36	31	33	--	41.2	17.0	102.6	0.66	2.9	2.7	1.1	0.15	2.7
720.8-720.3	SM	18.0	0	55	28	18	--	28.3	3.5	106.0	0.60					
719.1-718.6	SM	19.9	0	53	24	23	--	30.2	7.1	105.4	0.62					
718.6-718.1	SM	17.4	0	57	26	17	--	29.5	6.1	105.8	0.59				0.11	--
718.1-717.9	G-SM	11.8	37	37	16	10	--	NP	NP	--	--					
716.4-715.6	G-SP-SM	7.4	34	57	7	2	.09	NP	NP	--	--					

Table 2.5-5 WATTS BAR NUCLEAR PLANT SOIL INVESTIGATION SOUTH COOLING TOWER SUMMARY OF LABORATORY TEST DATA

	Soil	Nat.	Grain Size Analysis					Atterberg Limits		Dry	Void	Unconfirmed Compression		Sensitivity	Consolidation	
								Liquid	Plasticity							
Elevation	Symbol	Moist.	Gravel	Sand	Silt	Clay	D_{10}	Limit	Index	Density	Ratio	Undisturbed	Remolded	Ratio	Cc	Pc
			%	%	%	%	mm	%	%	pcf		tsf	tsf			tsf
Boring US-12; station 60+08, E+65; surface elevation 736.8																
729.8-729.0	CL	27.1	0	22	39	39	--	48.3	22.8	97.0	0.94	3.6	2.1	1.7	0.18	3.11
729.0-728.4	ML	25.8	0	30	43	27	--	40.1	12.4	98.9	0.72					
726.7-725.9	CH	27.0	0	28	34	38	--	53.6	27.3	97.5	0.73	1.9	1.9	1.0	0.17	2.8
725.9-725.4	SM	18.2	0	55	29	16	--	NP	NP	104.8	0.60					
723.8-722.4	SM	23.0	0	61	21	18	--	29.8	4.9	102.0	0.66					
721.7-720.7	SM	24.8	0	54	23	23	--	33.3	5.5	97.8	0.74				0.15	--
720.7-719.7	SM	31.1	0	82	14	4	.03	NP	NP	86.5	0.56					
718.8-716.5	MH	35.0	0	6	59	35	--	50.4	17.6	87.3	0.97	0.4	0.3	1.3	0.36	2.6
715.8-714.9	ML	31.6	0	34	39	28	--	38.8	9.5	88.9	0.92	0.6	0.5	1.2		
714.9-713.5	SM	25.3	0	88	11	7	.02	NP	NP	90.9	0.87				0.17	--
Boring US-13; station 68+86, A+91; surface elevation 741.2																
731.1-730.2	CH	27.2	0	21	34	45	--	50.2	26.6	94.9	0.77					
730.2-728.8	CL	23.1	0	49	27	24	--	32.1	8.8	101.4	0.64	1.9	1.2	1.6	0.14	1.8
728.2-725.8	ML	26.6	0	45	35	20	--	30.9	6.7	98.6	0.70					
725.2-722.8	ML	31.2	0	22	44	34	--	42.7	14.7	91.3	0.88	0.9	0.6	1.5	0.20	1.3
722.2-719.9	ML	32.9	0	35	40	25	--	36.6	11.1	89.1	0.89					
719.2-717.3	CL	32.1	0	32	41	27	--	36.4	13.1	88.3	0.93					
717.3-716.9	ML	29.6	0	21	44	35	--	44.4	17.4	94.2	0.82	1.4	0.6	2.3		
716.2-715.3	ML	30.2	0	21	44	35	--	46.7	17.9	93.4	0.85	0.9	0.9	1.0	0.24	1.4
715.3-713.8	SC	25.0	0	51	28	21	--	26.6	8.1	99.8	0.69					
713.2-712.6	CL	25.4	0	35	37	28	--	30.2	12.8	99.4	0.70	0.4	0.3	1.3		
712.6-712.1	CL	26.1	0	48	31	21	--	27.3	7.8	99.6	0.70				0.12	2.3

Table 2.5-6 Watts Bar Nuclear Plant Soil Investigation Diesel Generator Building Summary of Laboratory Test Data

												Atterb. Limits				Undisturbed		Remolded		Undis.	Remolded			
	Soil	Soil	Nat.	Std.	Grain Size Analysis**							Liquid	Plastic.	Dry	Void	Triaxial Q		Triaxial Q		Uncon.	Unconf.	Sensitivity	Consolidation	
Elevation	Symbol	Type	Moist.	Penet.	Sand	Silt	Clay	D_{10}	D_{50}	D_{60}	C_u	Limit	Index	Dens.	Ratio	φ	c	φ	c	Compr.	Compr.	Ratio	C_c	P_c
			%	***	%	%	%	mm	mm	mm		%	%	pcf		deg	tsf	deg	tsf	tsf	tsf			tsf
Boring US-25; surface elevation 734.6																								
720.1-717.9	CL	J	31.9	6	3	51	46	--				48.2	25.5	90.2	1.063	1.5	1.22	0.0	0.85	1.96	0.71	2.8		
717.1-715.5	CL	I	32.9	6	18	45	37	--				36.0	18.6	89.4	0.887	0.8	0.38	0.0	0.05	0.81	0.18	4.5	0.33	2.0
715.5-715.1	SM	K	31.0	2	69	19	12	.002	.13	.14	70.0	NP	NP	89.7	0.868	3.3	0.53							
714.6-713.0	SP-SM	L	29.3	2	89	5	6	.05	.28	.29	5.8	NP	NP	90.8	0.870									
Boring US-26; surface elevation 735.0																								
720.0-717.6	CL	J	31.2	7	8	49	43	--				47.8	25.4	90.5	0.883	4.7	0.28	0.5	0.20	.091	0.31	2.9	0.30	2.6
717.0-715.8	CL	I	30.4	3	27	45	28	--				31.9	14.0	91.0	0.861	1.2	0.43	0.0	0.13	.081	0.17	4.8	0.32	1.8
715.8-714.5	SM	K	30.9	4	62	29	9	.006	.09	.12	20.0	NP	NP	89.7	0.884	6.7	0.57						0.18	1.2
714.0-711.7	ML-CL	H	30.1	4	46	34	20	--				26.5	5.6	90.0	0.888	7.5	0.55	4.0	0.25	0.23	0.05	4.6	0.21	1.5
Boring US-28 surface elevation 734.0																								
720.0-718.1	CL	J	28.6	7	9	51	40	--				46.0	22.9	94.3	0.803	8.5	1.30	7.5	0.55	2.30	0.82	2.8		
717.0-714.6	CL	J	35.0	2	10	48	42	--				45.0	25.0	85.9	0.963	3.0	0.25	1.0	0.10	0.72	0.22	3.3	0.22	2.0
714.0-712.5	CL	H	32.1	3	37	42	21	--				27.3	8.2	88.2	0.903	4.6	0.20*			0.35	0.05	7.0	0.44	1.7

* After remodeling, the test specimens slumped under their own weight and could not be tested.

** No gravel in samples taken.

*** Standard penetration blows per foot in adjacent split-spoon boring.

Undisturbed borings made from building foundation (El 720) to top of firm gravel.

Water table El 727±.

Table 2.5-7 Watts Bar Nuclear Plant
Soil Investigation
Essential Raw Cooling Water Supply
Summary of Laboratory Test Data

								Atterberg Limits								Apparent		Effective		
	Soil	Nat.	Grain Size Analysis					Liquid	Plasticity	Dry	Void	Triaxial Q		Triaxial R		Triaxial R		Unconfined	Compression	Sensitivity
Elevation	Symbol	Moist.	Gravel	Sand	Silt	Clay	D_{10}	Limit	Index	Dens.	Ratio	φ	c	φ	c	φ	c	Undisturbed	Remolded	Ratio
		%	%	%	%	%	mm	%	%	pcf		deg.	tsf	deg.	tsf	deg.	tsf	tsf	tsf	
Boring SS-29; station 88+35, E+03; surface elevation 699.4																				
685.4-683.0	ML-CL	24.5	0	48	32	20	--	24.5	4.1	96.4	0.73	4.2	0.95	12.0	0.25	27.7	0.08	0.3	0.4	0.8
862.0-681.1	ML	17.6	0	48	37	15	--	NP	NP	99.5	0.69									
681.1-680.1	CL	24.4	0	30	45	25	--	29.2	7.4	96.3	0.74							0.4	0.3	1.3
679.4-677.0	SM	29.2	0	82	11	7	0.2	NP	NP	91.6	0.84	21.3	0.43	17.0	0.22	32.5	0.03			
676.4-675.1	SM	29.2	0	82	11	7	.02	NP	NP	91.6	0.84	21.3	0.43	17.0	0.22	32.5	0.03			
675.1-674-0	SM	29.0	0	67	21	12	--	NP	NP	94.6	0.78									
673.4-671.0	SM	30.0	0	69	20	11	--	NP	NP	93.0	0.80									
667.4-665.5	SM	26.6	0	61	25	14	--	NP	NP	96.0	0.76	12.0	1.40							

Table 2.5-8 Watts Bar Nuclear Plant
Intake Channel
Soil Investigation
Summary of Laboratory Test Data
(Sheet 1 of 2)

Elevation	Soil Symbol	Soil Type	Nat. Moist. %	Std. Penet. **	Grain Size Analysis* Sand %	Silt %	Clay %	D_{10} mm	D_{50} mm	D_{60} mm	C_u	Atterb. Limit Liq. Limit %	Plastic. Index %	Dry Dens. pcf	Void Ratio	Undisturbed Triaxial Q φ deg	c tsf	Remolded Triaxial Q φ deg	c tsf	Undisturbed Triaxial R φ deg	c tsf	Undist. Unconf Compr. tsf	Remolded Unconf. Compr. tsf	Sensitivity Ratio
Boring US-25; surface elevation 691.5																								
689.5-687.3	CL	D	28.9	6	16	45	49	--				46.2	20.4	89.6	0.897	6.5	0.50			10.5	0.85	0.66	0.81	0.8
686.5-684.7	CL	D	27.5	6	13	33	54	--				46.3	25.3	93.5	0.796	13.0	0.80			14.0	0.41	1.22	1.37	0.9
683.5-681.4	CL	c	24.3	13	5	45	50	--				41.2	20.9	102.4	0.634									
680.5-678.0	SM	g	28.3	6	85	11	4	.03	.160	.170	5.7	NP	NP	94.5	.771									
677.5-676.5	CL	a	25.3	10	15	48	35	--				30.3	12.2	99.9	.687									
676.5-675.1	SM	G	25.7	1	73	16	11	.004	.160	.170	42.2	NP	NP	95.4	0.767					35.0	0.00			
674.5-673.1	SM	g	25.1	0	71	20	9	.006	.110	.130	21.7	NP	NP	97.8	.716									
671.5-669.0	SM-SC	f	26.6	0	50	30	20	--				22.2	4.7	95.1	.767									
Boring US-35 surface elevation 704.4																								
702.4-700.3	CL	C	27.0	6	21	43	36	--				42.6	18.1	93.3	0.814	11.5	0.75			9.0	0.60	0.68	0.66	1.0
699.4-697.2	CL	C	22.6	12	19	47	34	--				39.0	15.2	100.3	0.719	21.7	1.23			24.5	0.70	2.07	1.86	1.1
696.4-695.0	CL	C	22.9	21	19	48	33	--				38.2	14.6	101.0	0.682	21.5	1.50			16.3	1.30	3.67	2.55	1.4
693.4-691.7	ML	E	22.2	11	39	37	24	--				34.0	10.1	95.4	0.788	25.5	0.85	24.0	0.50	9.0	0.87	1.68	0.69	2.4
690.4-688.7	SM	F	25.8	10	51	33	16	.0015	.081	.100	66.7	NP	NP	94.7	0.800					9.5	0.80			
687.4-685.4	ML	E	23.9	10	37	43	20	--				29.3	4.0	97.5	0.751	16.5	1.10			11.9	0.45	0.97	0.56	1.7
684.4-683.0	SM	G	25.1	8	79	10	11	.0035	.190	.230	65.7	NP	NP	93.0	0.801	31.0	0.40							
Boring US-40; surface elevation 693.4																								
691.4-690.6	SM	f	27.7	2	50	34	16	.002	.075	.090	45.0	27.4	3.7	87.4	.907									

Table 2.5-8 Watts Bar Nuclear Plant
Intake Channel
Soil Investigation
Summary of Laboratory Test Data
(Sheet 2 of 2)

690.6-689.5	SM	f	21.6	2	56	29	15	.0025	.091	.120	48.0	21.1	0.5	92.7	.802									
688.4-686.5	SM-SC	F	28.4	2	54	26	20	.001	.090	.130	130.0	26.3	6.0	86.4	0.952	20.5	0.35			14.5	0.00			
685.4-684.4	SM	F	25.9	3	56	25	19	.001	.093	.140	140.0	22.4	3.0	92.8	0.807	7.5	0.22			15.5	0.00			
684.4-683.4	CL	B	25.7	3	42	31	27	--				29.2	10.8	93.7	0.789	2.5	0.60	2.5	0.20	11.5	0.24	0.48	0.19	2.5
682.4-680.4	CL	D	27.4	9	21	41	38	--				40.5	17.7	93.9	0.795	3.5	1.06			11.5	0.82	1.29	0.68	1.9
679.4-677.0	SM	G	26.9	4	85	9	6	.03	.180	.200	6.7	NP	NP	94.8	0.792					40.5	0.65			
676.4-674.5	SM	G	26.1	2	76	17	7	.012	.150	.175	14.6	NP	NP	95.7	0.778	36.0	0.25			38.0	0.13			
673.4-671.6	SM	g	24.5	7	78	14	8	.012	.160	.170	14.2	NP	NP	100.4	.684									
670.4-669.4	CL	a	23.5	6	35	37	28	--				30.0	11.3	103.1	.647									
669.4-668.3	SM	g	31.1	7	82	12	6	.030	.170	.170	5.7	NP	NP	99.9	.669									

* No gravel in samples taken.
** Standard penetration blows per foot in adjacent split-spooon boring.
Undisturbed borings made to top of firm gravel.
Water table El 685±.

Table 2.5-9 Watts Bar Nuclear Plant
Intake Channel
Soil Investigation
Summary of Laboratory Test Data
(Sheet 1 of 2)

Elevation	Soil Symbol	Soil Type	Nat. Moist.	Std. Penet.	Grain Size Analysis* Sand	Silt	Clay	D_{10}	D_{50}	D_{60}	C_u	Atterb. Limit Liq. Limit	Plastic. Index	Dry Dens.	Void Ratio	Undisturbed Triaxial Q φ	c	Remolded Triaxial Q φ	c	Undisturbed Triaxial R φ	c	Undist. Unconf Compr.	Remolded Unconf. Compr.	Sensitivity Ratio
			%	**	%	%	%	mm	mm	mm		%	%	pcf		deg	tsf	deg	tsf	deg	tsf	tsf	tsf	
Boring US-41; surface elevation 705.9																								
703.9-701.5	CL	A	19.5	10	30	43	27					30.3	11.3	104.1	0.644	20.5	0.80			15.1	0.34	1.03	0.80	1.3
700.9-699.6	CL	A	20.4	11	49	23	28					34.4	11.0	104.5	0.632	28.5	0.40					1.24	1.31	1.0
697.9-696.3	CL	A	25.1	12	29	42	29					37.2	12.7	96.4	0.749	18.5	1.35			24.0	0.22	1.41	1.18	1.2
694.9-692.2	ML	E	24.8	24	45	37	18					28.9	4.3	98.0	0.734	19.5	0.80			17.5	0.38	0.85	0.45	1.9
689.9-688.3	SM	F	27.6	12	61	20	19	.001	.0150	.190	190.0	NP	NP	91.9	0.916	19.0	0.50							
Boring US-46 surface elevation 694.3																								
692.3-690.9	CL	B	27.2	4	30	41	29					37.2	13.8	89.5	.870	15.0	0.45	7.5	0.40	13.2	0.34	0.55	0.22	2.5
689.3-688.2	SM	g	17.3	3	84	7	9	.01	.180	.190	19.0	NP	NP	-	-									
688.2-687.1	CL	b	27.6	4	40	32	28	-	-	-	-	30.5	11.4	90.3	.865									
686.3-683.5	CL	B	26.7	3	41	34	25	-	-	-	-	28.7	8.9	91.3	.846	3.5	0.25	1.5	0.15	13.3	0.27	0.25	0.11	2.3
683.3-681.3	CL	b	30.1	4	32	38	30	-	-	-	-	32.5	11.6	92.2	.814									
680.3-678.3	CL	B	29.9	5	22	46	32	-	-	-	-	39.3	17.3	90.1	.885	8.0	0.70			13.9	0.37	0.43	0.29	1.5
674.3-673.3	SM	G	24.4	4	80	11	9	.009	.170	.175	19.4	NP	NP	96.6	.764	33.3	0.15							
673.3-671.9	SM	f	23.9	3	54	29	17	.0001	.085	.110	-	NP	NP	98.5	.724									
671.3-668.9	SM	f	25.1	4	66	21	13	.002	.110	.160	80.0	NP	NP	98.6	.716									
668.3-666.3	SM	f	24.7	12	60	28	12	.001	.130	.170	170.0	NP	NP	101.4	.668									

Table 2.5-9 Watts Bar Nuclear Plant
Intake Channel
Soil Investigation
Summary of Laboratory Test Data
(Sheet 2 of 2)

* No gravel in samples taken.

** Standard penetration blows per foot in adjacent split-spoon boring.

Undisturbed borings made to top of firm gravel.

Water table El 685+.

Table 2.5-10 Watts Bar Nuclear Plant Class IE Conduits Soil Investigation Summary Of Laboratory Test Data

Elevation	Soil Symbol	Soil Type	Nat. Moist.		Std. Penetr.	Grain-Size Analysis Gravel	Sand	Silt	Clay	D10	Atterb. Limits Liq. Limit	Plastic Index	Dry Dens.	Void Ratio	Unconfined Compression Undis.	Rem.	Sens. Ratio	Triaxial Q Undisturbed ∅	c	Triaxial R Natural Moisture ∅	c	Direct Shear S ∅	c
			%	% Sat.		%	%	%	%	mm	%	%	pcf		tsf	tsf		deg.	tsf	deg.	tsf	deg.	tsf
Boring US-55, Surface El. 727.1																							
726.1-723.7	ML	D	21.2	86.2	15	0	23	37	40	--	49.8	21.5	101.3	0.655	1.8	1.5	1.2	25.2	1.00	28.8	0.35	23.0	0.80
723.1-721.4	ML	d	20.1	100.0	18	1	30	29	40	--	44.7	16.9	112.7	0.484									
720.1-717.8	CL	b	18.4	93.1	15	0	40	28	32	--	40.7	17.2	110.9	0.542									
717.1-715.1	CL	b	21.2	94.6	15	0	37	28	35	--	44.6	20.5	105.6	0.609									
714.1-712.5	ML	d	24.2	100.0	22	0	22	34	44	--	49.2	19.8	102.6	0.649									
711.1-710.2	CL	B	19.2	85.2	21	6	33	33	28	--	35.1	14.5	104.9	0.614	2.8	2.0	1.4	7.0	2.05			34.0	0.20
708.1-707.2	CL	b	21.8	95.1	23	0	33	35	32	--	44.3	18.5	105.0	0.629									
702.1-701.0	SC	I	19.0	85.8	20	0	54	25	21	--	34.0	11.1	106.5	0.603								30.0	0.27
699.8-697.8	CL	b	23.0	94.6	20	8	33	31	28	--	50.2	23.7	101.8	0.656									
697.1-695.3	G-SM*	I	19.5	83.5	25	15	51	22	12	0.004	40.5	13.5	105.8	0.655				12.5	0.56	8.5	1.75	27.0	1.14
Boring US-59, Sufrace El. 699.7																							
698.7-696.3	ML-CL	G	23.8	93.7	6	0	32	50	18	--	22.7	5.4	98.0	0.664	1.6	0.5	3.2	4.3	1.50	31.0	0.45	30.0	0.18
695.7-694.5	CL	C	32.7	87.8	2	0	24	56	20	--	27.8	7.7	83.1	0.977	0.2	0.1	2.0	3.0	0.15	29.0	0.06		
692.6-692.1	G-CL*	c	20.0	70.9	6	11	37	35	17	--	23.9	8.5	95.1	0.753									
692.1-690.3	CL*	B	23.6	81.8	6	6	31	50	13	--	36.1	12.6	94.5	0.780								21.0	0.87
689.7-688.3	GC*	f	17.1	92.3	20	29	25	32	14	--	39.8	15.7	113.5	0.507									
Boring US-61, Surface El. 700.7																							
699.7-697.3	SM	J	24.8	85.1	12	0	57	25	18	--	NP	NP	95.0	0.801				29.5	0.50	30.0	0.35		
696.7-694.3	SM	J	24.8	97.1	10	0	69	22	9	0.0075	NP	NP	100.4	0.692				15.0	1.32	32.3	0.55	35.5	0.00
693.7-692.6	G-SC*	i	21.1	82.1	24	32	36	24	8	0.0065	34.7	14.7	101.1	0.716									
692.6-692.2	G-SC*	i	17.8	--	24	31	35	22	12	0.003	37.2	13.2	--	--									

*Weathered shale.

Table 2.5-11 Watts Bar Nuclear Plant
Class IE Conduits
Soil Investigation
Summary of Laboratory Test Data

Elevation	Soil Symbol	Soil Type	Nat. Moist.		Std. Pene tr.	Grain-Size Analysis Gravel	Sand	Silt	Clay	D10	Atterb. Limits Liq. Limit	Plastic Index	Dry Dens.	Void Ratio	Unconfined Compression Undis.	Rem.	Sens. Ratio	Triaxial Q Undisturbed ∅	c	Triaxial R Natural Moisture ∅	c	Direct Shear S ∅	c
			%	% Sat.		%	%	%	%	mm	%	%	pcf		tsf	tsf		deg.	tsf	deg.	tsf	deg.	tsf
Boring US-51, Surface El. 724.4																							
722.4-720.0	CL	b	21.8	100.0	15	0	32	29	39	--	46.8	21.1	105.9	0.579									
719.4-717.1	MH	H	20.8	88.5	19	0	26	28	46	--	54.8	21.1	104.1	0.644	1.9	1.4	1.4	4.0	3.20	20.8	1.10	26.0	0.47
716.4-714.1	CL	b	21.5	96.7	18	0	31	32	37	--	46.4	20.4	105.8	0.605									
713.4-711.6	ML	d	22.1	98.4	17	0	30	32	38	--	47.4	18.9	105.4	0.610									
710.4-708.0	CL	B	23.2	93.7	14	0	20	38	42	--	49.5	21.7	101.3	0.670	2.7	2.0	1.4	11.4	1.35	28.5	0.58	31.5	0.15
707.4-706.1	ML	d	24.1	93.9	20	0	27	42	31	--	45.0	16.6	100.3	0.700									
704.4-703.8	CL	b	16.4	83.2	32	4	31	39	26	--	37.5	14.3	109.1	0.527									
Boring US-53, Sufrace El. 726.8																							
725.3-723.4	MH	H	22.5	93.7	16	0	12	45	43	--	56.4	23.1	103.2	0.653	8.8	5.4	1.6	23.0	3.00	19.4	1.57		
722.8-720.4	ML	D	24.8	92.9	13	0	18	38	44	--	49.2	19.8	99.7	0.741	4.3	3.2	1.3	5.5	2.55	21.0	1.57	31.0	0.30
719.8-717.4	CL	b	20.9	95.3	15	0	38	31	31	--	40.9	16.0	106.4	0.596									
716.8-714.7	CL	b	18.5	96.2	14	0	40	26	34	--	41.1	17.7	111.5	0.523									
713.8-712.1	ML	d	17.9	99.5	27	0	34	32	34	--	43.3	16.2	113.5	0.485									
710.8-710.0	MH	h	22.9	97.2	20	0	25	32	43	--	54.5	23.6	103.1	0.635									
707.8-706.7	CL	B	20.7	88.4	18	0	35	32	33	--	43.7	19.9	103.7	0.637	2.3	1.7	1.4	14.0	1.65	28.0	0.37	35.0	0.34
705.8-705.0	SC	i	17.2	76.2	34	7	76	7	10	0.008	29.2	7.1	104.0	0.602									

Table 2.5-12 Soil Design Values

	Q		R		*R		S		Vm	Vs
	φ	c(tsf)	φ	c(tsf)	φ	c(tsf)	φ	c(tsf)	(pcf)	(pcf
In-situ										
IE Conduits	7°	1.2					31°	0.25		
ERCW & HPFP Piping	18°	0.6					32°	0.20		
ERCW Pipline and IE conduits										
a. Sands			14°	0.2	28°	0.4			119	124
b. Fine Grained			14°	0.2	28°	0.4			120	123
Borrow Area 4	6°	1.05	16°	0.075	30°	1.0	32°	0	121	128

*R-test at natural moisture content for In-situ soil and at moist conditions for borrow.

Table 2.5-13 Surface Settlements (S) and Average Deformation Moduli (E) for Center of Flexible Circular Footings Loaded With 5 Ksf

		10-Foot-Dia Footing		50-Foot-Dia Footing		100-Foot-Dia Footing		200-Foot-Dia Footing	
		S ,	E	S ,	E	S ,	E	S ,	E
Hole	**Station**	**in.**	**psi x 10^3**	**in**	**psi x 10^3**	**in**	**psi x 10^3**	**in**	**psi x 10^3**
20	L-61	0.36	11	0.63	32	0.84	48	1.04	77
29	M-63	0.41	10	0.78	26	0.95	42	1.18	68
39	N-65	0.22	18	0.60	33	0.77	52	0.97	82
41	O-60	0.27	15	0.54	37	0.70	57	0.94	85
43	O-62	0.11	36	0.39	51	0.55	73	0.79	100
52	P-65	0.22	18	0.49	41	0.64	62	0.88	91
Average Hole		0.24	17	0.56	36	0.73	55	0.97	83
All Type 1 Rock		0.46	8.7	1.06	19	1.48	27	2.18	37
All Type 2 Rock		0.10	39	0.27	74	0.39	102	0.54	148
All Type 3 Rock		0.0082	1000	0.041	1000	0.082	1000	0.16	1000

	300-Foot-Dia Footing	
	S ,	E
Hole	in	psi x 10^3
Average	1.21	99

Table 2.5-14 Effect of Removing Top 10 Feet of Rock on Settlement of 10-foot Diameter Flexible Footing

		Surface Rock Included		Top 10 Feet of Rock Removed	
Hole	Station	S , inches	E , psi x 10^3	S , inches	E , psi x 10^3
20	L-61	0.36	11	0.15	27
29	M-63	0.41	10	0.15	27
39	N-65	0.22	18	0.14	28
41	0-60	0.27	15	0.09	44
43	0-62	0.11	36	0.11	36
52	P-65	0.22	18	0.10	40
Average Hole		0.24	17	0.12	33
Maximum Differential Settlement		0.30		0.06	

Table 2.5-15 Average In Situ Down-hole Soil Dynamics
Diesel Generator Building

Compressional Velocity Ft/Sec	Shear Velocity Ft/Sec	Dynamic Shear Modulus psi	Dynamic Young's Modulus psi
3459	1042	21,110	61,230
	Intake Channel		
3123	942	17,100	50,050

Table 2.5-16 Average Seismic Refraction Soil Dynamics Diesel Generator Building

Velocity Zones Elevations	Compressional Velocity Ft/Sec	Shear Velocity Ft/Sec	Dynamic Shear Modulus psi	Dynamic Young's Modulus psi
734-728	1250	599	7,000	18,915
728-715	3162	1382	37,885	104,725
715-695	6100	1660	53,530	156,310
		INTAKE CHANNEL		
695-678	1183	537	5,610	15,370
678-653	4917	1261	34,370	99,283

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Table 2.5-17 In-Situ Soil Dynamic Properties Watts Bar Nuclear Power Plant Class IE Conduits and ERCW Piping (Sheet 1 of 2)

Borehole Number	Surface Elevation (Feet)	Depth of Refusal (Feet)	φ	Hydrophone Depth (Feet) (Z)			Hole Axle To Energy Source (Feet) (X)	Compressional Velocity (Measured) (Ft/Sec)			Shear Velocity (Measured) (Ft/Sec)			Poisson's Ratio (Calculated)			Young's Modulus PSI x 10^5 (Calculated)			Shear Modulus PSI x 10^4 (Calculated)			Simple Size (N)
				M	SD	SEM		M	SD	SEM	M	SD	SEM	M	SD	SEM	M	SD	SEM	M	SD	SEM	
SS-49	716.9	39.3	0	37.4	1.5	.7	37.9	2843	380	190	1169	52	26	.39	.04	.02	1.01	.09	.05	3.63	.32	.16	4
SS-53	727.0	43.4	0	41.7	-	-	41.7	3603	332	166	1257	99	50	.43	.01	.00	1.21	.19	.09	4.2	.66	.33	4
SS-55	727.0	60.1	0	45.2	-	-	45.2	3623	200	100	1172	74	37	.44	.01	.01	3.02	3.81	1.90	3.65	.45	.22	4
SS-56	727.1	48.7	0	37.3	-	-	37.3	3687	130	58	933	472	211	.45	.01	.00	.96	.17	.08	3.30	.62	.28	5
SS-59	699.7	28.6	0	25.0	.6	.3	26.8	4045	630	315	1395	192	96	43	.02	.01	1.5	.42	.21	5.24	1.51	.76	4
SS-60	726.1	45.9	0	36.1	-	-	36.1	2763	121	70	1292	85	49	.36	.01	.01	1.21	.15	.09	4.44	.59	.34	4
SS-62	697.7	23.2	0	22.0	.4	.2	22.0	2364	210	121	962	260	150	.40	.05	.03	.71	.35	.20	2.58	1.34	.77	3
SS-63	727.1	37.9	0	35.4	.5	.23	36.4	3339	140	70	1357	105	53	.40	.02	.01	1.37	.20	.10	4.91	.77	.38	4
SS-65	726.0	50.5	0	48.0	-	-	48.0	3195	267	133	1349	45	23	.39	.02	.01	1.35	.098	.049	4.83	.33	.16	4
SS-67	728.6	44.5	0	43.7	-	-	43.7	2469	160	113	1245	145	102	.33	.02	.02	1.10	.23	.16	4.15	.96	.68	2
SS-69	734.7	66.1	0	44.0	-	-	44.8	2998	282	141	1408	232	116	.36	.03	.02	1.45	.43	.02	5.37	1.68	8.41	4
SS-71	740.2	59.4	0	57.5	-	-	57.5	2242	28	16	1165	16	9	.32	0.0	0.0	.95	.02	.02	3.60	0.10	0.06	3
SS-73	737.2	37.8	0	36.2	-	-	36.2	1919	6	3	980	15	8	.32	.01	.00	.68	.02	.01	2.55	.07	.04	3
SS-75	722.7	41.5	0	30.4	2.4	1.2	30.0	3566	307	154	1502	34	42	.39	.04	.02	1.66	.15	.07	6.0	.67	.34	4
SS-84	733.4	35.6	0	34.3	-	-	34.3	4750	321	160	1756	77	39	.42	.01	.00	2.33	0.21	0.10	8.19	.71	.35	4
SS-86	727.5	38.5	30	34.9	-	-	34.9	3677	288	132	1434	67	39	.41	.01	.01	1.54	.15	.08	5.46	.51	.30	3
SS-88	720.2	42.1	47	36.8	5.1	2.5	36.5	3025	495	247	1100	95	48	.42	.02	.01	.92	.02	.01	3.23	.55	.27	4
SS-92	728.9	45.3	26	44.6	3.3	1.9	44.1	2336	116	67	963	5	3	.40	.01	.01	.69	.01	.01	2.46	.02	.01	3
SS-96	718.8	31.8	0	31.1	-	-	31.1	4247	227	114	1830	70	35	.38	.02	.01	2.46	.17	.08	8.89	.67	.34	4
SS-99	717.6	29.8	0	29.0	-	-	29.0	1899	83	42	700	13	7	.42	.01	0.0	.37	.01	.01	1.30	.48	.02	4
SS-101	724.5	24.8	0	23.5	-	-	23.5	4000	748	432	1637	157	90	.39	.03	.02	2.0	.40	.23	7.15	1.33	.77	3
SS-103	733.1	44.0	0	42.7	-	-	42.7	3375	245	123	1498	24	12	.38	.02	.01	1.64	.06	.03	5.96	.19	.10	4
SS-105	699.4	10.0	0	8.0	-	-	8.0	2050	352	176	919	75	38	.36	.05	.02	.61	.01	.01	2.25	.37	.18	4
SS-107	694.5	26.0	0	26.4	1.5	.9	25.8	3498	552	319	1498	170	98	.39	.02	.01	1.66	.40	.23	6.0	1.39	.08	3
SS-108	697.0	16.8	0	15.8	-	-	15.8	2887	252	145	1126	36	21	.41	.01	.01	.95	.01	.00	3.36	.02	.01	3

Table 2.5-17 In-Situ Soil Dynamic Properties Watts Bar Nuclear Power Plant Class IE Conduits and ERCW Piping (Sheet 2 of 2)

Notes:

1. Locations of soil dynamic test holes are shown on Figure 2.5-185,
2. Geometry of dynamic measurement configuration is shown in Figure 2.5-151.
3. Hydrophone depth (Z) is the difference in elevation between hydrophone center and shot point.
4. Best estimate of density is 123 lbs/ft^3.
5. M = arithmetic mean, SD = standard deviation, and SEM = standard error of the mean.
6. In-situ soil dynamic measurements were performed and interpreted by TVA Geologic Services Branch.

Table 2.5-17A Dynamic Soil Properties - Diesel Generator Building

Elevation	Material	Total Weight	Poisson's Ratio	Low Strain Shear Modulus	Shear Modulus vs. Strain	Damping Ratio vs. Strain	Remarks
741 to 730	Class A Backfill	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$psf	Figure 2.5-233A	Figure 2.5-233B	Vary G_{max} $\pm$50%
732 to 727	Crushed Stone (above groundwater)	133pcf	0.40	$G = 1000\ k_2\text{-}\sigma_m{}^{1/2}$ (k_2) max = 100 ko = 1-sin φ =0.4	Figure 2.5-233C	Figure 2.5-233D	Vary (k_2) max $\pm$50%
727 to 713	Crushed Stone (below groundwater)	142 pcf	0.40	$G = 1000\ k_2\text{-}\sigma_m{}^{1/2}$ (k_2) max = 100 ko = 1-sin φ =0.4	Figure 2.5-233C	Figure 2.5-233D	Vary (k_2) max $\pm$50%
730 to 718	In-Situ Cohesive Soils	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Figure 2.5-233E	Figure 2.5-233F	Vary G_{max} $\pm$ 50%
718 to 713	In-Situ Non-Plastic Soils*	120 pcf	0.38	$G = 1000\ k_2\text{-}\sigma_m{}^{1/2}$ (k_2) max = 38 ko = 0.5	Figure 2.5-233G	Figure 2.5-233H	Vary (k_2) max $\pm$ 50%
713 to 708	Basal Gravel	143 pcf	0.46	$G = 1000\ k_2\text{-}\sigma_m{}^{1/2}$ (k_2) max = 365 ko = 1-sin φ = 0.4	Figure 2.5-233I	Figure 2.5-233J	Vary (k_2) max $\pm$ 50%
708 to 693	Weathered Shale	127 pcf	0.35	$G_{max} = 12.8 \times 10^6$ psf (Vs. = 1800 fps)	Figure 2.5-233K	Figure 2.5-233K	Vary G_{max} $\pm$ 50%

* Includes CL-ML material with N values less than 10 blows per foot. For discussion of liquefaction potential for Watts Bar soils, see FSAR Section 2.5.4.8. Note: Design groundwater level = Elev. 727.

Table 2.5-17B Dynamic Soil Properties - Additional Diesel Generator Building

Elevation	Material	Total Weight	Poisson's Ratio	Low Strain Shear Modulus	Shear Modulus vs. Strain	Damping Ratio vs. Strain	Remarks
741 to 730	Class A Backfill	120 pcf	0.35	G_{max} = 4.5 x 10^6 psf	Figure 2.5-233A	Figure 2.5-233B	Vary G_{max} ± 50%
730 to 718	In-Situ Cohesive Soils	120 pcf	0.35	G_{max} = 4.5 x 10^6 psf	Figure 2.5-233E	Figure 2.5-233F	Vary G_{max} ± 50%
718 to 712	In-Situ Non-Plastic Soils*	120 pcf	0.38	G = 1000 k_2 -$\sigma_m{}^{1/2}$	Figure 2.5-233G	Figure 2.5-233H	Vary (k_2)max ± 50%
				(k_2) max = 38			
				ko = 0.5			
712 to 709	Basal Gravel	143 pcf	0.46	G = 1000 k_2 -$\sigma_m{}^{1/2}$	Figure 2.5-233I	Figure 2.5-233J	Vary (k_2)max ± 50%
				(k_2) max = 365			
				ko = 1-sin φ = 0.4			
709 to 693	Weathered Shale	127 pcf	0.35	G_{max} = 12.8 x 10^6 psf	Figure 2.5-233K	Figure 2.5-233K	Vary G_{max} ± 50%
				(Vs. = 1800 fps			

* Includes CL-ML material with N values less than 10 blows per foot. For discussion of liquefaction potential for Watts Bar soils, see FSAR Section 2.5.4.8. Note: Deisgn groundwater level = Elev 727.

Table 2.5-17C Dynamic Soil Properties - Refueling Water Storage Tanks

Elevation	Material	Total Weight	Poisson's Ratio	Low Strain Shear Modulus	Shear Modulus vs. Strain	Damping Ratio vs. Strain	Remarks
732 to 713	Class A Backfill	120 pcf	0.35	G_{max} = 4.5 x 10^6 psf	Figure 2.5-233A	Figure 2.5-233B	Vary G_{max} $\pm$ 50%
728 to 713	Crushed Stone	142 pscf	0.40	G = 1000 k_2 -$\sigma_m^{1/2}$	Figure 2.5-233C	Figure 2.5-233D	Vary (k_2) max $\pm$50%
				(k_2) max = 100			
				ko = 1 -sin φ = 0.4			
728 to 719	In-Situ Cohesive Soils	120 pcf	0.35	G_{max} = 4.5 x 10^6 psf	Figure 2.5-233E	Figure 2.5-233F	Vary G_{max} $\pm$ 50%
719 to 713	In-Situ Non-Plastic Soils*	120 pcf	0.38	G = 1000 k_2 -$\sigma_m^{1/2}$	Figure 2.5-233G	Figure 2.5-233H	Vary (k_2) max $\pm$ 50%
				(k_2) max = 38			
				ko = 1-sin φ = 0.5			
713 to 706	Basal Gravel	143 pcf	0.46	G = 1000 k_2 -$\sigma_m^{1/2}$	Figure 2.5-233I	Figure 2.5-233J	Vary (k_2) max $\pm$ 50%
				(k_2) max = 365			
				ko = 1-sin φ = 0.4			
706 to 693	Weathered Shale	127 pcf	0.35	G_{max} = 12.8 x 10^6 psf	Figure 2.5-233K	Figure 2.5-233K	Vary G_{max} $\pm$ 50%
				(Vs. = 1800 fps)			

* Includes CL - ML material with N values less than 10 blows per foot. For discussion of liquefaction potential for Watts Bar soils, see FSAR Section 2.5.4.8. Note: Design groundwater level - Elev. 727.

Table 2.5-17D Dynamic Soil Properties - North Steam Valve Room

Elevation	Material	Total Weight	Poisson's Ratio	Low Strain Shear Modulus	Shear Modulus vs. Strain	Damping Ratio vs. Strain	Remarks
728 to 683	Class A Backfill	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Figure 2.5-233A	Figure 2.5-233B	Vary $G_{max} \pm 50\%$
728 to 716	In-Situ Cohesive Soils	120 pcf	0.35	$G_{max} = 4.5 \times 10^6$ psf	Figure 2.5-233E	Figure 2.5-233F	Vary $G_{max} \pm 50\%$
716 to 698	Basal Gravel	143 pcf	0.46	$G = 1000\ k_2 \text{-} \sigma_m{}^{1/2}$	Figure 2.5-233I	Figure 2.5-233J	Vary (k_2) max $\pm$ 50%
				(k_2) max = 365			
				ko = 1-sin φ = 0.4			
698 to 683	Weathered Shale	127 pcf	0.35	$G_{max} = 12.8 \times 10^6$ psf	Figure 2.5-233K	Figure 2.5-233K	Vary $G_{max} \pm 50\%$
				(Vs. = 1800 fps)			

Note: Design groundwater level - Elev. 727

Table 2.5-18 Watts Bar Nuclear Plant
Borrow Investigation
Summary of Laboratory Test Data

					Grain-Size Analysis													Atterberg Limits				
	Percent of	Natural Moisture %			Gravel %			Sand %			Silt %			Clay %			Liquid Limit %			Plasticity Index %		
Symbol	Total	Min.	Max.	Avg.	Min.	Max.	Avg.	Min.	Max.	Avg.	Min.	Max.	Avg.	Min.	Max.	Avg.	Min.	Max.	Avg.	Min.	Max.	Avg.
CL	24.1	17.2	40.5	24.1	0	0	0	26	36	32	29	32	30	33	45	37	35.1	46.8	40.2	13.9	26.8	19.2
MH	34.0	19.0	33.9	24.9	0	0	0	22	26	24	27	30	29	47	48	48	50.1	51.7	50.9	17.9	18.5	18.2
SM-SC	9.9	5.6	35.1	22.2	0	0	0	50	60	55	19	26	23	21	24	23	24.3	24.4	24.4	5.2	5.5	5.4
ML	28.4	15.5	29.8	22.6	0	4	2	34	41	38	20	28	24	35	38	36	39.4	41.8	40.6	13.2	15.7	14.2
SM	2.1	37.6	53.8	45.5	-	-	0	--	--	52	--	--	23	--	--	25	--	--	30.1	--	--	6.9
SC	1.4	19.3	26.9	46.2	-	-	0	--	--	54	--	--	24	--	--	22	--	--	28.8	--	--	10.8

Table 2.5-19 Watts Bar Nuclear Plant Additional Borrow Areas Summary of Laboratory Test Data

Symbol	Percent of Total	Natural Moisture % Min.	Natural Moisture % Max.	Natural Moisture % Avg.	Grain-Size Analysis: Gravel % Min.	Gravel % Max.	Gravel % Avg.	Sand % Min.	Sand % Max.	Sand % Avg.	Silt % Min.	Silt % Max.	Silt % Avg.	Clay % Min.	Clay % Max.	Clay % Avg.	Atterberg Limits: Liquid Limit % Min.	Liquid Limit % Max.	Liquid Limit % Avg.	Plasticity Index % Min.	Plasticity Index % Max.	Plasticity Index % Avg.
<u>Borrow Area No. 1</u>																						
CL	56	21.6	30.4	25.7	0	0	0	9	29	19.0	34	45	39.0	37	46	42.0	41.6	49.8	45.9	17.7	22.6	20.0
MH	28	22.8	28.3	25.6	0	0	0	9	13	11.0	45	47	46.0	42	44	41.0	51.6	57.5	54.5	21.6	24.0	22.5
ML	16	24.2	31.3	28.4	0	0	0	--	--	11.0	--	--	46.0	--	--	43.0	--	--	49.7	--	--	17.6
<u>Borrow Area No. 2</u>																						
CL	74	12.0	27.2	21.4	0	0	0	13	24	20.0	39	54	44.0	31	40	36.0	37.5	46.6	43.5	14.9	22.0	18.9
MH	21	22.4	36.0	27.0	0	0	0	7	12	10.0	39	40	40.0	49	53	51.0	56.1	62.1	59.1	23.7	29.1	26.4
CH	5	19.1	20.1	19.6	-	-	0	--	--	11.0	--	--	42.0	--	--	47.0	--	--	55.3	--	--	30.6
<u>Borrow Area No. 3</u>																						
CL	50	20.9	24.2	22.3	0	0	0	18	45	32.0	28	38	33.0	27	44	36.0	36.0	43.6	39.8	14.2	22.5	18.4
ML	25	24.3	24.5	24.4	0	0	0	--	--	30.0	--	--	26.0	--	--	44.0	--	--	47.3	--	--	16.7
GC	25	--	--	10.5	-	-	43	--	--	33.0	--	--	10.0	--	--	14.0	--	--	33.0	--	--	12.7
<u>Borrow Area No. 4</u>																						
CL	57	21.2	24.3	35.5	0	0	0	25	38	31.5	34	41	37.5	28	34	31.0	31.8	38.1	35.0	9.1	15.9	12.5
ML	7	22.6	23.3	24.0	-	-	0	--	--	33.0	--	--	30.0	--	--	37.0	--	--	40.7	--	--	14.1
SM	36	18.7	40.1	22.8	-	-	0	--	--	51.0	--	--	28.0	--	--	21.0	--	--	24.0	--	--	2.4

Table 2.5-19a Soil Properties, Borrow Area 7

Soil	% of	Natural Moisture Content			Gravel			Sand			Silt			Clay			Liquid Limit			Plasticity Index		
Symbol	Total	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg	Max	Min	Avg
		%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%	%
CL	51.2	26.3	20.8	24.0	0	0	0	38	7	21	53	37	44	50	23	36	49	30	39	25	9	16
CL-ML	29.3	24.9	20.4	22.8	0	0	0	39	11	24	46	34	40	49	25	35	46	27	38	18	5	13
ML	12.2	27.7	21.1	23.5	0	0	0	38	13	30	38	31	35	49	26	34	43	32	38	14	4	10
CH-MH	4.9	33.7	26.5	30.1	0	0	0	8	5	7	44	44	44	51	48	50	54	52	53	25	23	24
MH	2.4	26.9	26.9	26.9	0	0	0	6	6	6	43	43	43	51	51	51	53	53	53	23	23	23

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Table 2.5-20 Grout Usage

Hole Number	Total Gallons of Grout	Hole Number	Total Gallons of Grout
1	4	15	4
2	4	16	13
3	4	17	13
4	8	18	13
5	9	19	6
6	4	20	10
7	4	21	11
8	4	22	10
9	4	23	9
10	7	24	9
11	14	25	10
12	15	26	10
13	13	27	10
14	4	28	9

**Table 2.5-21 Watts Bar Nuclear Plant Intake Channel
Summary of Laboratory Test Data Remolded Channel Area Soils**

Symbol	CL	CL	ML	SM	SM
Mechanical and hydrometer analysis					
Gravel, %	0	0	0	0	0
Sand, %	39	15	47	61	80
Silt, %	32	44	32	23	9
Clay, %	29	41	21	16	11
Atterberg limits					
Liquid limit, %	30.8	41.2	28.0	20.9	NP
Plastic limit, %	17.5	21.6	22.3	19.5	NP
Plasticity index, %	13.3	19.6	5.7	1.4	NP
Standard proctor compaction					
Average natural moisture, %	25.5	25.8	23.6	25.7	25.5
Optimum moisture, %	17.0	20.7	17.5	16.0	18.4
Maximum density, pcf	109.9	102.8	108.9	111.8	107.2
Triaxial Q shear strength at 95% of maximum density with 4% moisture					
$\underline{\varphi}$, degrees	4.8	8.5	15.0	22.0	31.3
c, tsf	0.95	0.90	0.80	0.65	0.55

Table 2.5-22 TVA Soil Testing Laboratory Summary of Test Results Watts Bar Liquefaction Study

Test No.	KC	γd	B	σ1c	σ3c	σac	R	N_L	N_{5*}	N_{10*}	N_{15*}
1	1.0	88.3	0.97	10.2	10.2	3.67	0.180	8	--	--	--
2	1.0	88.2	0.99	10.2	10.2	3.57	0.175	12	--	--	--
3	1.0	89.4	0.99	10.2	10.2	2.15	0.105	428	--	--	--
4	1.5	87.8	0.97	10.0	6.7	2.80	--*	--	6	7	7
5	1.5	87.8	0.97	10.0	6.7	2.00	--	--	46	50	53
6	1.5	87.5	0.98	10.0	6.7	1.07	--	--	500	606	690
7	2.0	88.3	0.98	10.0	5.0	2.40	--	--	2	3	4
8	2.0	88.3	0.98	10.0	5.0	2.05	--	--	9	10	10
9	2.0	88.3	0.98	10.0	5.0	1.50	--	--	34	36	38

Denotation: Kc=σ1c/σ3c

γd=dry density of specimen in pcf.

B=coefficient of pore pressure.

σ1c or σ3c=the consolidation pressure in the axial or lateral direction respectively in psi.

σac=the axial cyclic stress in psi

R=σac/2 σ3c when σ1c = σ3c (Kc = 1).

N_L=the number of cycles required to cause liquefaction.

N_{5}, $_{10}$, or $_{15}$= the number of cycles required to cause axial strain of 5, 10, or 15 percent.

*For anisotropic loading, strain is for compression only.

**Liquefaction did not occur under anisotropic loading

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Table 2.5-23 Waterways Experiment Station, Corps of Engineers Summary of Test Results Watts Bar Liquefaction Study Isotropic Loading

Test No.	Density σ_{dry} pcf	Back Pressure psi	B	σ_a or σ_{dc} psi	σ_{dc} psi	$\frac{\sigma_{dc}}{2\sigma_a}$	Accel g	Cycles to initial Liquefaction	% Strain @ initial Liquefaction	Cycles to 10% Strain*	Cycles to20% Strain*	K_c
						Isotropic Loading						
1	94.7	65.0	0.970	10.2	4.04	0.198	0.136	16	2.6	17(12.2)	19(22.1)	1.0
2	88.5	60.0	0.980	10.2	3.64	0.179	0.123	5	0.6	6(12.8)	7(22.1)	1.0
3	88.4	60.0	0.980	10.2	2.57	0.126	0.086	51	0.9	52(18.6)	53(22.2)	1.0
4	85.7	60.0	0.980	10.2	3.34	0.156	0.107	5	0.4	6(22.3)	7(22.3)	1.0
4A	88.7	60.0	0.980	10.2	3.82	0.187	0.128	4	0.4	5(17.2)	6(22.2)	1.0
						Anisotropic Loading						
5	86.4	65.0	0.970	6.8	2.11	-	-	-**	-	1(21.2)	1(21.2)	1.5
6	89.2	64.0	0.980	6.8	1.01	-	-	-	-	23(9.7)	32(20.7)	1.5
7	88.1	64.0	0.970	6.8	0.70	-	-	-	-	170(16.4)	171(21.6)	1.5
8	87.8	65.0	0.980	5.1	1.94	-	-	-	-	1(20.5)	1(20.5)	2.0
9	90.1	65.0	0.980	5.1	0.94	-	-	-	-	14(13.6)	19(20.7)	2.0
10	88.9	65.0	0.970	5.1	0.57	-	-	-	-	32(10.3)	34(20.6)	2.0

Note: Terms defined in text.

* Number of cycles shown is the cycle closest to 10% and 20% strain for the two columns. The number in parenthesis is the actual strain. For isotropic loading, the strain is for both compression and extension. For anisotropic loading, the strain is only for compression.

** Initial liquefaction did not develop in anisotropically consolidated specimens.

σ_{dry} – Dry unit weight of soil.
B – Ratio of the change in pore water pressure to an induced change in chamber pressure.
σ_a – Ambient consolidation stress.
σ_{3c} – Effective confining stress at the end of consolidation.
σ_{dc} – Cyclic deviator stress.
σ_{lc} – Effective axial stress.
K_c – σ_{1c}/σ_{3c}.
g – Acceleration.

Table 2.5-24 Watts Bar Nuclear Plant
ERCW and HPFP Systems
Soil Investigation
Summary of Laboratory Test Data
(Sheet 1 of 3)

Elevation	Soil Symbol	Soil Type	Nat. Moist. %	Nat. Moist. % Sat.	Std. Penetr.	Grain-size Analysis: Gravel %	Grain-size Analysis: Sand %	Grain-size Analysis: Silt %	Grain-size Analysis: Clay %	D_{10} mm	Atterb. Limits: Liq. Limit %	Atterb. Limits: Plastic. Index %	Dry Dens. pcf	Void Ratio	Unconfined Compression: Undis. tsf	Unconfined Compression: Rem. tsf	Sens. Ratio	Triaxial Q Undisturbed: φ deg	Triaxial Q Undisturbed: c tsf	Natural Moisture Triaxial R: φ deg	Natural Moisture Triaxial R: c tsf	Direct Shear S: φ deg	Direct Shear S: c tsf
Boring US- 75 Surface El. 722.7																							
721.7-720.4	CL	b	18.7	86.8	26	5	35	28	32	--	40.2	16.9	106.7	0.580									
718.7-716.4	SM	I	19.6	78.3	20	1	69	16	14	--	32.3	7.5	100.3	0.674				29.5	0.26	33.5	0.30		
Boring US- 77 Surface El. 731.1																							
730.1-727.7	CH	A	20.2	64.1	9	0	19	33	48	--	58.9	26.9	92.5	0.854	3.8	3.5	1.1	18.4	1.70	12.0	2.90	30.0	1.25
727.1-725.7	ML	D	22.2	82.6	18	0	43	34	23	--	36.4	8.6	98.4	0.734						25.5	1.10	35.5	0.38
724.1-721.7	CL	b	31.1	95.3	10	0	20	52	28	--	44.7	17.4	89.9	0.888									
721.1-718.7	CH	A	31.9	97.2	10	0	13	52	35	--	52.1	20.6	89.7	0.893	1.8	1.8	1.0	8.0	0.90	24.5	0.25	26.5	0.62
718.1-715.7	CL	B	31.6	95.4	4	0	10	53	37	--	49.0	24.5	85.5	0.901	1.3	1.1	1.2	6.5	0.40	21.5	0.71	30.0	0.14
715.1-712.7	SM	J	28.9	97.0	3	0	67	20	13	.0032	NP	NP	92.2	0.783				3.0	0.60	26.5	0.50	32.5	0.07
712.1-711.5	SP-SM	j	23.6	87.0	6	0	90	7	3	.075	NP	NP	97.4	0.731									
Boring US-92 Surface El. 728.9																							
727.9-725.5	ML	D	24.3	82.5	8	0	33	26	41	--	46.0	13.8	95.4	0.819	3.9	3.2	1.2	22.5	1.14	24.5	0.70	18.0	1.08
724.9-722.5	SM	I	19.6	80.1	14	0	57	28	15	--	31.7	7.9	96.5	0.684				19.5	0.73	25.2	0.90		
721.9-719.5	SM	I	18.0	75.7	5	0	59	20	21	--	28.1	4.3	103.1	0.644				30.5	0.85	21.5	1.30	31.0	0.25
718.9-716.5	G-SC	i	14.0	68.5	5	20	50	17	13	--	29.0	7.8	109.1	0.556									
715.9-714.9	SM	J	16.0	58.3	10	5	74	15	6	.013	NP	NP	96.6	0.740						31.2	0.45		
712.9-712.4	G-SM	j	16.7	62.4	43	13	66	13	8	.012	NP	NP	97.3	0.713									

Table 2.5-24 Watts Bar Nuclear Plant
ERCW and HPFP Systems
Soil Investigation
Summary of Laboratory Test Data
(Sheet 2 of 3)

											Atterb. Limits				Unconfined Compression			Triaxial Q		Natural Moisture		Direct Shear	
			Nat.		Std.		Grain-size Analysis				Liq.	Plastic.	Dry	Void			Sens.	Undisturbed		Triaxial R		S	
	Soil	Soil	Moist.		Penetr.	Gravel	Sand	Silt	Clay	D_{10}	Limit	Index	Dens.	Ratio	Undis.	Rem.	Ratio	φ	c	φ	c	φ	c
Elevation	Symbol	Type	%	% Sat.		%	%	%	%	mm	%	%	pcf		tsf	tsf		deg	tsf	deg	tsf	deg	tsf
Boring US-94 Surface El. 697.9																							
696.9-695.9	SM	j	24.6	100	24	0	54	37	9	.0075	NP	NP	104.8	0.585									
695.9-694.8	GC	f	20.3	75.3	42	46	29	16	9	.0065	39.8	16.6	98.4	0.738									
693.9-691.8	GC	f	12.3	47.7	36	41	40	14	5	.018	35.7	13.5	101.7	0.720									
690.9-689.8	GC	f	13.6	79.3	50+	46	38	12	4	.019	35.2	12.5	116.3	0.470									
Boring US- 97 Surface El. 717.9																							
716.9-714.5	CL	B	20.8	89.1	4	0	25	44	31	--	35.6	17.7	102.8	0.626	1.8	1.8	1.0	22.5	0.44	21.5	1.03	33.0	0.12
713.9-711.5	CL	b	16.3	91.5	15	0	43	34	23	--	37.4	20.6	113.1	0.474									
710.9-709.7	SC	i	19.5	90.0	13	6	45	25	24	--	31.3	11.1	106.5	0.583								25.5	0.59
Boring US- 103 Surface El. 733.1																							
724.1-721.7	MH	H	24.4	90.0	19	10	33	18	39	--	54.3	20.6	96.7	0.735	3.5	5.0	0.7	26.5	1.27	18.5	1.67	33.0	0.45
721.1-719.1	MH	H	31.1	85.9	12	0	34	22	44	--	50.9	20.9	89.3	0.780	2.6	4.9	0.5	15.0	1.65	24.0	1.15	32.5	0.56
718.1-717.4	ML	D	26.1	91.6	16	0	38	31	31	--	47.8	16.7	96.2	0.779								32.0	0.21
Boring US-106 Surface El. 714.7																							
713.7-711.9	CL	B	12.7	64.6	8	0	43	34	23	--	28.4	11.3	108.7	0.523	1.4	1.1	1.3	18.0	1.25	25.0	0.85	34.6	0.20
707.7-707.3	GC	f	15.1	--	40	46	32	11	11	.004	226.6	10.2	--		--								
707.3-707.0	GM	f	34.3	--	40	62	13	17	8	.008	40.7	14.6	--	0.608	--								
704.7-704.0	CL	b	15.1	67.5	32	8	33	31	28	--	44.1	21.6	105.2										
704.0-703.4	GM	f	19.8	74.5	32	45	27	22	6	.012	39.8	8.6	99.7	0.741									

Table 2.5-24 Watts Bar Nuclear Plant
ERCW and HPFP Systems
Soil Investigation
Summary of Laboratory Test Data
(Sheet 3 of 3)

Elevation	Soil Symbol	Soil Type	Nat. Moist. %	Nat. Moist. % Sat.	Std. Penetr.	Grain-size Analysis Gravel %	Grain-size Analysis Sand %	Grain-size Analysis Silt %	Grain-size Analysis Clay %	$\underline{D}_{10}$ mm	Atterb. Limits Liq. Limit %	Atterb. Limits Plastic. Index %	Dry Dens. pcf	Void Ratio	Unconfined Compression Undis. tsf	Unconfined Compression Rem. tsf	Sens. Ratio	Triaxial Q Undisturbed φ deg	Triaxial Q Undisturbed c tsf	Natural Moisture Triaxial R φ deg	Natural Moisture Triaxial R c tsf	Direct Shear S φ deg	Direct Shear S c tsf
Boring US-107 Surface El. 694.5																							
693.5-691.6	CL	C	24.0	82.9	2	0	49	33	18	--	24.8	7.6	94.9	0.789									
690.5-688.1	G-OH	A	53.2	93.2	1	16	23	28	33	--	55.6	30.1	66.3	1.466	0.3	0.3	1.0	2.5	0.12	11.0	0.22		
687.5-686.4	G-OL	b	24.9	99.1	15	20	25	31	24	--	32.2	16.9	99.3	0.666									
684.5-683.5	OL	D	78.1	87.9	15	0	40	49	11	.005	38.7	4.7	53.5	2.348	0.5	0.2	2.5	2.0	0.18				
678.5-677.7	ML	D	32.9	87.9	12	0	29	52	19	--	39.6	12.5	85.0	0.989					23.0	0.65			
675.5-674.7	G-ML	d	21.7	98.3	20	29	43	17	11	0042	37.9	10.9	105.9	0.597									
Boring US-108 Surface El. 697.0																							
696.0-693.7	CL	C	24.4	88.5	2	0	28	46	26 --	26.4	7.9	95.6	0.731	0.6	0.2	3.0	8.5	0.35	30.5	0.08	31.5	0.20	
693.0-691.0	GM	F	16.9	81.2	20	62	20	11	7	.011	28.9	13.3	110.2	0.581				17.0	0.10				
689.0-688.7	GM	f	21.4	--	21	56	22	17	5	.019	NP	NP	--	--	--								

Table 2.5-25 Watts Bar Nuclear Plant Summary Of Laboratory Test Data Borrow Soil Classes

Class	If	IId	IIIb
Symbol	SM	CL	ML
Mechanical and Hydrometer Analysis			
Gravel, percent	0	0	0
Sand, percent	51	38	33
Silt, percent	28	34	30
Clay, percent	21	28	37
Atterberg Limits			
Liquid limit, percent	24.0	31.8	40.7
Plastic limit, percent	21.6	22.7	26.6
Plasticity index, percent	2.4	9.1	14.1
Shrinkage limit, percent	--	--	--
Standard Proctor Compaction			
Optimum moisture, percent	15.1	17.6	20.4
Maximum density, pcf	113.3	108.9	103.7
Penetration resistance, psi			
Shear Strength at 3% Above Optimum Moisture and at 95% of Maximum Density			
Triaxial Q: φ degrees	8.5	6.6	2.5
c tsf	1.18	0.92	1.40
Shear Strength at 3% Below Optimum Moisture and at 95% of maximum Density			
Triaxial R: φ degrees	32.0	29.0	24.0
c tsf	0.85	1.15	1.35
Direct Shear S: φ degrees	34.0	38.0	44.5
c tsf	0.58	0.40	0.18

Table 2.5-26 Watts Bar Nuclear Plant
Intake Channel
Sand Material
Summary of Cyclic Loading Test Data

Sample No.	Dry* Density r_d (pcf)	Pore Pressure Coefficient	Consolidation Pressure Axial $\sigma 1c$ (psi)	Consolidation Pressure Lateral $\sigma 3c$ (psi)	$\sigma 1c/\sigma 3c$	$\sigma ac/2\ \sigma 3c$	% Strain at 1000 Cycles	Number of Cycles to Strain at** Liquefaction	5%	10%	20%
Isotropic Loading											
6	92.1	0.95	17.5	17.5	1.0	0.45		9	10	12	20
6	91.1	0.99	17.5	17.5	1.0	0.26		192	196	200	206
6	90.7	1.00	17.5	17.5	1.0	0.11	0.2	Static R Test			
D-1	111.3	0.98	20.0	20.0	1.0	0.43		19	23	34	60
D-1	110.8	0.97	20.0	20.0	1.0	0.26		122	130	138	164
D-1	110.8	0.97	20.0	20.0	1.0	0.10	0	Static R Test			
Anisotropic Loading											
6	93.1	0.97	35.0	17.5	2.0	0.44			127	228	500
6	91.9	0.97	35.0	17.5	2.0	0.25	0.5	Static R Test			
6	92.9	0.98	35.0	17.5	2.0	0.11	0.2	Static R Test			
D-1	111.6	1.00	40.0	20.0	2.0	0.45			161	370	891
D-1	112.2	0.97	40.0	20.0	2.0	0.25	0	Static R Test			
D-1	112.6	0.98	40.0	20.0	2.0	0.11	0	Static R Test			

* After consolidation.

** Zero to peak compressive strain.

Table 2.5-27 Watts Bar Nuclear Plant
Intake Channel
Clay Material
Summary of Static Test Data

									Atterb. Limits				Saturated Triaxial R			
		Soil			Grain-Size Analysis				Liq.	Plastic	Dry	Void	Apparent		Effective	
Elevation	Location	Symbol	Nat. Moist.		Gravel	Sand	Silt	Clay	Limit	Index	Dens.	Ratio	φ	c	φ	c
			%	% Sat.	%	%	%	%	%	%	pcf		deg	tsf	deg	tsf
660.7-660.0	A-2	SC	26.0	85.5	0	52	26	22	26.3	9.2	92.1	.816	10.5	0.80	31.7	0.04
660.6-660.0	B-1	CL	27.9	95.5	0	23	42	35	34.4	17.3	93.8	.790	11.0	0.53	26.5	0.23
660.6-660.0	C-1	CL	33.9	96.3	0	15	35	50	48.8	28.1	85.9	.970	12.0	0.35	28.0	0.03
660.6-656.0	D-1	CL	37.9	97.4	0	07	46	47	42.3	23.4	82.8	1.021	13.0	0.05	29.5	0.00

Average R Strength For Analysis

$\varphi = 12°$

c = 900 psf

Table 2.5-28 Drill rod lengths and weights versus SPT sample depths Applying to 1976 and 1979 reports

Boring Depth	Drill Rod* (AW) Weight	Drill Rod* (AW) Length
(ft)	(ft)	(lbs)
0 - 5	5	21
5 - 10	10	42
10 - 15	15	63
15 - 20	20	84
20 - 25	25	105
25 - 30	30	126
30 - 35	35	147
35 - 40	40	168
40 - 45	45	189
45 - 50	50	210
50 - 55	55	231
55 - 60	60	252
60 - 65	65	273
65 - 70	70	294
70 - 75	75	315
Weight of safety hammer and drive stem	178.8 lbs	
Weight of safety hammer	140.0 lbs	
Weight of drive stem	38.8 lbs	
Weight of split barrel sampler	15.7 lbs	
Length of split barrel sampler	2.8 ft	

*rods in 5 ft increments

Table 2.5-29 Watts Bar Nuclear Plant
ERCW Conduit
1976 Report
(Sheet 1 of 2)

Boring No.	Drill No.	Drill Model	Boring Depth
SS-65	91930	Mobile B-50	50.5
SS-67	91930	Mobile B-50	44.5
SS-69	91930	Mobile B-50	66.1
SS-71	91930	Mobile B-50	59.4
SS-73	92251	Mobile B-50	37.8
SS-74	92251	Mobile B-50	34.2
SS-75	92251	Mobile B-50	41.5
US-75	92251	Mobile B-50	10.0
SS-76	92251	Mobile B-50	31.5
SS-77	92251	Mobile B-50	40.9
US-77	92251	Mobile B-50	22.0
SS-78	92251	Mobile B-50	25.5
SS-80	92251	Mobile B-50	61.7
SS-82	91930	Mobile B-50	37.5
SS-84	91930	Mobile B-50	35.6
SS-86	92251	Mobile B-50	38.5
SS-87	92251	Mobile B-50	43.4
SS-88	91930	Mobile B-50	42.1
SS-90	91930	Mobile B-50	58.8
SS-92	91930	Mobile B-50	45.3
US-92	92251	Mobile B-50	22.0
SS-93	91930	Mobile B-50	19.3
SS-94	92251	Mobile B-50	12.2
US-94	92251	Mobile B-50	8.2
SS-95	92251	Mobile B-50	21.3
SS-96	91930	Mobile B-50	31.8
SS-97	91930	Mobile B-50	45.2
SS-97A	91930	Mobile B-50	14.5
US-97A	92251	Mobile B-50	8.3
SS-99	91930	Mobile B-50	29.8

Table 2.5-29 Watts Bar Nuclear Plant
ERCW Conduit
1976 Report
(Sheet 2 of 2)

Boring No.	Drill No.	Drill Model	Boring Depth
SS-101	91930	Mobile B-50	24.8
SS-103	91930	Mobile B-50	44.0
US-103	92251	Mobile B-50	19.1
SS-104	91930	Mobile B-50	33.3
SS-105	92251	Mobile B-50	10.0
SS-106	92251	Mobile B-50	31.9
US-106	92251	Mobile B-50	10.5
SS-107	91930	Mobile B-50	26.0
US-107	92251	Mobile B-50	23.3
US-108	92251	Mobile B-50	8.3
SS-108	91930	Mobile B-50	16.8

Table 2.5-30 Watts Bar Nuclear Plant ERCW Conduit 1976 Report

Boring No.	Drill No.	Drill Model	Boring Depth
SS-131	92357	CME-55	36.0
SS-132	419991	CME-75	38.0
SS-133	419991	CME-75	39.5
SS-134	419991	CME-75	45.5
SS-135	419991	CME-75	45.0
SS-136	91930	Mobile B-50	44.0
SS-137	419991	CME-75	32.0
SS-138	419991	CME-75	42.5
SS-139	92357	CME-75	54.0
SS-140	419991	CME-75	38.5
SS-141	419991	CME-75	39.5
SS-142	419991	CME-75	46.5
SS-143	419991	CME-75	46.0
SS-144	419991	CME-75	45.5
SS-145	419991	CME-75	40.5
SS-146	91930	Mobile B-50	71.5
SS-147	91930	Mobile B-50	57.5
SS-148	419991	CME-75	38.0
SS-149	419991	CME-75	36.0
SS-150	419991	CME-75	21.0
SS-151	419991	CME-75	34.0
SS-152	91930	Mobile B-50	26.0
SS-153	91930	Mobile B-50	26.0
SS-154	419991	CME-75	31.5
SS-155	91930	Mobile B-50	21.0
SS-156	419991	CME-75	21.0
SS-157	91930	Mobile B-50	25.5
SS-158	419991	CME-75	28.0
SS-159	419991	CME-75	33.5
SS-160	91930	Mobile B-50	33.5
SS-161	419991	CME-75	37.0
SS-162	91930	Mobile B-50	31.5
SS-163	419991	CME-75	33.5
SS-164	91930	Mobile B-50	40.0
SS-165	419991	CME-75	41.0
SS-166	91930	Mobile B-50	37.0
SS-167	91930	Mobile B-50	34.5
SS-168	419991	CME-75	37.0
SS-169	419991	CME-75	39.0
SS-170	419991	CME-75	71.0

Table 2.5-31 Recommended Procedures and Guidelines for Standard Penetration Testing
(Sheet 1 of 2)

General

The procedures shall conform to ASTM D 1586 with the following modifications and additions.

Drilling

1. Rotary drilling methods and drilling mud shall be used. Casing shall not be used except as needed in the upper few feet of the boring to provide good circulation of the drilling mud.
2. Drilling mud shall be sufficiently viscuous to lift the cuttings out of the boring and provide a clean hole at the time of sampling, to minimize caving and sloughing of the borehole walls, and to minimize water losses. As a guide- line, the marsh funnel viscosity of the drilling mud should be equal to or greater than 40.
3. The hole diameter shall be 4" to 5".
4. The drilling bits shall be fishtail bits equipped with deflectors to provide radial or upward discharge of the drilling fluid. The use of bits that discharge drilling fluid directly down onto the soil at the bottom of the borehole is not permitted.
5. The hole shall be thoroughly cleaned of cuttings prior to sampling.
6. The depth of the borehole shall be measured after drilling and prior to insertion of the sampler into the borehole. (This can be accomplished from knowledge of the lengths of drill rods in the hole during drilling.)

Sampling

1. The required sampler dimensions are given in ASTM D 1586. Typically, however, these samplers are manufactured with a slightly larger inside diameter to provide a space for thin liners. It is preferred to use the typical sampler but without using the liners.
2. The level of drilling mud in the boring is required by ASTM D 1586 to be at or above the ground water level. However, in rotary drilling, it is desirable and practical to have the water level essentially at the ground surface during both drilling and sampling.
3. The depth of the drill hole shall be measured after inserting the sampler. This depth shall be compared with the depth measured after drilling to indicate any accumulation of cuttings in the borehole.
4. A rope-and-cathead system shall be used to lift and release the falling weight. Two turns of rope shall be provided around the cathead.
5. The sampler should driven for the full 18". A record of the blows for each 6" of drive should be maintained.
6. After recovering the sample, the length of recovery shall be measured, and the entire sample shall be examined and classified.
7. Samples shall be stored in glass jars sealed to preserve the natural water content of the soil. The pieces of samples shall be maintained as intact as possible (i.e., intact sample pieces should not be broken up and mixed together).

Table 2.5-31 Recommended Procedures and Guidelines for Standard Penetration Testing
(Sheet 2 of 2)

<u>Record Keeping</u> In addition to the usual boring log, a log shall be maintained for each sample. It is suggested that this log be on an 8-1/2" by 11" sheet of paper showing the entire sample length. Information to be shown thereon includes: 1. Total length of drive of the sampler (usually 18"). 2. Position of the recovered sample in the sampler. 3. Total recovery (in inches) and percent recovery. 4. The record of the blows for each 6" of drive. 5. The description and classification of the sample along its length (different segments may have different description and classifications if changes in soil type occur in the sample.) 6. Identification of the jars containing the pieces of the sample.

Table 2.5-32 Drill Rod Lengths and Weights Versus SPT
1981 Report

Boring Depth	Drill Rod* (AW) Length	Drill Rod* (AW) Weight
(ft)	(ft)	(lbs)
0 - 6.5	5	21
6.5 - 9.0	10	42
9.0 - 14.0	15	63
14.0 - 19.0	20	84
19.0 - 24.0	25	105
24.0 - 29.0	30	126
29.0 - 34.0	35	147
34.0 - 39.0	40	168
Weight of safety hammer and drive stem		178.8 lbs
Weight of safety hammer		140.0 lbs
Weight of drive stem		38.8 lbs
Weight of split barrel sampler		15.7 lbs
Length of split barrel sampler		2.8 ft

*rods in 5 ft increments

Table 2.5-33 Watts Bar Nuclear Plant
ERCW Conduit
1981 Report

Boring No.	Drill No.	Drill Model	Boring Depth
SS-49A	93634	Mobile B-61	25.0
SS-50A	93634	Mobile B-61	26.8
SS-65B	93634	Mobile B-61	26.5
SS-134A	93634	Mobile B-61	26.0
SS-135A	93634	Mobile B-61	26.5
SS-138A	93634	Mobile B-61	26.0
SS-138B	93634	Mobile B-61	24.5
SS-138C	93634	Mobile B-61	24.5
SS-143A	93634	Mobile B-61	30.5
SS-143B	93634	Mobile B-61	29.5
SS-143C	93634	Mobile B-61	30.0
SS-158A	93634	Mobile B-61	21.5
SS-161A	93634	Mobile B-61	26.5
SS-163A	93634	Mobile B-61	30.5

Table 2.5-34 Watts Bar Nuclear Plant
Essential Raw Cooling Water Piping System
Liquefaction Investigation
Summary Of Laboratory Test Data

				Atterb. Limits								
	Soil	Nat.				Grain-Size Analysis			Liq.	Plastic	Dry	Void
Elevation	Symbol	Moist.		Gravel	Sand	Silt	Clay	D10	Limit	Index	Dens.	Ratio
		%	% Sat.	%	%	%	%	mm	%	%	pcf	
Boring No. US-50-1, 1650.0S 785.0E, Surface Elevation 716.9												
701.4-700.7	SM	27.6	89.6	0	59	25	16	-----	31.6	6.1	92.4	.841
698.9-696.6	SM	33.0	87.5	0	82	14	4	.0504	NP	NP	84.0	1.002
696.4-695.3	SM	28.9	93.6	0	88	9	3	.0567	NP	NP	93.1	.828
695.3-694.5	SM	31.1	95.9	0	53	34	13	.0029	23.1	1.0	90.4	.863
694.2-692-1	SM	30.5	99.5	0	80	15	5	.0245	NP	NP	93.5	.829
Boring No. US-50-1A, 1645.0S 785.0E, Surface Elevation 717.0												
703.9-702.5	SM	25.9	87.6	0	64	24	12	.0035	NP	NP	95.0	.796
701.6-699.4	SM	37.8	88.7	0	67	22	11	.0033	NP	NP	79.2	1.148
Boring No. US-65-1, 1367.0S 1005.7E, Surface Elevation 726.9												
711.9-709.6	SM	22.2	65.1	0	70	22	8	.0078	NP	NP	88.6	.904
709.4-707.3	SM	22.7	68.8	2	60	25	13	-----	NP	NP	90.3	.879
707.2-705.2	SM	33.4	94.2	0	65	24	11	.0039	NP	NP	87.0	.951
705.2-704.0	SM	31.6	100	3	49	32	16	-----	26.1	2.8	92.3	.835
703.8-703.2	ML	34.3	100	1	44	35	20	-----	28.2	5.0	90.8	.861
703.0-701.8	G-SM	18.8	----	32	48	14	6	.0205	NP	NP	----	----

Table 2.5-35 Laboratory Procedure For Performing Cyclic Triaxial R Tests

1. Test specimens were hand-trimmed from the undisturbed samples by a senior technician using a split-trimming tube 2.8‘ in diameter and 6.3' in height.
2. After removal from the trimming tube, the specimen was encased in a rubber membrane, the average thickness of which had been previously determined, and was then placed on the bottom platen of the triaxial testing machine.
3. The membrane was sealed at the top and bottom platens with O-rings. A small vacuum of about 5' of mercury was applied to the specimen.
4. Measurements of specimen diameter were made with pi tape at the center and at the quarter points. The specimen height was determined with a steel rule at 90N intervals around the specimen.
5. After zeroing the readout of axial load, deformation, pore water pressure, and cell pressure, the cyclic triaxial cell was assembled. Then the vacuum in the specimen was gradually reduced to zero while simultaneously increasing cell pressure to 3 lb/in^2.
6. The specimen was flushed continuously and slowly with 10' waterhead until no air bubbles were observed exiting from the specimen.
7. A back pressure was then applied to the specimen in an increment of 10 lb/in^2. The pressure differential between the cell and back pressure was maintained at 3 lb/in^2 throughout the saturation phase.
8. Step 6 was repeated at every level of the back pressure increment. At the final stage of back pressure saturation, Skempton's pore pressure parameter B was checked with drainage lines closed and at 6-lb/in^2 confining pressure. The parameter B was defined as:

   $$B = \frac{u}{3}$$

   where u = pore pressure increase
   $_3$ = an increase in confining pressure
9. After completion of saturation, the specimen was consolidated overnight at 2000-lb/ft^2 confining pressure.
10. Prior to the cyclic loading test, the B value was checked again. Step 6 was repeated if needed.
11. During consolidation, the change in height and the volume change of the specimen were measured. Thus, the area, volume, and dry density of the specimen after consolidation could be calculated.
12. In addition, specimen and pore water pressure system leaks were checked by closing the drainage lines and measuring pore water pressure response. The change of pore water pressure was less than 2% of the confining pressure over a 5-minute interval.
13. The specimen was cyclically loaded without drainage using a pneumatic system which applied a square wave with a degraded rise time at a frequency of 1 Hz. (See NRC publication NUREG-0031, page 96.)
14. During cyclic loading, changes in axial load and deformation, pore water pressure, and confining pressure were recorded on 8' photosensitive paper using a Honeywell Visicorder.
15. Cyclic loading was continued until a double-amplitude strain of 20% was attained.
16. After completion of cyclic loading, the test specimen was dried in a conventional oven for determination of moisture content.

Table 2.5-36 Results of Stress-Controlled Cyclic Triaxial Tests on ERCW Route Soils

Boring No. (Sample No.)	Dry Density Yd (lb/ft^2)	Confining Pressure σ3 (lb/ft^2)	Consolidation Ration $K_c = \frac{\sigma_1}{\sigma_3}$	Average Stress Ration $\sigma d/2\sigma_3$	Number of Cycles To: Initial Liquefaction*	Double Amplitude Strain: 5%	10%	15%	20%
US-50-1	90.2	1000	2.0	0.52	45	10	39		
(2)				0.51				74	106
US-50-1	93.9	1000	2.0	0.44		3			
(3)				0.43			8		
				0.42	10			13	
				0.41					20
US-50-1	93.9	1000	2.0	0.27		3			
(3)				0.28			12		
US-50-1	93.9	1000	2.0	0.26		28	104	247	
(3)				0.25					303
US-50-1 (3)	93.9	1000	2.0	0.17	0.2% Strain after 1000 cycles of cyclic stress, Δμ/σ3 = 0.27.				
US-50-1 (4)	97.0	1000	2.0	0.51	3% Strain after 1000 cycles of cyclic stress, Δμ/σ3 = 0.79.				

*Defined as $\Delta\mu = \sigma_3$

Table 2.5-37 Summary Of Classification Data

Pit No.	Sample No.	Gravel (%)	Sand (%)	Fines (%) Silt (%)	Fines (%) Clay (%)	Class	LL	PI	w (%)
1	1A-1	0	57	27	16	SM	NP	NP	24.7
(el. 706.6)	1A-2	0	67	21	12	SM	NP	NP	28.6
	1A-3	0	63	23	14	SM	NP	NP	28.5
	1A-4	0	64	24	12	SM	NP	NP	26.9

Split-Spoon Boring Sample	Elevation	Gravel (%)	Sand (%)	Fines (%)	Class	LL	PI	w (%)
SS-134	710.2	0	74	26	SM	NP	NP	29.3
	702.2	0	69	31	SM	NP	NP	27.5
SS-134A	710.2	0	65	35	SM	23.0	1.0	30.0
	709.6	0	69	31	SM	NP	NP	29.1
	707.7	0	63	37	SM	24.0	2.0	27.9
	707.2	0	57	43	SM	24.0	1.0	28.9
	706.4	0	68	32	SM	NP	NP	31.9
SS-135A	714.5	0	51	49	SM	31.0	3.0	24.3
	712.5	0	67	33	SM	NP	NP	22.8
	710.5	0	71	29	SM	NP	NP	24.3
	708.5	0	71	29	SM	NP	NP	34.2
	706.8	0	67	33	SM	22.0	1.0	27.0
	704.2	2	63	35	SM	NP	NP	30.9

Table 2.5-38 Summary of Classification Data

Pit No.	Sample No.	Gravel (%)	Sand (%)	Fines (%) Silt (%)	Clay (%)	Class	LL	PI	w (%)
2	2A-1	0	69	22	9	SM	NP	NP	26.7
Red to	2A-2	0	69	20	11	SM	NP	NP	28.9
Brown Sand	2A-3	0	66	25	9	SM	NP	NP	26.1
(el. 707.5)	2A-4	0	67	23	10	SM	NP	NP	26.2
2	1A-1	0	66	25	9	SM	NP	NP	33.3
Dark Brown	1A-2	0	64	25	11	SM	NP	NP	32.4
Sand	1A-3	0	64	26	10	SM	NP	NP	31.2
(el. 706.5)									

Split-Spoon Boring Sample	Elevation	Gravel (%)	Sand (%)	Fines (%)	Class	LL	PI	w (%)
SS-138	712.0	0	51	49	SM	28.1	2.5	24.0
SS-138A	713.2	0	50	50	SM	29.0	3.0	25.1
	711.2	0	64	36	SM	NP	NP	22.1
	707.4	0	60	40	SM	28.0	2.0	35.6
	705.4	0	69	31	SM	22.0	1.0	27.8
	705.0	0	79	21	SM	NP	NP	29.1
	703.0	0	79	21	SM	NP	NP	38.4
SS-138B	710.6	0	58	42	SM	27.0	3.0	24.7
	708.6	0	54	46	SM	34.0	5.0	36.2
	706.6	0	63	37	SM-SC	27.0	5.0	30.0
SS-138C	710.6	0	62	38	SM-SC	27.0	4.0	27.5
	708.6	0	54	46	SC	31.0	11.0	34.1

Table 2.5-39 Summary of Classification Data

Split-Spoon Boring Sample	Elevation	Gravel (%)	Sand (%)	Fines (%)	Class	LL	PI	w (%)
SS-49A	690.7	2	67	31	SM	NP	NP	30.0
SS-50	697.8	0	57	43	SM	NP	NP	28.2
SS-50	693.8	0	53	47	SM	NP	NP	31.5
SS-134	710.5	0	74	26	SM	NP	NP	29.3
SS-134A	709.5	0	65	35	SM	23.0	1.0	30.0
SS-135A	708.5	0	71	29	SM	NP	NP	34.2
SS-65	706.0	0	66	34	SM	28.9	3.5	28.2
SS-65B	709.2	0	62	34	SM	25.0	1.0	33.1
SS-65B	707.2	0	66	34	SM	25.0	1.0	32.5
SS-138A	707.2	10	46	44	SM	25.0	2.0	28.1
SS-140	706.7	0	64	36	SM	NP	NP	38.7

Table 2.5-40 Summary of Classification Data

Split-Spoon Boring Sample	Elevation	Gravel (%)	Sand (%)	Fines (%)	Class	LL	PI	w (%)
SS-158	711.5	0	56	44	SM	22.7	2.5	32.2
SS-162	713.8	0	64	36	SM	NP	NP	34.3
SS-163	717.0	0	55	45	SM	27.2	3.3	31.1
SS-163	715.0	0	57	43	SM	29.7	4.7	33.5
SS-163A	717.5	0	55	45	SM	30.0	3.0	36.3
SS-84	713.4	0	58	42	SM	24.8	2.2	30.1
SS-128	712.1	0	83	16	SM	NP	NP	23.7
SS-125	714.4	0	92	8	SM	NP	NP	20.0
SS-25	715.6	0	52	48	SM	NP	NP	29.2
SS-130	715.7	0	77	23	SM	NP	NP	17.8
SS-130	713.7	0	85	15	SM	NP	NP	15.5

Table 2.5-41 Comparison of Classification and Density Data of Test Pit and Undistributed Boring Samples (Sheet 1 of 2)

			Grain Size Analysis								
Pit No.	Sample No.	Class	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL (%)	PI (%)	W (%)	pcf[d]	R_D[1] (%)
1 (el. 706.6)	1A-1	SM	0	57	27	16	NP	NP	24.7	91.6	88.5
	1A-2	SM	0	67	21	12	NP	NP	28.6	88.7	78.5
	1A-3	SM	0	63	23	14	NP	NP	28.5	89.5	81.5
	1A-4	SM	0	64	24	12	NP	NP	26.9	88.8	78.9
2 Red to Brown Sand (el. 707.5)	2A-1	SM	0	69	22	9	NP	NP	26.7	84.6	68.8
	2A-2	SM	0	69	20	11	NP	NP	28.9	82.9	61.9
	2A-3	SM	0	66	25	9	NP	NP	26.1	83.6	64.7
	2A-4	SM	0	67	23	10	NP	NP	26.2	82.8	61.5
2 Dark Brown Sand (el. 706.5)	1A-1	SM	0	66	25	9	NP	NP	33.3	85.7	61.7
	1A-2	SM	0	64	25	11	NP	NP	32.4	86.1	63.3
	1A-3	SM	0	64	26	10	NP	NP	31.2	86.4	64.3

Table 2.5-41 Comparison of Classification and Density Data of Test Pit and Undistributed Boring Samples (Sheet 2 of 2)

			Grain Size Analysis								
Undistributed Boring No.	el.[2]	Class	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL (%)	PI (%)	W (%)	pcf[d]	R_D[1] (%)
US-50-1	701.4	SM	0	59	25	16	31.6	6.1	26.6	92.4	ND[3]
	698.9	SM	0	82	14	5	NP	NP	33.0	84.0	ND
	696.4	SM	0	88	9	4	NP	NP	28.9	93.1	ND
	695.3	SM	0	53	34	14	23.1	NP	31.1	90.4	ND
	694.2	SM	0	80	15	5	NP	NP	30.5	93.5	ND
US-50-1A	703.9	SM	0	64	24	12	NP	1.0	25.9	95.0	ND
	701.6	SM	0	67	22	11	NP	NP	37.8	79.2	ND
US-65-1	711.9	SM	0	70	22	8	NP	NP	22.2	88.6	ND
	709.4	SM	2	60	25	13	NP	NP	22.7	90.3	ND
	707.2	SM	0	65	24	11	NP	NP	33.4	87.0	ND
	705.2	SM	3	49	30	16	26.1	2.8	31.6	92.3	ND
US-77	715.1	SM	0	67	22	13	NP	NP	28.9	92.2	ND
US-92	715.9	SM	5	74	15	6	NP	NP	16.0	96.6	ND

Notes:

1 R_D was determined in accordance with ASTM D2049.

2 Elevation at top of sample.

3 Not Determined.

Table 2.5-42 Watts Bar Nuclear Plant
Soil-Supported Structures
Representative Basal Gravel Samples
Summary of Laboratory Test Data

Boring No.	Soil Elevation	Symbol	Average In-Situ Dry Density (From Shelby Tubes)	Consolidated Dry Density (From 12" x 12" x 6" Shear Box)	Direct Shear Tests: Test Dry Density	φ	c	Test Dry Density	φ	c
				pcf	pcf	deg	tsf	pcf	deg	deg
125	711.4-705.2	GP-GM	97.0	121.6	113.8	40.0	0.00	121.3	41.0	0.03
126	714.1-708.0	G-SM	131.0	121.8	120.0	36.5	0.00	-----	----	----
127, 128	711.2-708.6	GP-GM	126.9	133.7	123.7	42.0	0.00	132.5	43.5	0.07
129	712.1-706.6	GP-GM	117.0	122.0	116.2	39.0	0.00	123.6	41.0	0.0
130	712.1-705.0	G-SM	120.3	121.6	120.8	37.5	0.10	-----	----	----

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Table 2.5-43 Watts Bar Nuclear Plant
Soil-Support Structures
Undistributed Sampling
Summary of Laboratory Test Data
(Sheet 1 of 3)

Elevation	Soil Symbol	Nat. Moist		Grain-Size Analysis					Atterb. Limits				Triaxial Q				Saturated Triaxial R				
				Gravel	Sand	Silt	Clay	D_{10}	Liq. Limit	Plastic Index	Dry Dens.	Void Ratio	Undisturbed		Apparent		Effective		Consolidation		
		%	% Sat.	%	%	%	%	mm	%	%	pcf		φ deg.	c tsf	φ deg.	c tsf	φ deg.	c tsf	Cc	Cr	Pc tsf
Boring US-125, 104.3 S 382.5 E, Surface Elevation 727.4																					
706.8-705.8	GW	11.1	—	62	33	5	0	.3024	NP	NP	—	—									
705.8-705.3	G-SP-SM	27.8	100.0	22	70	8	0	.1542	NP	NP	98.4	.699									
705.3-704.3	G-SP-SM	22.0	88.0	12	80	8	0	.1700	NP	NP	100.1	.671									
Boring US-126, 184.5 N 379.9 E, Surface elevation 729.1																					
709.6-709.1	GP-GM	9.1	—	54	38	8	0	.1348	NP	NP	131.3	—									
709.1-707.5	SM	19.0	83.4	6	46	38	10	.0045	29.1	3.4	106.4	.636	8.0	1.03	30.8	0.00	37.5	0.00	0.14	0.01	2.4
Boring US-126A, 186.4 N 381.9 E, Surface elevation 729.1																					
709.4-707-9	SM	17.3	82.1	11	66	19	4	.0248	NP	NP	108.8	.579									
Boring US-127, 235.8 N 117.5 E, Surface elevation 728.2																					
711.2-709.6	GP	7.8	85.4	63	33	4	0	.2789	NP	NP	135.0	.243									
709.0-707.2	G-SM	23.3	90.5	14	56	22	8	.0083	37.3	8.6	100.5	.708	5.0	0.78	15.2	0.25	22.6	0.15	0.15	0.01	5.0
707.0-705.3	SM	24.4	96.1	1	49	42	8	.0067	35.9	5.0	100.9	.697									
Boring HD-127, 235.8 N 122.5 E, Surface elevation 728.1																					
709.9-709.3	GP	23.9	—	61	34	5	0	.2410	NP	NP	—	—									
709.1-707.2	G-SM-SC	15.3	90.2	13	57	26	4	.0156	26.7	6.4	117.6	.410									
Boring US-128, 191.3 N 7.5 W, Surface elevation 727.1																					
710.1-708.1	GP-GM	8.6	—	61	33	6	0	.1816	NP	NP	128.7	—									

**Table 2.5-43 Watts Bar Nuclear Plant
Soil-Support Structures
Undistributed Sampling
Summary of Laboratory Test Data
(Sheet 2 of 3)**

																	Saturated				
									Atterb. Limits				Triaxial Q				Triaxial R				
	Soil	Nat.			Grain-Size Analysis				Liq.	Plastic	Dry	Void	Undisturbed		Apparent		Effective		Consolidation		
Elevation	Symbol	Moist		Gravel	Sand	Silt	Clay	D_{10}	Limit	Index	Dens.	Ratio	φ	c	φ	c	φ	c	Cc	Cr	Pc
		%	% Sat.	%	%	%	%	mm	%	%	pcf		deg.	tsf	deg.	tsf	deg.	tsf			tsf
702.9-702.4	ML-CL	30.2	—	0	40	48	12	.0035	43.5	17.2	105.1	—									
Boring US-128A, 187.3 N 7.5 W, Surface elevation 727.1																					
710.6-710.1	GP-GM	10.1	—	52	40	8	0	.1073	NP	NP	122.6	—									
710.1-708.6	CL	18.7	82.8	0	10	57	33	—	40.8	18.5	105.6	.621	14.0	0.60	23.5	0.20	34.5	0.00	0.13	0.01	2.3
Boring SH-128, 189.3 N 7.5W, Surface elevation 727.1																					
707.8-706.1	G-SM	10.9	71.0	15	57	24	4	.0148	28.3	4.7	121.1	.427									
703.1-702.7	SC	24.6	—	1	49	37	13	.0035	43.5	17.2	105.1	—									
Boring US-129, 144.3 N 227.5 W, Surface elevation 725.3																					
708.3-707.0	GP-GM	15.3	—	45	45	10	0	.0756	NP	NP	111.2	—									
707.0-706.1	CL-CH	30.4	93.5	1	25	34	40	—	50.6	25.1	90.7	.898	0.5	0.59	14.3	0.23	25.0	0.00	0.32	0.06	0.8
Boring HD-129, 144.3 N 232.5 W, Surface elevation 725.6																					
709.1-707.7	GP-GM	14.8	100.0	53	35	8	4	.0671	19.2	3.0	127.7	.367									
707.7-706.5	G-CH	33.4	94.8	18	16	39	27	-----	51.9	26.0	87.7	.981									
Boring US-130, 95.7 S 142.5 W, Surface elevation 724.7																					
710.2-709.7	GP-GM	11.0	95.9	49	39	8	4	.0650	NP	NP	129.0	.311									
709.7-709.4	GP-GM	8.8	100.0	67	25	5	3	.3090	NP	NP	138.5	.213									
704.7-703.3	CL	29.5	95.1	0	9	58	33	----	48.2	20.8	92.4	.851	2.0	1.17	15.0	0.15	24.5	0.10	0.22	0.02	3.6
703.2-701.1	G-SM-SC	8.8	----	17	61	18	4	.0217	25.8	6.4	108.1	---									

**Table 2.5-43 Watts Bar Nuclear Plant
Soil-Support Structures
Undistributed Sampling
Summary of Laboratory Test Data
(Sheet 3 of 3)**

Elevation	Soil Symbol	Nat. Moist %	Nat. Moist % Sat.	Gravel %	Grain-Size Analysis Sand %	Grain-Size Analysis Silt %	Grain-Size Analysis Clay %	D_{10} mm	Atterb. Limits Liq. Limit %	Atterb. Limits Plastic Index %	Dry Dens. pcf	Void Ratio	Triaxial Q Undisturbed φ deg.	Triaxial Q Undisturbed c tsf	Apparent φ deg.	Apparent c tsf	Saturated Triaxial R Effective φ deg.	Saturated Triaxial R Effective c tsf	Consolidation Cc	Consolidation Cr	Consolidation Pc tsf
700.7-699.3	SC	14.8	----	0	63	32	5	.0112	27.6	7.7	119.9	---									
Boring HD-130, 95.7 S 137.5 W, Surface elevation 724.7																					
707.7-706.3	GP-GM	11.1	----	52	41	7	0	.1777	NP	NP	107.5	---									
706.2-705.4	G-SM	12.4	81.9	30	58	9	3	.0523	NP	NP	119.7	.411									
705.4-704.4	CL	24.1	99.3	0	12	54	34	----	48.8	24.0	104.4	.683									
Boring SS-130-A, 100.0 S 130.0 W, Surface elevation 724.7																					
706.0-705.6	GC	12.5	64.2	47	35	11	7	.0144	29.1	8.5	112.0	.536									
705.6-704.0	CH-MH	24.8	93.8	0	9	52	39	----	57.5	28.4	100.3	.743									

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