TENNESSEE VALLEY AUTHORITY

CHATTANOOGA, TENNESSEE 37401
400 Chestnut Street Tower II

January 12, 1984

Director of Nuclear Reactor Regulation
Attention: Ms. E. Adensam, Chief
Licensing Branch No. 4
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Dear Ms. Adensam:

In the Matter of the Application of	)	Docket Nos. 50-390
Tennessee Valley Authority	)	50-391

Enclosed for NRC review is information concerning TVA activities at Watts Bar Nuclear Plant related to the issue of liquefaction potential. Enclosure 1 provides TVA responses to NRC and Corps of Engineer concerns about slope stability. Enclosure 2 provides amended FSAR material which reflect remedial action. Enclosure 3 provides a report entitled "Site-Specific Top-of-Ground Motion for ERCW Pipeline with Response to the NRC Staff Concerns." This report dated September 23, 1983 was prepared for TVA by Woodward-Clyde Consultants.

If you have any questions concerning this matter, please get in touch with D. P. Ormsby at FTS 858-2682.

Very truly yours,

TENNESSEE VALLEY AUTHORITY

D S Kammer

D. S. Kammer
Nuclear Engineer

Sworn to and subscribed before me
this 12th day of January 1984
Paulette H. White
Notary Public
My Commission Expires 9-5-84

Enclosures (3)
cc: U.S. Nuclear Regulatory Commission (Enclosures)
Region II
Attn: Mr. James P. O'Reilly, Regional Administrator
101 Marietta Street, NW, Suite 2900
Atlanta, Georgia 30303

Question 1:

There is a large distance, covered by the future 161 kV switchyard, between the ERCW piping and trench. Has the stability of this area been checked for a seismic event?

Response:

The area designated as the future 161 kV switchyard has been regraded as shown on proposed figure 2.5-584 of the FSAR. The grade (slope) of this area is very gentle (between 1 and 2 degrees). The soil profile is fairly consistent between trench A and the ERCW piping. A clay cap approximately 14 feet thick overlays a silty sand layer. A typical cross section is shown in figure 1. The stability of this area was checked using the block and wedge method of analysis with the same soil strengths used in the underground barrier analysis as discussed in section 2.5.5.2.3 of the FSAR. The pseudo-static analysis performed for the underground barrier assumed zero shear strength (i.e., complete liquefaction) for the silty sand layer and a peak acceleration occurring simultaneously. This method is very conservative and has limits on its appropriateness. The most conservative assumption is to assume the total mass has a constant acceleration in the direction of instability. An earthquake motion is cyclic in nature in that the earthquake load randomly changes direction during an earthquake. Another conservative assumption is to assume complete soil liquefaction occurs prior to or in conjunction with the peak acceleration. This assumption was used in the barrier analysis to achieve the most conservative (highest) earth pressure in conjunction with the peak acceleration for determining the barrier width.

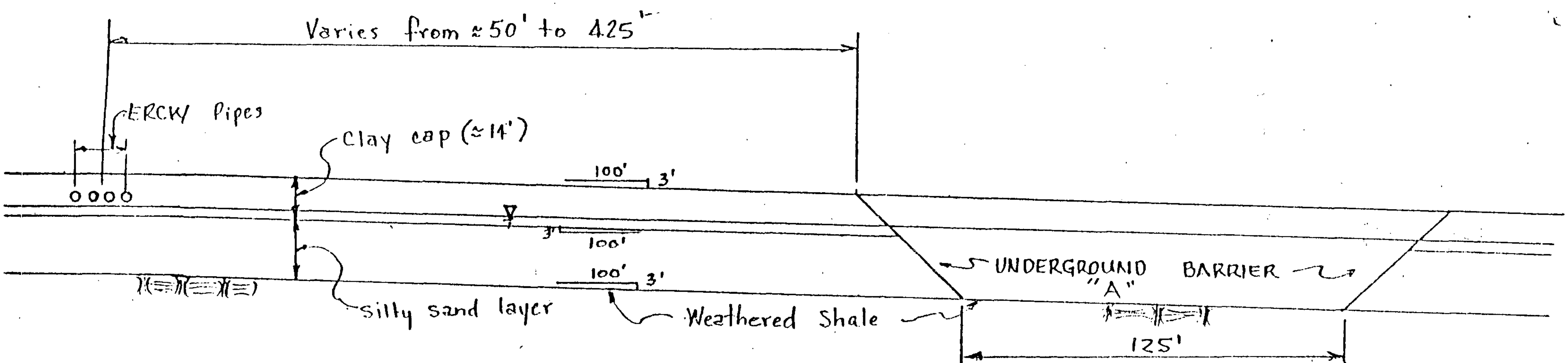
Because (1) the area being considered is relatively flat (see figure 1), (2) the earthquake loads are cyclic (i.e., not constant in the direction of instability), and (3) the potential soil liquefaction will not occur until after the peak acceleration, the most appropriate case to examine for the area between trench A and the ERCW piping is the postearthquake situation assuming complete liquefaction of the silty sand layer. The factor of safety for this case is 1.7. The factor of safety for the area prior to an earthquake is 12.6. Based on these analyses, it is concluded that these slopes are stable.

Question 2:

There is a grade slope above the trenches. Has the effect of this slope been accounted for in calculations of earth pressure used in the stability of the underground barrier?

Response:

Because the slight slope of the ground was so insignificant above the trenches, the slope was not considered in the initial calculation. The effect of this slope has now been evaluated in the analysis and the result is that the effect of the slope is negligible and affects the resultant factors of safety less than 1 percent.



Varies from ≈50' to 425'
ERCW Pipes
Clay cap (≈14')
100'
3'
3'
100'
100'
3'
Silty sand layer
Weathered Shale
UNDERGROUND BARRIER
"A"
125'

FIGURE 1

Question 3:

What is the effect of the soil liquefaction on the stability of the intake pump station?

Response:

The stability of the intake pump station (IPS) was checked assuming liquefaction of the sandy soils behind the IPS. The controlling case was when the SSE is combined with a 25-year flood (to raise the saturation level in the soil) followed by sudden drawdown in the intake channel due to downstream dam failure (least hydrostatic resistance and weight in the IPS). This case assumes the above scenario in conjunction with liquefaction and the peak earthquake acceleration. The resultant factor of safety for the stability analysis was 1.4 (minimum required factor of safety is 1.1).

ENCLOSURE 2

WATTS BAR NUCLEAR PLANT UNITS 1 AND 2

AMENDED FSAR MATERIAL WITH DESCRIPTION OF
TVA REMEDIAL ACTION FOR POTENTIAL LIQUEFACTION

NOTE: Five copies of the following oversized drawings were forwarded directly to the NRC project manager with a copy of this letter.

2.5-220
2.5-221
2.5-221a
2.5-273
2.5-520
2.5-521
2.5-522
2.5-523
2.5-544
2.5-545
2.5-546
2.5-547
2.5-548
2.5-571 (Sheets 1-4)
2.5-572
2.5-573
2.5-575
2.5-576 (Sheets 1 and 2)
2.5-577
2.5-578
2.5-579
2.5-580
2.5-581
2.5-582
2.5-583
2.5-584
3.8.4-46

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 0 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, -221, and -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. 50
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 -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, 50 since only fine-grained soils were used for Earthfill A1. Table 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 prepares a family of compaction curves for all soil classes at the site (see Figure 2.5-235 and 2.5.271), further divided into subclasses to be used by the inspectors of backfill placing and the project laboratory for construction control and day-to-day testing of fill compaction. These tests by the project laboratory for dry density, moisture content, and degree of compaction. A minimum of at least one test for each 2000 cubic yards placed shall be performed throughout the course of the work. Additional sampling and testing are done as required by the inspectors or engineers in charge.

The quality of the backfill is documented by measuring the in-place density. The inplace compaction is expressed as a percent 44 40 of the maximum density at optimum moisture content for the backfill material being placed. A backfill log book is maintained containing all pertinent information concerning daily backfill operation.

In addition, a penetrometer is used, correlated with penetra-tion charts prepared by the central laboratory (see Figure 2.5-234 and 44 2.5.272) to maintain a continual check on the compaction of the backfill. At Watts Bar Nuclear Plant, Class A backfill is placed around all Category I structures. This material, which is selected earth placed in not more than 6-inch layers, has a minimum required compaction of 95 percent of the maximum standard density at optimum moisture content.

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

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

A third class of fill is also used, Class C, using unclassified fills to be placed in approximately 12-inch layers and compacted with hauling equipment. This fill class is 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 is composed of material originally excavated from the intake channel. The material is compacted to 95 percent of maximum density at optimum moisture content.

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

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

In areas where earthfills with differing compaction requirements adjoin, the most heavily compacted fill is placed prior to the placement of the fill of lower density.

2.5.4.5.1.4 Construction Control

All earthfills is placed in accordance with the provisions of TVA's General Construction Specification No. G-9 for Rolled Earthfill for Dams and Power Plants. The following information summarizes the construction control which is described in that document. This program is also applicable for all engineered granular fills.

All fill operations are accomplished in the presence of a trained inspector. The inspector has the authority to suspend fill operations whenever weather or material conditions are judged unsuitable. His responsibilities include material quality, selection, excavation, hauling, placement, and compaction control. During placement, periodic construction control tests are made to ensure that a suitable fill is obtained. This testing determines soil classification, moisture content, inplace density, relative density (granular fill only), and degree of compaction (earthfill only). The frequency of testing is as specified in General Construction Specification G-9. The inspector may require additional testing to conclusively identify material or check compaction. A project laboratory has been established at the plant site to perform the necessary testing. A project foundation specification and a series of construction control procedures relay unique construction requirements to the construction personnel.

2.5.4.5.2 Granular Fill

2.5.4.5.2.1 General

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

Section 1032 Material - 50

A granular fill material, consisting of crushed stone or sand and gravel, placed around and below safety-related features in lieu of earthfill in certain locations. The granular fill material is suitable for compaction to a dense, stable mass and consists of sound, durable particles which are 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 is 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 are as follows: 50

Sieve Size	Percent (by Weight) Passing 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 -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 are 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 will be incremented during analysis to check the effect of pore pressure buildup.

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

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 is placed adjacent to an earthfill, the granular fill is placed and compacted prior to the placement of the earthfill. Granular fill is placed and compacted to a relative density as specified on drawings or in construction specifications and as determined by ASTM D 2049. 50
The moisture content of the material is adjusted as necessary to obtain the required relative density. The construction control program for granular fill is 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.

Section 1075 Material -

A free-draining granular fill material, consisting of crushed stone or sand and gravel, 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.

The granular fill material is graded within the following limits:

Sieve Size	Percent (by Weight) Passing: 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	50
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. The particle diameter at 60 percent 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 percent during ground motions with accelerations in excess of approximately 0.1 g; and that for relative densities greater than 75 percent, liquefaction for most earthquake loadings is unlikely.

Using the rules outlined above, only one area of the plant site was found which contained potentially liquefiable soils. 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. 26

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. 26 Q362.5

As a result of several meetings with the NRC and the NRC's review references 165, 166, and 167, 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 50 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 -64 tabulates the samples that would potentially liquefy, i.e., (FS 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 -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 -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 50
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.

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 8, 9, and 10. 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 164). 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
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 -578 show the potential settlement calculated using the 1.5% strain (1.5%E) 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%E) criteria are also shown on Figures 2.5-571 through -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%) 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 3.7.2.1.2.

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, Seismic Design.

2.5.4.10 Static Analysis

2.5.5 Stability of Slopes

2.5.5.1 Slope Characteristics for Essential Raw Cooling Water Intake Channel Slopes 50

The intake channel is a manmade feature extending approximatelv 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 will be constructed by excavating two 50
trenches. The location of the underground barrier trenches are shown on Figures 2.5-580 and -581. The locations were based on the extent of the potentially liquefiable soils along the piping and conduit alignments as shown on Figures 2.5-571 through -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 will be backfilled with soils excavated from the trenches, if acceptable, and soil from approved onsite borrow areas. The method of construction and construction control will be in accordance with the requirements and notes on Figures 2.5-580 and -581. The results of the soils investigation and testing of the borrow materials is 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 - 575, and 2.5-577 and -578), there are no 50
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) and, 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 Cooling 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 with reservoir elevation 675, groundwater 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. 24

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

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 28
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 soild 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. Q362.17

2.5.5.2.3 <u>Design Criteria and Analysis for the Underground Barrier for the ERCW Pipeline and 1E Conduit Alignment</u>

The location of the underground barrier is shown on Figures 2.5-580 and -581. The underground barrier was analyzed for the following cases:

Case	Required Factor of Safety
1. Safe Shutdown Earthquake, but prior to liquefaction	1.0
2. Safe Shutdown Earthquake after liquefaction, but prior to dissipation of pore water pressure	1.0

50

Section 2.5.4.6 describes the study made to determine the design groundwater for the piping and conduit alignments. The results of that groundwater study were used in the analyses of the underground barrier. Figure 2.5-583 shows a loading diagram of how the underground barrier was analyzed. Seven sections of the barrier were analyzed. Figure 2.5-582 shows the locations of the seven sections. The most critical sections were Section 1 for Trench A and Sections 6 and 7 for Trench B. Case 2 is the controlling case in the analysis for each section, since passive earth pressure is included in Case 1, but assumed to be zero for Case 2. Figure 2.5-583 shows the results of the analysis. Case 1 was dropped from the analysis, when it became obvious that Case 2 controlled the design and analysis of the barrier. Due to the urgency to complete the construction of the barriers 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 tests 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 test results are presented in Section 2.5.4.5.1.

50

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. The results of this analysis are given on Figure 2.5-583. Figure 2.5-584 shows the final grading for the area of the underground barrier.

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 Sections 2.5.4.5.1.3 and 2.5.4.5.2.2 respectively.

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.

148. Philippe, R. R., (1940), 'Shear Tests; Foundation of Lock Wall, Watts Bar Dam,' TVA Watts Bar Hydro File 61-10, January 6, 1940.

149. TVA, (1948), Drawings for the Watts Bar Project, Technical Monograph No. 61.

150. TVA, (1949), The Watts Bar Project, Technical Report No. 9.

151. Terzaghi, K. and R. B. Peck, (1967), Soil Mechanics in Engineering Practice, John Wiley and Sons, Inc.

152. Timoshenko, S. and J. N. Goodier, (1951), Theory of Elasticity, McGraw-Hill.

153. Terzaghi, (1947), Theoretical Soil Mechanics, Wiley.

154. Mooney, H. M., (1973), 'The Handbook of Engineering Seismology,' Bison Instruments, Inc., Minneapolis, Minnesota, 105 p.

155. D'Appolonia, E., 'Dynamic Loadings,' Journal of the Soil Mechanics and Foundations Division, ASCE, January 1970.

156. Brown, L., D. Albaugh, J. Brewer, F. Cook, L. Jensen, S. Kaufman, G. Long, J. Oliver, S. Schilt, and D. Steiner, (Department of Geological Sciences, Cornell University, Ithaca, New York 14853) in Abstracts of American Geophysical Union Annual Meeting, Washington, D.C., May-June 1979.

157. Cook, F. A., D. S. Albaugh, L. D. Brown, S. Kaufman, J. E. Oliver, and R. D. Hatcher, 1979, Thin-skinned tactonics in the crystalline southern Appalachians: COCORP seismic reflection profiling of the Blue Ridge and Piedmont: Geology, v. 7, p. 563-567.

158. Seed, H. Bolton and Idriss, I. M., 'Evaluation of Liquefaction Potential of Sand Deposits Based on Observations of Performance in Previous Earthquakes,' In Situ Testing to Evaluate Liquefaction Susceptibility, Proceeding of a Session at ASCE National Convention, St. Louis, Missouri, October 26-30, 1981.

159. Seed, H. Bolton and Idriss, I. M., 'Simplified Procedure for Evaluating Soil Liquefaction Potential, ASCE, Journal of the Soil Mechanics and Foundations Division, September 1971.

160. NUREG-0026, 'Evaluation of Soil Liquefaction Potential for Level Ground During Earthquakes,' Prepared by Shannon & Wilson, Incorporated, and Agabian Associated for the U.S. Nuclear Regulatory Commission.

161. Seed, H. Bolton, Idriss, I. M. Makdisi, F., and Banerje N., 1975, 'Representation of Irregular Stress Time Histories by Equivalent Uniform Stress Series in Liquefaction Analyses,' Earthquake Engineering Research Center, Report No. EERC 75-29, University of California, Berkeley, October.

162. Silver, M. L., and Seed, H. Bolton, 1969, 'The Behavior of Sands under Seismic Loading Conditions,' Earthquake Engineering Reserch Center, Report No. EERC 69-16, University of California, Berkeley.

163. University of California, 1973, 'Computer Program for Slope Stability Analysis (STABR),' Civil Engineering Department, University of California, Berkeley. 50

164. Lee, K. L. and Albaise, A., 'Earthquake Induced Settlement in Saturated Sands,' ASCE, vol 100, GT4, April 1974.

165. 'Watts Bar Nuclear Plant - Liquefaction Evaluation of the ERCW Pipeline Route,' February 8, 1982. (Submitted to NRC by letter from L. M. Mills to E. Adensam dated February 9, 1982.)

166. 'Watts Bar Nuclear Plant - Liquefaction Evaluation of the ERCW Pipeline Route - Revision 1,' March 17, 1982. (Submitted to NRC by letter from L. M. Mills to E. Adensam dated March 19, 1982.)

167. 'Liquefaction Evaluation of the ERCW Pipeline Route - Watts Bar Nuclear Plant - Task 9,' November 1982. (Submitted to NRC by letter from L. M. Mills to E. Adensam dated November 16, 1982.)

3.7.3.12 Buried Seismic Category I Piping Systems and Tunnels

Category I buried piping which penetrates structures where fill settlement or seismic movements are expected to be high is protected from differential movement of the soil and structure by Category I concrete slabs or encasements. The slab or encasement is supported by a bracket on the structure on one end and on undisturbed or Class A backfill at the other end. Bearing piles 41 are used if required to support the slab. The encased pipes are insulated to prevent bonding between the pipes and concrete. For details of the slab at the intake pumping station and the encasement at the Diesel Generator Building, refer to Section 3.8.4.4.8.

For seismic classed buried piping that penetrates structures in areas where very little fill is involved and seismic movements are low, protection from differential movement of the soil and structure is provided by an oversized opening in the structure. The annular space between the pipe and opening is filled with a resilient material. The first support inside the structure is located to allow for relative movement of the pipe and structure. The soil-structure interface is treated as an anchor, and stresses are limited to code allowables.

The ERCW piping was evaluated for potential settlement due to soil liquefaction as discussed in Section 2.5.4.8. The potential settlements used for the evaluation were determined in the liquefaction evaluation using the strain criteria specified by the NRC staff which are shown on Figures 2.5-571 through -575. The effect of these potential settlements was evaluated for the entire length of pipe and also at all building interfaces. The evaluation of the effect of these potential settlements was done in two phases.

The first phase was a preliminary screening which involved calculations to identify areas of the pipe which may undergo excessive settlement. In the preliminary screening, the 50 boundaries of the pipe system, the pipe sizes, and pipe materials were determined. Because of the size and length of pipe involved, a 60' length was chosen as sufficient to model the system. A fixed-fixed end model was assumed to describe the piping for the initial calculations. Using the standard equation for maximum deflection for a fixed-fixed end model:

$$Y_{max} = \frac{ML^2}{32EI}$$

M = Resultant moment

L = Span length

E = Young's modulus

I = Moment of inertia

The settlement can be determined if the resulting moment were known. ASME III Code (1971 edition, Summer 1973 Addenda, NC-3652.3) states that the effects of any single nonrepeated anchor movement is governed by Equation 12:

$$\frac{iM_{\Delta}}{Z} \leq 3.0\ S_c$$

i = Stress intensification factor

Z = Section modulus

S_c = Allowable stress at room temperature

To expand this equation to include thermal effects (assuming $M_c = 0$) would involve adding it to Equation 11 (1971 ASME III Code, Summer 1973 Addenda, NC-3652.3) thus;

$$\frac{iM_{\Delta}}{Z} \leq 3.0\ S_c + S_A$$

S_A = Allowable stress for expansion

Since the pipe sizes and materials are known, and the stress intensification factor can be calculated, the resultant moment at any point on the pipe can be determined. Thus the potential settlement can be found by using the standard equation for the fixed-fixed end model. The results from these preliminary screening calculations were used in conjunction with the potential settlement evaluation, Section 2.5.4.8, to identify potential areas of excessive settlement, either at the buildings or along the pipeline. 50

The second phase of the evaluation consisted of making rigorous piping analyses at the potential areas of excessive settlement. There were three areas along the pipeline with apparent problems that were modeled into the T-PIPE piping analysis program. These areas were modeled for a distance on both sides of the potential high settlement area. The areas that were modeled were: (1) from the intake pump station to boring SS-131; (2) from boring SS-141 to boring SS-90; and (3) from boring SS-163 to boring SS-159.

At these areas the potential settlements were used as input in the phase II analysis to give the most conservative results. In all cases, the stress levels are below the ASME Code allowable for settlement induced loads (Reference 1971 ASME Code, Summer 1973 Addenda, NC-3652.3).

Where practical, seismic classed buried piping is routed to avoid areas of weak soils. Where weak soils are encountered, the bad material is removed and replaced by backfill. The backfill is placed to standards that insure suitable bearing conditions, therefore, the transition from one material to another, i.e., insitu soil to backfill should not be a problem. In lieu of the above, in some cases an analysis is performed to show that the 41 pipe has sufficient strength to bridge the discontinuity and support the soil above the pipe without exceeding the allowable stress of the piping material.

Category I piping supported by two structures is attached to only one of the two at the interface of the two structures. Sufficient clearance is provided between the pipe and the second structure to permit maximum relative longitudinal and radial movements. The seismic spectral data for these systems are developed by superimposing data from both buildings and developing curves which envelop the individual spectral data for two perpendicular, horizontal plant directions.

Buried piping complies with the ASME Boiler and Pressure Vessel Code, Section III and is analyzed seismically as follows.

> The soil is considered to be a horizontal 1-layer system which responds to the earthquake by moving in a continuous sinusoidal plane wave and supported by a second layer or base material. The top layer is assumed to pick up accelerations from the base material. 49

Utilizing the average values for the shear wave velocity and density for the top layers, the ground deformation pattern in terms of wave length and amplitude is determined. The buried pipes are assumed to deform along with the surrounding soil layers. No relative displacement between the soil and the buried piping is considered.

The average shear wave velocity of a single layer representation of a multi-layed soil system may be determined by:

$$V_{ST} = \frac{\sum V_S \ h'}{h}$$

Where: V_{ST} = Average shear velocity in the top layers of soil, ft/sec
V_S = Shear velocity in each layer of soil, ft/sec
h = Depth of each layer of soil, ft
h = Total depth of top layers of soil, ft 49

The fundamental period of the single layer is calculated from the following equation:

$$T = \frac{4\ h}{V_{ST}} \qquad \text{(seconds)}$$

If the depth of the soil layer varies over the distance traversed by the buried pipe, both cases, for maximum and minimum depths, are considered.

The maximum amplitude of the sine wave which represents the maximum displacement of the pipe is:

$$A = \text{Displacement} = \left(\frac{T}{2\pi}\right)^2 * \text{Accel}$$

where: T = Fundamental period, sec
Accel = Amplified soil acceleration value, in/sec^2

The wave length, L, is calculated as:

$$L = V_{ST}\ T$$

The bending moment resulting from the seismic disturbance, assuming the pipe follows the soil and deforms as a sine wave, is given by

$$M = \frac{2\ E\ I\ A}{(L/2)^2}$$ 49

Where: M = Maximum bending moment, in-lb
E = Modulus of the pipe, psi
I = Moment of inertia of the pipe, in^4
A = Maximum amplitude, in.
L = Wave length, in.

The corresponding bending stress is obtained by dividing the moment by the section modulus of the pipe. The above bending stress is combined with bending stresses due to other loads according to the applicable loading combinations.

The geotechnical parameters used in the seismic analysis of the ERCW system buried piping are:

Average soil shear wave velocity, V_{ST} = 1000 f/s (approx.)
Soil unit weight = 120 pcf
Average rock shean wave velocity = 5900 f/s
Rock unit weight = 170 pcf

The average soil shear wave velocity was determined by the layered approach using cross-hole geophysical data and corresponds well with the downhole geophysical data. In addition, a ± 30% variation of shear wave velocity is considered.

where

V_{ST} = Average shear wave velocity of the soil deposit

Using the results from the above equations, the bending moment due to the earthquake is

$$M = EID\left(\frac{\pi}{L}\right)^2$$ 43

where

E = Young modulus of conduit bank
I = Moment of inertia of conduit bank
L = One-half of the wave length

The conduit banks were evaluated for settlement due to the potential liquefaction of the underlying soil as discussed in Section 2.5.4.8 (see Figures 2.5-574 through -578 for the potential settlement values). The banks were evaluated for potential settlements between manholes and at building/conduit interfaces. The only area of potential structural inadequacy was at the intake pumping station (IPS). The conduit banks in this area (see Figure 3.8.4-46) required modification to accommodate the potential settlements. This modification consists of cutting 10 grooves on the sides and bottom of the banks. The 4 inch deep by 50 2 inch wide grooves begin 76' from the IPS and are spaced at 8 inch between centers for a distance of 6' along each bank. Settlement of the conduit banks will cause plastic hinges to develop at the grooves and at the pile supports farthest from the IPS. This results in a structural mechanism which will allow the conduit bank to settle without compromising the intended function of the encased conduits.

Class IE Electrical Systems Manholes and Handholes

These manholes and handholes are rigid structures which have the same motion as the soil deposits in which they are located. The soil deposits were analyzed as explained in Section 3.7.2.4. The accelerations obtained for the soil deposit at the level of the manholes and handholes were used to determine the inertia force on the structures and to calculate the increase in the static soil pressure using the shaking table experiments performed for the design of TVA's Kentucky hydro project 1 as discussed in Section 3.7.2.1.1.

Miscellaneous Yard Structures

The ERCW discharge overflow structures, ERCW standpipe, and other 43 miscellaneous yard structures are normally rigid structures.

These structures are designed for a rigid body acceleration. Dynamic soil pressures on the walls, if appropriate, are determined in accordance with Reference 1.

Category I Pile Supported Structures

For structures founded on piles, the acceleration at top of rock was considered to be amplified through the soil as discussed in Paragraph 3.7.2.4. The translational and rocking foundation springs included in the lumped mass model of the structure to characterize soil-structure interaction were calculated using Reference 3. The damping ratio used for soil-supported structures depended upon the predominant type of motion as explained in Reference 5. 43

A more detailed description of the seismic analysis of Category I pile-supported strucures is discussed below.

TABLE 2.5-54

SUMMARY OF LABORATORY TEST DATA

	Maximum Fines Gradation		Average Fines Gradation		Minimum Fines Gradation	
Minimum density, pcf	107.1		103.1		108.7	
Maximum density, pcf	143.1		139.5		143.9	
	Ø	C (tsf)	Ø	C (tsf)	Ø	C (tsf)
Triaxial Shear (Q)						
At 80% R_d	38.7	0.73	38.3	1.46	40.5	1.91
At 70% R_d	38.5	0.50	42.5	0.80	42.0	1.64
Triaxial Shear ($\overline{R}$)						
At 80% R_d	39.3	1.93	41.8	0.99	43.7	0.34
Direct Shear (S)						
At 80% R_d	39.4	0.30	42.0	0.52	44.2	0.63
At 70% R_d	36.0	0.35	44.0	0.24	42.5	0.52

R_d = Relative density

Added by Amendment 50

TABLE 2.5-45

WATTS BAR NUCLEAR PLANT

ERCW LIQUEFACTION

TRENCH A

SUMMARY OF LABORATORY TEST DATA

BORROW SOIL CLASSES

Class	I	II	III
Symbol	SM-SC	SC	CL
Mechanical and Hydrometer Analysis			
Gravel, percent	0	0	0
Sand, percent	70	51	40
Silt, percent	15	24	29
Clay, percent	15	25	31
Atterberg Limits			
Liquid limit, percent	24	28	34
Plastic limit, percent	19	17	19
Plasticity index, percent	5	11	15
Shrinkage limit, percent	--	--	--
Standard Proctor Compaction			
Optimum moisture, percent	13.1	14.1	15.9
Maximum density, pcf	116.6	114.4	110.8
Penetration resistance, psi	910	840	760
Shear Strength at 3% Dry of Optimum Moisture and at 95% of Maximum Unit Weight			
Triaxial R: ϕ, degrees	15.0	14.8	18.0
c, tsf	0.29	0.11	0.03
Shear Strength at 3% Dry of Optimum Moisture and at 100% of Maximum Unit Weight			
Triaxial R: ϕ, degrees	--	15.7	16.8
c, tsf	--	0.19	0.10
Percent of class in area	8	61	31
Natural moisture content, percent	18.5	19.4	20.7

Added by Amendment 50

TABLE 2.5-45a

WATTS BAR NUCLEAR PLANT

ERCW LIQUEFACTION, TRENCH A

SUPPLEMENTAL BORROW

SUMMARY OF LABORATORY TEST DATA

BORROW SOIL CLASSES

Group Symbol	1 ML	2 SM	3 ML
Mechanical and Hydrometer Analysis			
Gravel, percent	0	0	0
Sand, percent	16	54	43
Silt, percent	44	31	15
Clay, percent	40	15	22
Atterberg Limits			
Liquid limit, percent	47	26	34
Plastic limit, percent	29	25	26
Plasticity index, percent	18	1	8
Shrinkage limit, percent	--	--	--
Standard Proctor Compaction			
Optimum moisture, percent	21.4	17.3	18.8
Maximum density, pcf	99.7	108.4	105.3
Penetration resistance, psi	1180	860	800
Shear Strength at 3% Dry of Optimum Moisture and at 100% of Maximum Unit Weight*			
Triaxial R: ϕ, degrees	13.0	11.6	12.9
c, tsf	0.45	0.46	0.69
Percent of class in area	--	--	--
Natural moisture content, percent	--	--	--

*Group 2 tested at 95 percent of maximum unit weight.

Added by Amendment 50

TABLE 2.5-46

WATTS BAR NUCLEAR PLANT

ERCW LIQUEFACTION

TRENCH B

SUMMARY OF LABORATORY TEST DATA

BORROW SOIL CLASSES

Class	I	II	III
Symbol	SM	SM-SC	CL
Mechanical and Hydrometer Analysis			
Gravel, percent	0	0	0
Sand, percent	66	55	43
Silt, percent	22	24	28
Clay, percent	12	21	29
Atterberg Limits			
Liquid limit, percent	NP	28	30
Plastic limit, percent	NP	22	19
Plasticity index, percent	NP	6	11
Shrinkage limit, percent	--	--	--
Standard Proctor Compaction			
Optimum moisture, percent	15.3	15.6	15.8
Maximum density, pcf	110.7	110.3	109.8
Penetration resistance, psi	770	1025	1425
Shear Strength at 3% Dry of Optimum Moisture and at 95% of Maximum Unit Weight			
Triaxial R: ϕ, degrees	7.6	5.5	10.4
c, tsf	1.67	1.05	0.32
Percent of class in area	26	22	52
Natural moisture content, percent	25.0	28.4	22.2

Added by Amendment 50

TABLE 2.5-47

WATTS BAR NUCLEAR PLANT

ERCW LIQUEFACTION

BORROW AREA 9

SUMMARY OF LABORATORY TEST DATA

BORROW SOIL CLASSES

Class	I	II
Symbol	CL	CL-ML
Mechanical and Hydrometer Analysis		
Gravel, percent	0	0
Sand, percent	24	32
Silt, percent	40	27
Clay, percent	36	41
Atterberg Limits		
Liquid limit, percent	31	40
Plastic limit, percent	15	25
Plasticity index, percent	16	15
Shrinkage limit, percent	--	--
Standard Proctor Compaction		
Optimum moisture, percent	16.4	19.6
Maximum density, pcf	110.3	104.0
Penetration resistance, psi	350	680
Shear Strength at 3% Dry of Optimum Moisture and at 95% of Maximum Unit Weight		
Triaxial R: ϕ, degrees	12.3	8.0
c, tsf	0.11	0.57
Shear Strength at 3% Dry of Optimum Moisture and at 100% of Maximum Unit Weight		
Triaxial R: ϕ, degrees	11.6	--
c, tsf	0.28	--
Percent of class in area	50	50
Natural moisture content, percent	13.1	21.7

Added by Amendment 50

TABLE 2.5-48

WATTS BAR NUCLEAR PLANT

ERCW LIQUEFACTION

BORROW AREA 10

SUMMARY OF LABORATORY TEST DATA

BORROW SOIL CLASSES

Class	I	II
Symbol	CL	CL-ML
Mechanical and Hydrometer Analysis		
Gravel, percent	0	0
Sand, percent	33	19
Silt, percent	31	33
Clay, percent	36	48
Atterberg Limits		
Liquid limit, percent	39	45
Plastic limit, percent	23	26
Plasticity index, percent	16	19
Shrinkage limit, percent	--	--
Standard Proctor Compaction		
Optimum moisture, percent	20.6	25.4
Maximum density, pcf	103.0	93.2
Penetration resistance, psi	620	860
Shear strength at 3% dry of optimum moisture and at 95% of maximum unit weight.*		
Triaxial R: ϕ, degrees	11.9	15.2
c, tsf	0.21	0.09
Shear Strength at 3% Dry of Optimum Moisture and at 100% of Maximum Unit Weight		
Triaxial R: ϕ, degrees	--	15.0
c, tsf	--	0.12
Percent of class in area	86	14
Natural moisture content, percent	23.9	27.6

*At a density of 90 pcf on class II.

Added by Amendment 50

TABLE 2.5-49

WATTS BAR NUCLEAR PLANT

ERCW LIQUEFACTION

BORROW AREA 11

SUMMARY OF LABORATORY TEST DATA

BORROW SOIL CLASSES

Class	I
Symbol	ML
Mechanical and Hydrometer Analysis	
Gravel, percent	0
Sand, percent	21
Silt, percent	35
Clay, percent	44
Atterberg Limits	
Liquid limit, percent	44
Plastic limit, percent	29
Plasticity index, percent	15
Shrinkage limit, percent	--
Standard Proctor Compaction	
Optimum moisture, percent	22.2
Maximum density, pcf	99.8
Penetration resistance, psi	850
Shear strength at 3% dry of optimum moisture and at 95% of maximum unit weight.	
Triaxial R: ϕ, degrees	13.2
c, tsf	0.21
Percent of class in area	100
Natural moisture content, percent	26.9

Added by Amendment 50

TABLE 2.5-50

WATTS BAR NUCLEAR PLANT

ERCW LIQUEFACTION

BORROW AREA 12

SUMMARY OF LABORATORY TEST DATA

BORROW SOIL CLASSES

Class	I	II	III
Symbol	SM	CL-ML	CL-ML
Mechanical and hydrometer analysis			
Gravel, percent	0	0	0
Sand, percent	50	22	22
Silt, percent	26	39	40
Clay, percent	24	39	38
Atterberg limits			
Liquid limit, percent	32	40	42
Plastic limit, percent	25	25	26
Plasticity index, percent	7	15	16
Shrinkage limit, percent	--	--	--
Standard proctor compaction			
Optimum moisture, percent	16.8	17.8	19.2
Maximum density, pcf	108.8	106.5	103.7
Penetration resistance, psi	1165	1150	1140
Shear strength at 3% dry of optimum moisture and at 95% of maximum unit weight			
Triaxial R:			
ϕ, degrees	9.5	12.0	16.4
c, tsf	0.57	0.29	0.04
Shear strength at 3% dry of optimum moisture and at 100% of maximum unit weight			
Triaxial R:			
ϕ, degrees	--	--	12.5
c, tsf	--	--	0.39
Percent of class in area	12	55	33
Natural moisture content, percent	21.6	24.9	25.2

Added by Amendment 50

TABLE 2.5-51

WATTS BAR NUCLEAR PLANT

ERCW LIQUEFACTION

BORROW AREA 13

SUMMARY OF LABORATORY TEST DATA

BORROW SOIL CLASSES

Class	I	II	III
Symbol	ML	ML	MH
Mechanical and Hydrometer Analysis			
Gravel, percent	0	0	0
Sand, percent	24	23	12
Silt, percent	42	39	41
Clay, percent	34	38	47
Atterberg Limits			
Liquid limit, percent	37	41	52
Plastic limit, percent	26	27	35
Plasticity index, percent	11	14	17
Shrinkage limit, percent	--	--	--
Standard Proctor Compaction			
Optimum moisture, percent	19.2	20.0	23.3
Maximum density, pcf	106.6	105.1	98.8
Penetration resistance, psi	650	800	740
Shear strength at 3% dry of optimum moisture and at 95% of maximum unit weight			
Triaxial R: ϕ, degrees	15.6	14.5	18.3
c, tsf	0.15	0.14	0.02
Shear strength at 3% dry of optimum moisture and at 100% of maximum unit weight			
Triaxial R: ϕ, degrees	11.7	14.5	14.7
c, tsf	0.66	0.51	0.44
Percent of class in area	45	50	5
Natural moisture content, percent	19.6	22.7	27.6

Added by Amendment 50

TABLE 2.5-52

WATTS BAR NUCLEAR PLANT

ERCW LIQUEFACTION

BORROW AREA 2C

SUMMARY OF LABORATORY TEST DATA

BORROW SOIL CLASSES

Class	I	II	III	IV	V	VI
Symbol	ML	SM-SC	CL	CL	CL-ML	MH
Mechanical and Hydrometer Analysis						
Gravel, percent	0	0	0	0	0	0
Sand, percent	48	65	48	30	23	5
Silt, percent	40	16	23	34	39	40
Clay, percent	12	19	29	36	38	55
Atterberg Limits						
Liquid limit, percent	NP	25	36	41	44	62
Plastic limit, percent	NP	19	22	24	27	35
Plasticity index, percent	NP	6	14	17	17	27
Shrinkage limit, percent	--	--	--	--	--	--
Standard Proctor Compaction						
Optimum moisture, percent	12.1	13.9	16.6	18.1	19.5	26.8
Maximum density, pcf	117.7	114.0	109.0	106.2	103.5	90.8
Penetration resistance, psi	1000	1125	1050	760	840	950
Shear Strength at 3% Dry of Optimum Moisture and at 95% of Maximum Unit Weight*						
Triaxial R: ϕ, degrees	17.5	**	13.4	9.0	18.1	19.0
c, tsf	0.63	**	0.11	0.33	0.00	0.00
Shear Strength at 3% Dry of Optimum Moisture and at 100% of Maximum Unit Weight***						
Triaxial R: ϕ, degrees	--	--	13.0	--	15.3	17.4
c, tsf	--	--	0.58	--	0.22	0.24
Percent of class in area	1	<1	3	31	63	1
Natural moisture content, percent	21.7	20.5	26.4	22.9	23.6	31.6

*Class VI tested at 90.0 pcf.
**Class II is less than 1% of total borrow and no shear tests were conducted on this class
***Class VI tested at 105% of maximum unit weight.

TABL[illegible]-53

WATTS BAR NUCLEAR PLANT

ERCW LIQUEFACTION

BORROW AREA 2C EXTENSION

SUMMARY OF LABORATORY TEST DATA

BORROW SOIL GROUPS

Group	1	2	3	4	5	6	7	8	9
Symbol	CL	CL	CL	CL	CL-ML	MH	CL	CL-ML	SM
Mechanical and Hydrometer Analysis									
Gravel, percent	0	0	0	0	0	0	0	0	1
Sand, percent	23	30	24	20	23	15	36	42	55
Silt, percent	48	42	43	40	36	27	36	33	30
Clay, percent	29	28	33	40	41	58	28	25	14
Atterberg Limits									
Liquid limit, percent	34	34	40	41	47	58	37	35	21
Plastic limit, percent	21	22	24	24	28	32	23	23	20
Plasticity index, percent	13	12	16	17	19	26	14	12	1
Shrinkage limit, percent	--	--	--	--	--	--	--	--	--
Standard Proctor Compaction									
Optimum moisture, percent*	16.6	17.3	18.8	20.2	21.7	28.1	16.6	16.6	14.8
Maximum density, pcf	109.0	107.7	104.8	102.3	99.6	88.0	109.0	109.0	112.8
Penetration resistance, psi	--	--	--	--	--	--	--	--	--
Percent of group in area	3	24	11	13	11	3	33	3	1
Natural moisture content, percent	21.5	16.4	21.5	21.8	26.5	27.2	14.0	16.6	10.1

*Standard proctor compaction results are based on borrow area 2C family of curves.

Note: Shear strength tests were not conducted on the extension of borrow area 2C.

Added by Amendment 50

TABLE 2.5-55

GRANULAR MATERIAL DESIGN VALUES
SECTION 1032 MATERIAL

Relative Density	Unit Weight γ_m (pcf)	Unit Weight γ_{sat} (pcf)	Shear Strength Values Q ϕ	Q C (tsf)	$\overline{R}$&S* ϕ	$\overline{R}$&S* C (tsf)
80%	135	143	39°	1.0	40°	0.5
70%	133	142	39°	0.7	38°	0.35

*For an analysis where pore pressure buildup has to be considered, estimated pore pressure should be incremented (suggest 10% increments) to a reasonable maximum level to check the effect of pore pressure buildup.

γ_m = Moist unit weight

γ_{sat} = Saturated unit weight

Q = Unconsolidated - undrained triaxial shear test

$\overline{R}$ = Consolidated - undrained triaxial shear test (effective)

S = Direct shear test

Added by Amendment 50

TABLE 2.5-56

Relative Density Test Results on Engineered Granular Fill Beneath the Diesel Generator Building

Sample	Max. Dry Density (pcf)	Min. Dry Density (pcf)	Field Density (pcf)	Relative Density (%)
158	144.6	100.4	132.0	78
159	144.6	100.4	133.0	80
160	144.6	100.4	135.0	84
162	144.6	100.4	137.75	82
163	144.6	100.4	136.5	87
164	144.6	100.4	131.25	77
167	144.6	100.4	135.5	85
168	144.6	100.4	138.0	89
169	144.6	100.4	135.75	85
170	144.6	100.4	131.5	77
171	144.6	100.4	136.75	87
172	144.6	100.4	133.25	81
178	144.6	100.4	130.25	75
179	144.6	100.4	131.5	77
180	144.6	100.4	131.0	76
184	144.6	100.4	130.75	78
185	144.6	100.4	137.5	88
186	144.6	100.4	130.5	76
190	144.6	100.4	138.5	90
191	144.6	100.4	136.25	86
192	144.6	100.4	134.75	83
194	144.6	100.4	128.75	72
195	144.6	100.4	132.0	78
196	144.6	100.4	131.5	77
199	144.6	100.4	129.5	76
200	144.6	100.4	137.25	88
201	144.6	100.4	130.75	77
204	144.6	100.4	125.75	66
205	144.6	100.4	127.75	70
206	144.6	100.4	127.75	70
210	144.6	100.4	128.25	71
211	144.6	100.4	137.0	87
212	138.8	109.9	133.5	83
213	138.8	109.9	137.0	96
214	138.8	109.9	136.5	93
217	138.8	109.9	133.75	86.5
218	138.8	109.9	136.5	94.5

Added by Amendment 50

TABLE 2.5-57

PROJECT: *Watts Bar Nuclear Plant*

SIEVE ANALYSIS OF 1032 GRAVEL

TENNESSEE VALLEY AUTHORITY

Sheet 1

SCREEN SIZE				PERCENT PASSING									
				1½"	1"	3/4"	3/8"	#4	#10	#16	#40	#100	#200
SPECIFICATION LIMITS 1032.02				100	95-100	70-100	50-85	36-65	20-45	NA	8-25	NA	0-10
DATE	TIME	SAMPLE SOURCE	SAMPLE WT(lbs)							NA		NA	
3-21-75	12:10 P.	TVA STOCKPILE	16.7	100.0	100.0	91.1	66.6	44.5	26.2	NA	10.4	NA	4.1
3-25-75	1:00 P.	TVA STOCKPILE	15.0	100.0	100.0	91.1	76.7	54.2	32.1	NA	10.2	NA	3.9
3-26-75	11:15 a	TVA STOCKPILE	16.7	100.0	97.2	87.2	62.4	43.7	26.8	NA	5.7	NA	Left off By Mistake
3-27-75	2:30 p	TVA STOCKPILE	16.7	100.0	93.6	85.8	65.3	47.1	24.4	NA	11.2	NA	4.8
3-28-75	9:15 A	TVA STOCKPILE	16.6	100.0	100.0	93.0	79.3	58.2	38.0	NA	14.4	NA	4.4
3-31-75	9:40 A	TVA STOCKPILE	16.7	100.0	100.0	92.9	66.6	42.2	23.9	NA	9.5	NA	4.2
4-1-75	10:30 A	TVA STOCKPILE	15.0	100.0	96.2	93.7	78.4	56.9	34.1	NA	13.4	NA	5.1
4-2-75	12:00 p	TVA STOCKPILE	16.4	100.0	100.0	91.6	64.6	42.6	24.5	NA	9.0	NA	3.9
4-3-75	10:46 A	TVA STOCKPILE	16.3	100.0	98.1	90.1	74.3	55.8	31.6	NA	5.3	NA	1.6
4-4-75	9:30 A	TVA STOCKPILE	17.0	100.0	97.8	92.1	77.2	59.6	38.7	NA	13.5	NA	4.2
4-7-75	12:30 p	TVA STOCKPILE	16.7	100.0	98.7	95.6	77.2	57.6	34.3	NA	10.3	NA	4.4
4-8-75	12:15 p	TVA STOCKPILE	16.7	100.0	95.7	88.3	61.9	39.9	25.1	NA	10.1	NA	3.4
4-9-75	12:10 p	TVA STOCKPILE	16.7	100.0	100.0	95.8	77.5	53.2	32.1	NA	11.7	NA	3.4
4-10-75	1:00 p	TVA STOCKPILE	16.7	100.0	100.0	87.2	49.9	32.2	20.0	NA	8.9	NA	3.9

Added by Amendment 50

TABLE 2.5-57 (Continued)

PROJECT: WATTS BAR NUCLEAR PLANT

SIEVE ANALYSIS OF 1032 GRAVEL

TENNESSEE VALLEY AUTHORITY

Sheet 2

SCREEN SIZE				PERCENT PASSING									
				1¼"	1"	3/4"	3/8"	#4	#10	#16	#40	#100	#200
SPECIFICATION LIMITS 1032.02				100	95-100	70-100	50-85	36-65	20-45	NA	8-25	NA	0-10
DATE	TIME	SAMPLE SOURCE	SAMPLE WT(lbs)							NA		NA	
4-11-75	10:00A	TVA STOCKPILE	20.0	100.0	98.6	90.5	63.6	44.0	26.5	NA	8.8	NA	3.0
4-14-75	10:00A	TVA STOCKPILE	20.0	100.0	98.5	91.6	64.0	40.8	33.5	NA	8.6	NA	4.8
4-15-75	10:15A	TVA STOCKPILE	20.0	100.0	100.0	91.2	67.7	41.3	23.4	NA	8.1	NA	2.7
4-16-75	12:05A	TVA STOCKPILE	16.7	100.0	100.0	88.9	63.9	43.1	26.4	NA	9.7	NA	3.8
4-17-75	2:00P	TVA STOCKPILE	16.5	100.0	100.0	91.7	64.6	42.8	24.5	NA	9.2	NA	4.0
4-18-75	1:35P	TVA STOCKPILE	16.8	100.0	99.0	94.2	73.7	54.4	33.9	NA	11.5	NA	4.8
4-21-75	3:30P	TVA STOCKPILE	16.7	100.0	100.0	91.3	54.5	33.4	19.8	NA	8.5	NA	3.6
4-22-75	3:30P	TVA STOCKPILE	16.6	100.0	100.0	87.6	55.0	34.1	19.3	NA	7.9	NA	3.4
4-23-75	12:05	TVA STOCKPILE	20.0	100.0	99.0	92.9	64.5	25.7	19.3	NA	10.1	NA	2.1
4-24-75	10:00A	Diesel Generator Bldgs	20.0	100.0	100.0	95.7	81.2	59.1	32.9	NA	9.5	NA	4.2
4-25-75	10:50A	TVA STOCKPILE	16.7	100.0	97.1	86.3	56.9	37.8	24.1	NA	9.1	NA	3.8
4-28-75	9:30A	TVA STOCKPILE	16.8	100.0	100.0	95.7	81.0	60.1	38.8	NA	14.5	NA	4.6
4-29-75	12:30P	TVA STOCKPILE	16.7	100.0	100.0	90.2	72.6	54.7	35.3	NA	12.8	NA	3.9
4-30-75	1:00P	TVA STOCKPILE	16.9	100.0	100.0	95.3	72.2	49.8	30.3	NA	11.1	NA	3.5

Added by Amendment 50

TABLE [illegible]7 (Continued)

PROJECT: Watts Bar Nuclear Plant

SIEVE ANALYSIS OF 1032 GRAVEL

TENNESSEE VALLEY AUTHORITY

Sheet 3

SCREEN SIZE				PERCENT PASSING									
				1¼"	1"	3/4"	3/8"	#4	#10	#16	#40	#100	#200
SPECIFICATION LIMITS 1032.02				100	95-100	70-100	50-85	36-65	20-45	NA	8-25	NA	0-10
DATE	TIME	SAMPLE SOURCE	SAMPLE WT(lbs)							NA		NA	
5-1-75	10:00 A.	TVA Stockpile	15.0	100.0	100.0	97.2	73.9	48.3	31.1	NA	16.3	NA	8.9
5-2-75	10:00 A.	TVA Stockpile	16.7	100.0	100.0	98.7	77.9	56.3	35.9	NA	13.9	NA	4.5
5-5-75	1:00 P.	TVA Stockpile	16.8	100.0	100.0	94.1	81.0	63.4	42.8	NA	17.2	NA	5.6
5-6-75	9:00 A.	TVA Stockpile	17.0	100.0	100.0	97.5	78.5	57.0	35.7	NA	11.6	NA	3.8
5-7-75	8:30 A.	TVA Stockpile	16.9	100.0	98.9	91.6	64.4	44.3	27.1	NA	10.3	NA	3.6
5-8-75	12:45 P.	TVA Stockpile	16.8	100.0	100.0	92.2	74.8	53.9	34.3	NA	12.3	NA	3.9
5-9-75	13:01 P.	TVA Stockpile	16.8	100.0	100.0	98.2	74.5	51.5	31.7	NA	11.9	NA	3.7
5-12-75	10:00 A	TVA Stockpile	16.7	100.0	100.0	95.7	77.4	53.2	32.2	NA	11.7	NA	3.4
5-13-75	1:00 P	TVA Stockpile	16.7	100.0	98.1	91.9	75.4	55.5	35.9	NA	12.5	NA	4.0
5-14-75	9:45 A.	TVA Stockpile	16.9	100.0	100.0	89.3	65.7	43.1	28.1	NA	10.1	NA	4.3
5-15-75	9:45 A.	TVA Stockpile	17.0	100.0	100.0	93.2	73.4	52.4	32.8	NA	13.0	NA	4.5
5-16-75	10:30 A	TVA Stockpile	16.6	100.0	100.0	91.4	71.3	49.4	29.3	NA	10.1	NA	4.3
5-19-7	9:45 A.	TVA Stockpile	16.7	100.0	100.0	97.7	76.1	59.9	39.1	NA	10.2	NA	3.5
5-20-7	1:30 P	TVA Stockpile	16.1	100.0	100.0	86.9	52.2	30.0	15.0	NA	6.6	NA	8.3

Added by Amendment 50

TABLE 2.5-5 (Continued)

PROJECT: *Watts Bar Nuclear Plant*

SIEVE ANALYSIS OF 1032 GRAVEL

TENNESSEE VALLEY AUTHORITY

Sheet 4

				PERCENT PASSING									
SCREEN SIZE				1¼"	1"	3/4"	3/8"	#4	#10	#16	#40	#100	#200
SPECIFICATION LIMITS 1032.02				100	95-100	70-100	50-85	36-65	20-45	NA	8-25	NA	0-10
DATE	TIME	SAMPLE SOURCE	SAMPLE WT(lbs)							NA		NA	
5-21-75	9:30 A	TVA Stockpile	16.8	100	100	95.6	80.9	60.0	38.4	NA	14.5	NA	4.6
5-22-75	10:00 A	TVA Stockpile	16.7	100	100	87.3	49.9	32.2	19.9	NA	8.9	NA	3.9
5-23-75	9:30 A	TVA Stockpile	16.6	100	100	91.7	71.4	49.4	29.1	NA	10.2	NA	4.5
5-27-75	9:30 A	TVA Stockpile	16.7	100	100	87.2	49.8	32.3	20.0	NA	8.9	NA	3.8
5-28-75	9:30 A	TVA Stockpile	16.8	100	100	96.3	72.6	54.8	35.3	NA	12.8	NA	3.8
5-29-75	9:30 A	TVA Stockpile	16.7	100	98.6	86.4	64.4	43.3	24.3	NA	8.6	NA	3.5
5-30-75	9:30 A	TVA Stockpile	16.3	100	97.3	95.4	73.4	56.0	36.8	NA	13.7	NA	4.5
6-2-75	9:15 P	TVA Stockpile	16.7	100	100	87.2	49.8	32.2	20.0	NA	8.9	NA	3.8
6-3-75	1:00 P	TVA Stockpile	15.4	100	100	81.3	54.0	30.2	20.2	NA	14.5	NA	4.8
6-4-75	10:30 A	TVA Stockpile	16.6	100	100	91.7	66.9	43.7	24.5	NA	9.5	NA	3.2
6-5-75	5:00 P	TVA Stockpile	16.8	100	98.1	91.2	73.3	53.1	29.8	NA	11.2	NA	4.5
6-6-75	9:30 A	TVA Stockpile	15.3	100	97.3	95.2	73.3	55.9	36.5	NA	13.5	NA	3.7
6-9-75	11:00 A	TVA Stockpile	16.6	100	100	95.6	77.7	58.0	33.8	NA	10.5	NA	3.5
6-10-75	10:00 A	TVA Stockpile	16.5	100	100	91.5	71.1	49.1	29.0	NA	9.0	NA	2.8

Added by Amendment 50

TABLE 2.5-57 (Continued)

PROJECT: *Watts Bar Nuclear Plant*

SIEVE ANALYSIS OF 1032 GRAVEL

TENNESSEE VALLEY AUTHORITY

Sheet 5

SCREEN SIZE				PERCENT PASSING									
				$1\frac{1}{4}$"	1"	3/4"	3/8"	#4	#10	#16	#40	#100	#200
SPECIFICATION LIMITS 1032.02				100	95-100	70-100	50-85	36-65	20-45	NA	8-25	NA	0-10
DATE	TIME	SAMPLE SOURCE	SAMPLE WT(lbs)							NA		NA	
6-12-75	7:30 A.	TVA Stockpile	16.6	100	99.5	95.8	77.5	57.6	34.4	NA	10.3	NA	3.5
6-13-75	9:30 A.	TVA Stockpile	16.6	100	100	97.7	82.2	63.5	41.3	NA	16.7	NA	5.9
6-16-75	9:30 A	TVA Stockpile	16.6	100	99.6	95.7	77.5	57.9	34.4	NA	10.3	NA	3.4
6-17-75	12:30 P	TVA Stockpile	16.7	100	97.2	91.9	77.4	57.4	35.6	NA	12.0	NA	2.5
6-18-75	9:30 A	TVA Stockpile	16.7	100	100	93.8	76.8	56.5	35.1	NA	11.6	NA	2.8
6-19-75	9:30 A	TVA Stockpile	16.7	100	100	*	67.1	48.6	29.8	NA	9.6	NA	3.4
6-20-75	1:00 P.	TVA Stockpile	15.1	100	100	99.9	75.6	51.5	26.9	NA	7.1	NA	1.5
6-23-75	9:30 A	TVA Stockpile	16.6	100	97.8	90.4	69.8	46.9	27.3	NA	11.3	NA	4.2
6-24-75	9:30 A	TVA Stockpile	16.7	100	99.8	93.3	64.1	38.0	22.4	NA	8.3	NA	3.2
6-25-75	9:30 A.	TVA Stockpile	15.6	100	100	95.2	77.0	58.2	33.6	NA	8.7	NA	1.6
6-26-75	9:30 P.	TVA Stockpile	16.7	100	99.6	95.5	77.9	57.8	34.6	NA	11.0	NA	4.2
6-27-75	10:00 A.	TVA Stockpile	15.8	100	100	92.0	56.1	37.1	22.9	NA	9.4	NA	3.3
6-30-75	9:30 A.	TVA Stockpile	16.7	100	100	89.4	67.4	48.6	29.9	NA	9.8	NA	3.1
7-1-75	12:30 P.	TVA Stockpile	16.2	100	99.2	90.1	75.6	57.8	31.2	NA	9.8	NA	1.7

*Omitted by mistake

Added by Amendment 50

TABLE [illegible]7 (Continued)

PROJECT: Watts Bar Nuclear Plant

SIEVE ANALYSIS OF 1032 GRAVEL

TENNESSEE VALLEY AUTHORITY

Sheet 6

SCREEN SIZE				PERCENT PASSING									
				$1\frac{1}{4}$"	1"	3/4"	3/8"	#4	#10	#16	#40	#100	#200
SPECIFICATION LIMITS 1032.02				100	95-100	70-100	50-85	36-65	20-45	NA	8-25	NA	0-10
DATE	TIME	SAMPLE SOURCE	SAMPLE WT(lbs)							NA		NA	
7-2-75	10:30 A	TVA Stockpile	16.5	100	100	91.8	64.7	42.7	24.5	NA	9.0	NA	4.0
7-3-75	9:30 A	TVA Stockpile	16.5	100	97.4	94.7	73.1	52.5	30.7	NA	8.3	NA	2.5
7-9-75	5:30 P	TVA Stockpile	16.7	100	100	93.2	68.3	46.7	27.5	NA	8.7	NA	3.1
7-10-75	10:00 A	TVA Stockpile	16.8	100	97.7	89.7	73.3	53.7	34.3	NA	12.9	NA	3.6
7-11-75	9:30 A	TVA Stockpile	15.7	100	100	89.2	61.7	41.9	23.9	NA	7.0	NA	2.6
7-14-75	9:30 A	TVA Stockpile	15.8	100	100	91.8	56.4	37.6	22.7	NA	9.5	NA	3.5
7-15-75	2:00 P	TVA Stockpile	16.7	100	100	97.6	75.9	54.8	34.2	NA	10.2	NA	3.4
7-16-75	1:00 P	TVA Stockpile	15.8	100	100	96.5	79.6	58.4	34.0	NA	9.9	NA	0.0
7-17-75	2:00 P	TVA Stockpile	16.9	100	100	98.0	77.0	57.7	35.0	NA	9.2	NA	1.6
7-18-75	2:00 P	TVA Stockpile	15.2	100	100	99.4	76.0	51.4	27.2	NA	7.5	NA	0.0
7-21-75	1:00 P	TVA Stockpile	16.8	100	100	96.1	75.5	52.8	31.3	NA	10.1	NA	3.2
7-22-75	1:30 P	TVA Stockpile	16.5	100	100	97.5	77.0	51.9	28.7	NA	9.7	NA	4.0
7-23-75		TVA Stockpile	16.8	100	100	95.6	80.4	64.8	43.3	NA	16.6	NA	6.5
7-24-75	10:30 P	TVA Stockpile	16.7	100	100	97.5	64.9	40.2	21.7	NA	6.8	NA	1.2

Added by Amendment 50

TABLE 57 (Continued)

PROJECT: WATTS BAR NUCLEAR PLANT

SIEVE ANALYSIS OF 1032 GRAVEL

TENNESSEE VALLEY AUTHORITY

Sheet 7

SCREEN SIZE				PERCENT PASSING									
				1¼"	1"	3/4"	3/8"	#4	#10	#16	#40	#100	#200
SPECIFICATION LIMITS 1032.02				100	95-100	70-100	50-85	36-65	20-45	NA	8-25	NA	0-10
DATE	TIME	SAMPLE SOURCE	SAMPLE WT(lbs)							NA		NA	
7-25-75		TVA Stockpile	16.8	100	100	92.2	74.8	53.9	34.3	NA	12.3	NA	3.9
7-28-75	9:30A	TVA Stockpile	16.9	100	100	95.3	63.4	46.5	28.6	NA	8.6	NA	2.4
7-29-75	9:10A	TVA Stockpile	16.2	100	99.1	90.2	75.3	57.8	31.1	NA	4.9	NA	1.1
7-30-75	10:30A	TVA StockPile	16.8	100	100	85.6	68.8	43.2	26.1	NA	8.9	NA	3.6
										NA		NA	
										NA		NA	
										NA		NA	
										NA		NA	
										NA		NA	
										NA		NA	
										NA		NA	
										NA		NA	
										NA		NA	
										NA		NA	

Added by Amendment 50

TABLE 2.5-62

SUMMARY OF SPT SAMPLES OF SILTY SANDS (SM) BELOW ERCW PIPELINES HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION

Boring No.	Elev. (ft)	SPT Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-49	700.9	13	SM-SC	28.3	6.5	25.1	0.074	49.0	
SS-49A	700.7	5	SM	NP	NP	26.5	0.110	31.0	same sample
	700.7	5	SM	23.0	1.0	29.0	0.990	42.0	
	698.7	6	SM	23.0	1.0	29.9	0.990	41.0	
	696.7	5	SM	NP	NP	31.8	0.120	29.0	same sample
	696.7	5	SM	29.0	4.0	32.4	0.080	47.0	
	692.7	5	SM	23.0	1.0	28.7	0.080	47.0	
	690.7	6	SM	NP	NP	30.0	0.120	31.0	
	688.7	17	SM	NP	NP	31.2	0.120	38.0	same sample
	688.7	17	SM	NP	NP	21.2	0.650	19.4	
SS-131	699.9	4	SM	30.8	6.9	28.1	.080	48.0	
	697.9	5	SM	25.9	3.3	30.1	0.080	45.0	
	695.9	5	SM	25.9	3.3	29.7	0.080	45.0	
	693.9	7	SM	NP	NP	26.2	0.085	45.0	
	691.9	7	SM	NP	NP	24.0	0.085	45.0	
SS-50A	702.2	14	SM	NP	NP	25.5	0.010	35.0	
	700.2	11	SM	27.0	2.0	28.8	0.100	37.0	same sample
	700.2	11	SM	NP	NP	26.9	0.173	22.0	
	698.2	13	SM	26.0	2.0	27.4	0.100	38.0	same sample
	698.2	13	SM	NP	NP	28.8	0.120	29.0	
	696.2	9	SM	NP	NP	33.5	0.130	26.0	same sample
	696.2	9	SM	NP	NP	33.5	0.120	26.0	
	694.2	5	SM	NP	NP	38.4	0.090	39.0	

Added by Amendment 50

TABLE 2.5-62 (Continued)

SUMMARY OF SPT SAMPLES OF SILTY SANDS (SM) BELOW ERCW PIPELINES HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION (continued)

Boring No.	Elev. (ft)	SPT Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-50	701.8	10	SM	34.1	7.6	22.4	0.084	47.0	
	697.8	5	SM	NP	NP	28.2	0.098	43.0	
	695.8	8	SM	NP	NP	29.1	0.093	43.0	
	693.8	2	SM	NP	NP	31.5	0.087	47.0	
	691.8	10	G-SM	NP	NP	23.7	0.190	33.9	
SS-133	704.0	19	G-SM	NP	NP	17.3	0.250	29.0	
SS-134	710.5	3	SM	NP	NP	29.3	0.148	26.0	
	708.5	8	SM	NP	NP	27.5	0.141	31.0	
SS-134A	709.5	4	SM	23.0	1.0	30.0	0.105	35.0	same sample
	709.5	4	SM	NP	NP	29.1	0.110	30.0	same sample
	707.5	9	SM	24.0	2.0	27.9	0.100	27.0	same sample
	707.5	9	SM	24.0	1.0	28.9	0.090	43.0	same sample
SS-135	712.0	11	SM	34.1	8.7	23.6	-	-	
	710.9	12	SM	30.0	4.4	20.1	-	-	
	708.9	8	SM	NP	NP	-	-	-	
	706.9	8	SM	NP	NP	-	-	-	
	704.9	8	SM	NP	NP	25.3	-	-	
SS-135A	714.5	13	SM	31.0	3.0	24.3	0.078	48.0	
	712.5	7	SM	NP	NP	22.8	0.105	33.0	
	710.5	7	SM	NP	NP	24.3	0.120	29.0	
	708.5	5	SM	NP	NP	34.2	0.120	29.0	
	706.5	8	SM	22.0	1.0	27.0	0.120	33.0	
	704.5	7	SM	NP	NP	30.9	0.100	35.0	

Added by Amendment 50

TABLE 2.5-62 (Continued)

SUMMARY OF SPT SAMPLES OF SILTY SANDS (SM) BELOW ERCW PIPELINES HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION (continued)

Boring No.	Elev. (ft)	SPT Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-65B	713.2	9	SM	29.0	2.0	25.7	0.085	43.0	
	711.2	6	SM	25.0	1.0	27.5	0.090	41.0	
	709.2	3	SM	25.0	1.0	33.1	0.100	38.0	same sample
	709.2	3	SM	NP	NP	32.9	0.110	31.0	
	707.2	5	SM	25.0	1.0	32.5	0.100	34.0	
	705.2	7	SM	26.0	2.0	27.1	0.075	50.0	same sample
	705.2	7	SM	25.0	1.0	30.8	0.100	35.0	
SS-65	712.0	12	SM	33.1	6.6	21.5	0.077	48.0	
	710.0	10	SM	NP	NP	15.7	0.132	32.5	
	708.0	7	SM	30.1	5.1	23.7	0.091	43.0	
	706.0	5	SM	28.9	3.5	28.2	0.140	34.0	
	704.0	8	-	-	-	-	-	-	no sample
SS-136	710.9	5	SM	NP	NP	26.3	0.100	40.0	
	708.9	8	SM	NP	NP	28.5	0.122	35.0	
	706.9	12	SM	NP	NP	21.9	0.145	33.0	
SS-137	712.9	9	SM	25.9	1.8	20.7	-	-	
SS-138	713.2	6	SM	28.1	2.5	23.4	0.079	49.0	
	711.2	7	SM	28.1	2.5	24.5	0.079	49.0	
	705.2	13	SM	26.4	2.3	15.0	-	-	
SS-138A	713.2	8	SM	29.0	3.0	25.1	0.073	50.0	
	711.2	8	SM	NP	NP	22.1	0.100	36.0	
	709.2	12	SM	29.0	1.0	27.1	0.073	49.0	
	707.2	4	SM	28.0	2.0	35.6	0.090	41.0	
	705.2	9	SM	22.0	1.0	27.8	0.140	31.0	same sample
	705.2	9	SM	NP	NP	29.1	0.180	21.0	

Added by Amendment 50

TABLE 2.5-62 (Continued)

SUMMARY OF SPT SAMPLES OF SILTY SANDS (SM) BELOW ERCW PIPELINES HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION (continued)

Boring No.	Elev. (ft)	SPT Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-138B	710.6	8	SM	27.0	3.0	24.7	0.090	42.0	
	708.6	9	SM	34.0	5.0	36.2	0.080	46.0	
	706.6	8	SM-SC	27.0	5.0	30.0	0.105	35.0	
	704.6	7	SM-SC	26.0	5.0	32.5	0.110	35.0	
SS-138C	710.6	8	SM-SC	27.0	4.0	27.5	0.095	38.0	
SS-139	711.5	8	SM	NP	NP	15.5	0.110	35.0	
	709.5	9	SM	NP	NP	18.2	0.110	35.0	
	705.5	14	SM	NP	NP	22.1	0.375	13.0	
SS-140	706.7	4	SM	NP	NP	38.7	0.110	36.0	
SS-87	707.6	12	SM	31.6	6.2	27.5	0.078	48.0	
SS-141	704.6	17	G-SM	NP	NP	7.8	0.79	19.0	
SS-143	695.1	7	–	–	–	–	–	–	no sample
	693.1	9	G-SP-SM	NP	NP	13.5	1.80	12.0	
SS-143A	701.0	3	SM-SC	21.0	5.0	21.2	0.093	45.0	
	697.0	8	SM	37.0	11.0	43.1	0.130	41.0	
SS-143B	696.3	21	SM	37.0	7.0	27.7	0.300	34.0	
SS-146	702.4	13	G-SM	21.6	1.9	14.6	0.200	25.0	
SS-147	701.7	18	G-SM	NP	NP	17.1	0.460	14.0	
SS-153	707.7	15	G-SW-SM	NP	NP	10.8	2.500	10.0	

Added by Amendment 50

TABLE 2.5-62 (Continued)

SUMMARY OF SPT SAMPLES OF SILTY SANDS (SM) BELOW ERCW PIPELINES HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION (continued)

Boring No.	Elev. (ft)	SPT Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-158	711.5	2	SM	22.9	2.5	32.2	0.088	44.0	
SS-159	712.0	20	G-SM	NP	NP	13.7	0.430	21.0	
SS-160	720.9	15	SM	NP	NP	22.5	0.134	39.0	
	718.9	7	SM	24.2	1.7	23.8	0.173	34.0	
	716.9	12	SM	27.0	3.0	25.8	0.153	33.0	
	714.9	5	SM-SC	32.1	8.5	30.2	0.105	46.0	
	710.9	5	GM	26.2	2.2	24.3	0.210	37.0	
SS-161A	720.9	10	SM	26.0	2.0	23.8	0.120	32.0	
	718.9	13	SM	NP	NP	17.8	0.230	17.0	same sample
	718.9	13	SM	NP	NP	17.0	0.180	20.0	same sample
SS-161	718.4	9	SM	NP	NP	18.4	0.230	24.0	
	716.4	10	SM	NP	NP	21.5	0.220	24.0	
	708.4	19	G-SM	NP	NP	12.7	0.220	32.0	
SS-162	717.8	20	SM	28.3	1.6	27.7	0.090	47.0	
	715.8	19	SM	27.6	3.0	30.2	0.122	39.0	
	713.8	5	SM	NP	NP	34.3	0.115	36.0	
	711.8	11	G-SW-SM	NP	NP	20.4	2.000	11.0	
SS-163	721.0	5	SM-SC	30.4	7.1	28.4	0.084	47.0	
	719.0	6	SM-SC	30.4	7.1	26.9	0.084	47.0	
	717.0	3	SM	27.2	3.3	31.1	0.097	45.0	
	715.0	4	SM	29.7	4.7	33.5	0.090	43.0	
	713.0	17	G-SM	28.7	3.8	27.3	0.190	26.0	

Added by Amendment 50

TABLE 2.5-62 (Continued)

SUMMARY OF SPT SAMPLES OF SILTY SANDS (SM) BELOW ERCW PIPELINES HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION (continued)

Boring No.	Elev. (ft)	SPT Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-163A	721.5	7	SM	31.0	7.0	28.9	0.080	48.0	
	719.5	11	SP-SM	NP	NP	28.2	0.220	8.0	
	717.5	4	SM	30.0	3.0	36.3	0.080	45.0	
	715.5	5	SM	31.0	5.0	34.3	0.098	43.0	
SS-80	721.2	3	SM	41.6	14.6	29.1	0.120	44.0	
	715.2	7	SM	24.5	0.7	25.4	0.161	29.0	
SS-164	719.0	9	SM-SC	31.5	8.6	27.4	0.240	33.0	
	717.0	15	G-SP-SM	NP	NP	16.2	0.750	12.0	
	715.0	20	G-SP-SM	NP	NP	20.9	0.340	10.0	
	713.0	11	SM	31.1	5.7	26.6	0.174	33.0	
SS-165	716.7	3	SM-SC	30.7	8.1	23.3	--	--	
	714.7	2	SM-SC	30.7	8.1	34.4	--	--	
SS-84	713.4	2	SM	24.8	2.2	30.1	0.110	41.0	
SS-130	715.7	10	SM	NP	NP	17.8	0.240	22.0	
	713.7	9	SM	NP	NP	15.5	0.290	15.0	
SS-128	712.1	2	SM	NP	NP	23.7	0.280	16.0	
SS-127	712.2	0	SM-SC	23.3	4.4	36.1	0.079	48.0	
SS-125	714.4	2	SM	NP	NP	29.0	0.130	8.0	
	708.4	16	G-SP-SM	NP	NP	21.7	0.660	8.0	
	706.4	17	G-SP-SM	NP	NP	12.8	3.00	10.0	
SS-25	715.6	2	SM	NP	NP	29.2	0.076	48.0	

Added by Amendment 50

TABLE 2.5-62 (Continued)

SUMMARY OF SPT SAMPLES OF SILTY SANDS (SM) BELOW ERCW PIPELINES HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION (concluded)

Boring No.	Elev. (ft)	SPT Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-28	713.4	10	SM	NP	NP	31.0	0.18	27.5	
SS-170	719.2	4	G-SM-SC	34.8	11.5	29.1	0.125	42.0	
	717.2	17	G-SM-SC	34.8	11.5	23.6	0.125	42.0	
	715.2	18	G-SM-SC	NP	NP	19.2	0.450	11.0	

Added by Amendment 50

TABLE 2.5-63

SUMMARY OF SPT SAMPLES OF SILTS (ML) BELOW ERCW PIPELINES HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION

Boring No.	Elev. (ft)	SPT Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-49	698.9	14	ML	28.8	5.3	26.1	0.070	53.0	
	696.9	12	ML	28.8	5.3	26.8	0.064	53.0	
SS-49A	694.7	6	ML	22.0	1.0	28.3	0.070	53.0	same sample
	694.7	6	ML	22.0	3.0	28.0	0.070	54.0	same sample
	692.7	5	ML	NP	NP	27.8	0.070	56.0	
SS-50A	694.2	5	ML	29.0	3.0	34.8	0.070	55.0	
SS-50	703.8	10	ML	37.5	11.3	22.1	0.050	54.0	
SS-132	702.1	13	ML	43.1	15.2	25.7	–	–	
	700.1	15	ML	45.8	17.5	23.4	–	–	
SS-135	714.9	12	ML	42.2	13.8	26.3	< 0.074	69.0	
SS-135A	706.5	8	ML	27.0	2.0	32.1	0.073	51.0	same sample
	706.5	8	ML	29.0	7.0	–	–	–	same sample
	704.5	7	ML	25.0	2.0	32.1	0.073	50.0	
SS-65B	715.2	14	ML	35.0	6.0	26.7	0.060	60.0	
SS-65	714.0	16	ML	46.1	15.6	29.2	0.030	72.0	
SS-136	712.9	9	ML	32.8	5.7	25.0	0.070	53.0	
SS-137	714.9	11	ML	35.6	9.6	24.2	0.058	62.0	
	710.9	7	ML	31.7	5.6	25.0	0.070	52.0	
	708.9	8	ML	31.7	5.6	25.3	0.070	52.0	

Added by Amendment 50

TABLE 2.5-63 (Continued)

SUMMARY OF SPT SAMPLES OF SILTS (ML) BELOW ERCW PIPELINES HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION (continued)

Boring No.	Elev. (ft)	SPT Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-138	709.2	7	ML	32.7	5.9	28.4	0.070	53.0	
	707.2	5	ML-CL	27.0	5.1	29.6	0.067	52.0	
SS-139	707.5	7	ML	31.0	3.9	32.8	0.056	63.0	
SS-140	710.7	12	ML	34.1	6.2	25.0	0.061	54.0	
	708.7	3	ML	–	–	17.4	0.073	50.0	
SS-87	711.6	13	ML	37.4	12.9	43.9	0.038	62.0	
SS-143C	696.6	3	CL-ML	32.0	10.0	46.5	< 0.074	72.0	
SS-101	712.5	3	ML	24.7	2.0	31.9	0.072	53.0	
SS-159	718.0	6	CL-ML	26.8	4.2	29.4	0.064	59.0	
SS-161A	714.9	5	ML	38.0	12.0	35.7	0.055	58.0	
SS-161	714.4	3	CL-ML	36.8	13.2	35.8	–	–	
	712.4	5	ML	25.7	2.3	30.9	0.076	51.0	
SS-80	719.2	5	ML	24.6	2.4	28.1	0.075	51.0	
SS-164	721.0	6	CL-ML	36.0	12.1	28.2	0.059	53.0	
SS-165	720.7	5	ML	37.4	11.5	31.9	0.060	58.0	
	718.7	6	CL-ML	39.0	14.2	31.2	0.015	63.0	

Added by Amendment 50

TABLE 2.5-63 (Continued)

SUMMARY OF SPT SAMPLES OF SILTS (HL) BELOW ERCW PIPELINES HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION (concluded)

Boring No.	Elev. (ft)	SPT Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-166	720.5	13	ML	48.8	19.8	13.0	0.011	87.0	
	718.5	11	ML	48.8	19.8	11.0	0.011	87.0	
	716.5	6	CL-ML	31.4	9.1	28.4	0.056	63.0	
SS-84	711.4	3	ML	24.5	1.3	31.4	0.070	52.0	
SS-130	717.7	7	ML	35.7	11.3	20.8	-	-	
SS-26	718.0	3	ML	24.4	0.6	29.7	0.051	61.0	
	716.0	4	ML	NP	NP	31.0	0.074	51.0	
SS-27	713.1	3	ML	23.1	2.9	24.5	0.072	51.0	
SS-169	119.1	8	CL-ML	43.0	17.0	31.8	0.021	78.0	
	117.1	6	ML	41.4	13.7	34.3	0.043	68.0	
	115.1	6	ML	41.4	13.7	32.3	0.043	68.0	
	113.1	5	ML	40.8	13.7	33.1	0.043	65.0	

TABLE 2.5-64

SUMMARY OF SPT SAMPLES OF SILTY SANDS (SM) BELOW ELECTRICAL CONDUITS HAVING FACTOR OF SAFETY LESS THAN UNITY FOR 0.4 G PEAK GROUND SURFACE ACCELERATION

Boring No.	Elev. (ft)	Blow Counts	Soil Type	Liquid Limit	Plasticity Index	Water Content (%)	D_{50} (mm)	Fines Content (%)	Remarks
SS-171	708.2	6	SM	NP	NP	26.7	0.20	13.0	
	706.2	9	SP-SM	NP	NP	26.5	0.26	7.0	
	704.2	9	SP-SM	NP	NP	24.1	0.27	9.0	
	702.2	12	SP-SM	NP	NP	30.9	0.27	8.0	
SS-53	708.0	18	SM	27.1	3.1	19.6	0.15	40.0	
SS-173	709.0	20	SM-SC	37.0	12.0	20.6	0.086	47.0	
SS-63	713.1	17	SM	36.0	10.0	21.6	0.078	48.0	
	711.1	10	SM	36.0	10.0	20.7	0.078	48.0	
	709.1	10	SM	36.0	10.0	27.0	0.078	48.0	
SS-57	715.0	14	SP-SM	NP	NP	6.4	0.75	9.0	

Added by Amendment 50

TABLE 2.5-63

STRAIN CRITERIA FOR DETERMINING

POTENTIAL SETTLEMENT OF SOILS SUBJECT

TO EARTHQUAKE WITH PEAK TOP-OF-GROUND

ACCELERATION OF 0.40g AT

WATTS BAR NUCLEAR PLANT

MATERIAL CLASSIFICATION	PERCENT VERTICAL STRAIN ($\%\epsilon_v$) BELOW WATER TABLE	ABOVE/BELOW WATER TABLE
SP (<12% fines)	6[1]	3[2]
SM or ML (clean)	3[1]	1.5[2]
SC	1[1]	0.5[2]
CL or ML-CL	0.75[1]	0.5[3]

1. If potentially liquefiable
2. If loose N≤15 but not potentially liquefiable
3. If soft N≤15 but not potentially liquefiable
4. Classification of SP-SM will be treated as SP for criteria
5. Classification of G-SM or SM-SC will be treated as SM for criteria

Added by Amendment 50