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Date	February 8, 1982		
Prepared	C.-Y. Chang		
Checked	Frank R Hand		
Reviewed	D.M. Adrian		
Submitted	R. Joe Hunt		
Recommended	Ruben O Hurley		
Approved	R.O. Barnett		

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PDR ADOCK 05000390
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1. INTRODUCTION

In response to the NRC's questions related to seismic stability of the foundation soil along the Essential Raw Cooling Water (ERCW) pipeline route at the Watts Bar Nuclear Plant Site, this study was conducted to evaluate the liquefaction potential of the foundation soils in accordance with well-established evaluation procedures. The evaluations were made for the Safe Shutdown Earthquake (SSE) with a peak acceleration of 0.18g at the ground surface. This report describes the field exploration, the results of the evaluation conducted to date (February, 1982), and ongoing supplemental studies.

1.1 Organization of Report

The following sections of the report are organized as follows: Section 2 describes the field exploration and subsurface soil conditions along the ERCW pipeline route; Section 3 describes the methodology used for the evaluation performed in this study; Section 4 presents the results of the evaluation; Section 5 describes the on-going studies. There are five appendices to the report: Appendix A describes the piezometers installed to monitor the ground water level along the pipeline route and the results of ground water observations to date (October 1981 to January 1982); Appendix B describes the design ground water level established along the pipeline route for use in the evaluation of liquefaction potential; Appendix C summarizes the additional borings drilled for this study; Appendix D describes the two test pits excavated for visual inspection of the subsurface soil stratifications by the TVA and NRC staff; and Appendix E describes an additional field exploration program using a cone penetrometer conducted at the site.

2.0 FIELD EXPLORATION AND SUBSURFACE SOIL CONDITIONS

2.1 Field Exploration

The original field exploration and laboratory testing for the ERCW piping system were conducted in 1975 to 1976. Supplementary field exploration and laboratory testing programs were completed in 1979. Locations of the borings are shown in Figure C-1 in Appendix C. At each of these borings (except borings to obtain undisturbed samples), standard penetration tests and split-spoon samples were obtained at approximately two-foot intervals. The field exploration and the standard penetration tests were conducted using hollow-stem auger drilling techniques. The drilling techniques used no drilling fluid during drilling and sampling. Boring logs showing the SPT blow counts and the results of laboratory testing including grain size distributions, soil classification, and Atterberg limits are presented in the FSAR Section 2.5 and they are summarized in Appendix C.

In 1981 eight piezometers were installed along the ERCW pipeline route for purposes of monitoring ground water level. The locations of these piezometers and the variation of the ground-water level observed are described in Appendix A.

Ten additional borings and SPT sampling were made in 1981 in the vicinity of ten selected old boring locations. These borings include No. SS-49A, -50A, -134A, -135A, -65B, -138A, -143A, -158A, -161A, and -163A. At the locations of Boring SS-138A and -143A, redrilling was made to examine variability of the SPT blow counts. These borings were identified as No. SS-138B, -138C, -143B and -143C. These additional borings were made using the rotary drilling technique with drilling mud. Standard penetration tests and split-spoon samples were obtained at these borings. The results of this additional field exploration and

laboratory testing are presented in Appendix C and summarized in Figure C-2. The SPT values obtained in the 1981 borings generally are higher than those obtained in borings drilled prior to 1981.

As part of the field exploration, two test pits were excavated to a depth of approximately 22 feet below ground surface. Visual inspection of the subsurface soil conditions was made by the NRC staff on December 15, 1981. A description of the test pits is presented in Appendix D.

In addition to the SPT split-spoon borings, field exploration using a cone penetrometer was performed to investigate subsurface soil conditions in the slope area adjacent to the ERCW pipeline route. The results of the cone penetrometer probing are being evaluated and will be available later. A brief description of the cone penetrometer testing program is presented in Appendix E.

2.2 Subsurface Soil Conditions

Along the ERCW pipeline route the soil conditions encountered in the borings consist of the following soil types in the order as they exist below the ground surface.

1. Silty clay fill
2. Terrace alluvial clays or silts
3. Terrace alluvial silty sands
4. Terrace alluvial gravels
5. Laminated residuum
6. Weathered shale
7. Shale bedrock

The soil profile along the ERCW pipeline route, as indicated by the borings, is depicted in Figure 17. It indicates that soil strata are very localized in many cases.

The thickness of the alluvial soils varies significantly along the ERCW pipeline route from a few feet at Borings SS-93 and -94 to approximately 35 feet at Boring SS-146 (see Figure 17). Underlying the alluvial soils are the laminated residuum and/or the weathered shale and the shale bedrock. At Boring SS-146 the alluvial soils consist predominantly of alluvial gravel, gravelly sand, and silty clay. An approximately five-foot thick layer of silty sand is present at Boring SS-146. The alluvial sand present along the ERCW pipeline route is generally classified as silty sand (SM). However, the silty sand contains very large fines content with the percent passing a No. 200 U.S. Sieve generally in the range of 30 to 50 percent. Most of the silty sand tested also shows some plasticity. Soil classification and SPT blow counts of each sample encountered in each boring are presented in Figure 17. A description of soil types and the results of laboratory tests are contained in boring logs retained in the project file.

3.0 METHODOLOGY OF EVALUATION

3.1 General Procedures for Evaluation of Liquefaction Potential

There are basically two methods available for evaluating the liquefaction potential of a deposit of saturated cohesionless soils subjected to earthquake shaking:

1. Using methods based on field observations of the performance of deposits in previous earthquakes and involving the use of some in situ characteristic of the deposits to determine probable similarities or dissimilarities between these sites and a proposed new site with regard to their potential behavior.

2. Using methods based on an evaluation of the cyclic stress or strain conditions likely to be developed in the field due to the design earthquake and a comparison of these stresses or strains with those observed to cause liquefaction of representative samples of the deposit in laboratory cyclic tests.

The two methods involve the same basic approach and differ only in the manner in which the field liquefaction characteristics of a deposit are determined.

In this study, both methods are used for evaluating the liquefaction potential of the alluvial soils present along the ERCW pipeline route.

3.2 Method Based on Empirical Correlations

3.2.1 Historical Development of Empirical Correlations - An evaluation of the liquefaction potential of a deposit of satu-

rated sand subjected to earthquake shaking can be made based on empirical correlations. These empirical correlations were developed from field observations of the performance of sand deposits in previous earthquakes and some in-situ characteristic of the deposits (Seed and Peacock, 1971; Seed, 1976 and 1979). Most of the previous empirical correlations were developed for deposits of clean sand using the standard penetration resistance (SPT). It is generally recognized, however, that SPT values are sensitive to grain size characteristics of a soil deposit. For example, SPT values for medium stiff fine grained silty or clayey deposits are much lower than those for medium dense sandy deposits. It might well be anticipated therefore that the SPT of a fine grained soil deposit will be much smaller than that of a sandy deposit even if the two deposits have an equal cyclic shear resistance.

Tatsuoka et al. (1978) conducted an extensive study to evaluate the effects of fines content on the cyclic shear resistance of silty sands. The study included cyclic triaxial tests on undisturbed samples from seven sites with various levels of fines content (percent passing 0.074 mm) and determination of the corresponding standard penetration values. The fines contents of the soils examined ranged from zero to 60 percent and values of D_{50} ranged from 1 mm to 0.04 mm. The results of the study showed that for fine sands for which D_{50} is less than 0.3 mm, the cyclic strength increases with an increase in fines content for the same SPT blow count. Similarly, for the same SPT blow count the cyclic strength increases as the value of D_{50} decreases from approximately 0.5 mm to 0.04mm.

Based on investigations of sites where liquefaction occurred during the Tangshan earthquake of 1976 and penetration resistance data (cone penetration data) in two districts having sand and silty sand deposits, Zhou (1981) concluded that correlations between penetration resistance and liquefaction characteristics

for clean sands are not applicable for silty sands unless they are modified to allow for the fines content of the silty sands. Zhou suggested that for soils with the same N-value, this allowance would take the form of an increase in penetration resistance, dependent on the fines content, if the same correlations for sands are used for silty sands. For soils with 30 percent fines, this increase in N- value would be about 6.

Seed and Idriss (1981) incorporated more recent field data for sites known to have liquefied and sites for which there was no apparent liquefaction in the correlation developed in 1975. These data include the following:

(a) Data fromm the Haicheng (1974) and Tangshan (1976) earthquakes in China (Magnitudes 7.3 and 7.8, respectively) by Xie (1979).

(b) Data from the Guatemala Earthquake of 1976 (Magnitude 7.6) by Seed et al. (1981).

(c) Data from the Argentina Earthquake of 1977 (Magnitude 7.4) by Idriss et al. (1980).

(d) Data from the Miyagiken-Oki Earthquake of 1978 (Magnitude 7.4) by Tokimatsu and Yoshimi (1981)

These recent data agreed well with the earlier data and the boundary line established in 1976 separating sites known to have liquefied from sites which apparently have not liquefied for sandy soils. The study by Tokimatsu and Yoshimi provides field data for both sands (D_{50} > 0.25 mm) and silty sand (D_{50} < 0.25 mm). The data for silty sands (D_{50} < 0.25 mm) were used by Seed and Idriss as a basis for developing a correlation between the N-value and the cyclic strength characteristic of silty sands. These correlations are consistent with the data of Tatsuoka et

al. (1978) and Zhou (1981) described earlier. The study by Seed and Idriss shows that the boundary line previously established for sands with $D_{50} > 0.25$ mm can be used for silty sands with $D_{50} < 0.15$ mm provided the normalized blow counts (N_1 values) for the silty sand site are increased by 7.5.

A recent study by the Corps of Engineers for a dam-site in Oklahoma (Seed, 1982) and data obtained by Hammond (1982) for a site in Northern California showed that silty sands and sandy silts with low N-values had resistances to cyclic loading that agreed well with results obtained using the correlations developed by Seed and Idriss (1981).

For silts (ML), no empirical correlations have yet been developed and thus the cyclic mobility of these soils can only be determined at the present time by cyclic tests. For a preliminary evaluation of the cyclic mobility of these soils, the correlations for silty sands proposed by Seed and Idriss (1981) can be used. However, because the approach is likely to be very conservative for these materials, the results are interpreted accordingly.

For clayey soils (silty clay, sandy clay, clayey silt or clayey sand) Seed and Idriss (1981) suggested the following criteria for evaluating liquefaction potential:

(a) If the clay content (percent finer than 0.005 mm) is greater than 20%, consider the soil non-liquefiable.

(b) If the water content of any clayey soil is smaller than 0.9 LL (Liquid Limit), consider the soil non-liquefiable.

(c) If the soils have the following characteristics:

Percent finer than 0.005 mm < 15%

Liquid Limit < 35

Water Content > 0.9 LL

and they plot above the A-line, determine their liquefaction characteristics by cyclic tests.

3.2.2 Procedure Used for Evaluation of Liquefaction Potential - The procedure used in this study follows that described in Seed (1979) and Seed and Idriss (1981). The procedure involves the following three essential steps:

1. Calculation of induced shear stress due to earthquake excitation;

2. Evaluation of cyclic strength characteristic of a soil deposit based on the standard penetration resistance and the empirical correlations; and

3. Comparisons of the induced shear stress with the cyclic strength of the soil deposit. Details of these steps are described below.

Step 1 - Calculation of Induced Shear Stress

The cyclic stress ratio induced in a soil deposit due to earthquake excitation is computed from the following equation:

$$\frac{(\tau_h)_{av}}{\sigma_o'} = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d \qquad (1)$$

where a_{max} = maximum acceleration at the ground surface

σ_o = total overburden pressure on sand layer under consideration

σ_o' = initial effective overburden pressure on sand layer under consideration

r_d = a stress factor varying from a value of unity at the ground surface to a value of 0.9 at a depth of about 30 feet as shown in Figure 1.

Step 2 - Evaluation of Cyclic Strength Characteristics

The cyclic strength of sands and silty sands was evaluated based on the standard penetration resistance (N-value) and the empirical correlations between the normalized N-value (N_1) and the cyclic stress ratio required to cause liquefaction. The N_1 value is the measured penetration resistance corrected to an effective overburden pressure of one ton/sq ft and can be determined from the following relationship:

$$N_1 = C_N \cdot N \qquad (2)$$

where C_N is a function of the effective overburden pressure at the depth where the penetration test was conducted. Values of C_N are determined from the curve shown in Figure 2. In this study, the curve for D_r = 40 to 60 percent was conservatively used.

The cyclic stress ratio causing liquefaction for the sand is estimated from the chart shown in Figure 3 with the value of N_1 calculated in Equation 2. For this study, the SSE corresponds to an earthquake with a magnitude equal to 5.8 (M = 5.8). Thus, the corresponding curve for M = 6 was used. For silty sands (D_{50} < 0.15 mm) the correlation established for silty sands by

Seed and Idriss (1981) was used. This correlation is equivalent to increasing the N_1 value of the silty sands by 7.5. The adjusted N_1 value is then used to enter the curve in Figure 3 to estimate the cyclic strength of the silty sands and silts. For silty sands with 0.25 mm > D_{50} > 0.15 mm the cyclic strength was interpolated from the strength for sands (D_{50} > 0.25 mm) and silty sands (D_{50} < 0.15 mm).

Step 3 - Comparison of Induced Shear Stress with Cyclic Strength

The induced shear stress ratio computed in Step 1 is compared with the cyclic stress ratio causing liquefaction evaluated in Step 2 for each SPT. The factor of safety is computed as the ratio of cyclic strength to induced stress.

It should be noted that the effect on the performance of the ERCW pipeline of a computed factor of safety being less than unity depends on the extent of the zone involved and its location with respect to the topographical features of the site. In a flat area, low factors of safety in zones of limited thickness and extent may result in high excess pore water pressures that, in turn, may cause only differential settlements in the foundation soils subsequent to dissipation of the excess pore water pressures. In an area adjacent to a slope, high excess pore pressures may cause a reduction in strength that would lead to landsliding or lateral spreading of the foundation soils. However, even this will not necessarily lead to adverse consequences. Any adverse effects depend on the extent of the loose sand layer and the topographical features at the location of concern.

The procedures described above were used for evaluating liquefaction potential of sands and silty sands present at the site. The evaluations were made by using the SPT values and the design

ground water level established for each boring location. The determination of the design ground water level is described in Appendix B.

3.3 Method Based on Cyclic Strength Determined by Laboratory Cyclic Testing

As described above, evaluation of the liquefaction potential of a sand deposit can also be made based on cyclic strength characteristic determined from laboratory cyclic tests on representative samples of the soil deposit involved. The method of evaluation involves the same basic approach as that based on empirical correlations described in Section 3.2 except that the liquefaction characteristic of the soil is determined from laboratory testing rather than from the SPT values.

As described in Section 5.1, a series of cyclic triaxial tests are being conducted on block samples of silty sands from two test pits. Four tests have been completed. The preliminary results of these four tests are summarized in Table 5 and Figure 4. Cyclic strength characteristics applicable to an in situ sand deposit can be obtained from the cyclic triaxial test results by the following equation:

$$\frac{\tau_f}{\tau_o'} = C_r \cdot \frac{\sigma_{dp}}{2\sigma_{3c}} \qquad (3)$$

where τ_f = cyclic shear stress required to cause a prescribed strain in the field.

$\sigma_{dp}/2\sigma_{3c}$ = pulsating deviator stress ratio measured in the laborary.

C_r = correction factor for cyclic triaxial tests.

The value of C_r applicable to normally consolidated soils is approximately equal to 0.6 (Seed et al., 1975). Based on the preliminary results shown in Figure 4, the cyclic shear stress ratio required to cause $\pm$ 2.5 percent strain in the field for 6 cycles (corresponding to M = 5.8 earthquake, Seed et al., 1975) is calculated to be 0.22. The cyclic shear stress ratio causing a prescribed strain is compared with the induced shear stress ratio calculated by equation (1) at depths of silty sand layers to compute factors of safety.

4.0 RESULTS OF EVALUATION

4.1 Introduction

Evaluation of the liquefaction potential for subsurface soils along the ERCW pipeline route was conducted by using the standard penetration test (SPT) blow counts and index properties of each sample obtained in each boring in accordance with the evaluation procedure described in Section 3.2. In addition, evaluations were also made based on the procedure described in Section 3.3 using the cyclic strength determined from laboratory testing. To illustrate the step-by-step evaluation procedure, analyses of three selected borings are described in Section 4.2. Results of the evaluation along the entire ERCW pipeline route are presented in Section 4.3. A summary of the results is presented in Section 4.4.

In the evaluation using the empirical correlations, the following general rules were followed. The corrected blow counts (N_1) were calculated based on the ground water level at the time of drilling and the elevation of the top of the boring. The induced cyclic shear stress ratios were calculated based on the design ground water level and the elevation of finished grade. The design ground water level for each boring was determined from the projected seasonal high ground water level contours given in Figure B-5. The basis for these contours is discussed in Appendix B. Table 1 lists the resulting design ground water level as well as the ground water level at the time of drilling and the elevation of finished grade for each boring.

4.2 Evaluations at Selected Boring Locations

In this section, SPT blow counts and index properties of samples obtained at three boring locations are analyzed. The three boring locations are the location of Borings SS-134, SS-134A,

Borings SS-138, SS-138A, and Borings SS-161 and SS-161A as shown in Figure C-1. As described in Section 2.1 and Appendix C, Borings SS-134, SS-138 and SS-161 were drilled in 1979 using hollow-stem auger drilling techniques without use of drilling fluid and Borings SS-134A, SS-138A and SS-161A were drilled in 1981 using rotary drilling techniques with drilling mud. The location of Borings SS-134A, SS-138A, and SS-161A are approximately 5 feet away from the locations of Borings SS-134, SS-138 and SS-161, respectively, in the direction of the pipeline route. See Figures C-5, C-8, and C-11 in Appendix C for exact locations.

4.2.1 Borings SS-134 and SS-134A - The soil profile, blow counts and index properties obtained at Borings SS-134 and SS-134A are depicted in Figure C-15. The ground surface elevations at Borings SS-134 and SS-134A are at El. 726.5 and 725.5 feet, respectively. The bottom of the pipeline is located at El. 717 feet at this location. The soil profile depicted in Figure C-15 consists of a 6.5 feet thick fill of silty clay (CL) underlain by a 15.5 feet thick layer of alluvial soils. The alluvial soils consist of 8 feet of sandy to gravelly clay (CL) of medium plasticity underlain by 4.5 feet of silty sand (SM) of low plasticity and 3 feet of sandy gravel. Below the alluvial soil is a weathered shale and bedrock. The water table elevations observed at the time of drilling at Boring SS-134 and inferred from a nearby piezometer for Boring SS-134A are at El. 710 and 709, respectively, and are depicted in Figure C-15. The design water table is at El. 716.

Based on the criteria established by Seed and Idriss (1981) and other published results, the upper 8 feet of the alluvial sandy to gravelly clay is considered to be non-liquefiable. The 3 feet layer of sandy gravel with high blow counts (N = 26 to greater than 50) is also considered to be non-liquefiable. Thus, the only layer that needs to be evaluated against liquefaction

potential is the silty sand approximately 4.5 feet thick. The blow counts obtained from two samples in this layer were 3 and 8 in Boring SS-134 and 4 and 9 in Boring SS-134A. The grain size distributions of the two split-spoon samples from the two borings are shown in Figures 5 and 6 for Borings SS-134 and -134A, respectively. As illustrated in Figures 5 and 6 the grain size distributions of the silty sand from both borings are very similar. Values of D_{50} range from 0.14 to 0.15 mm in Borings SS-134 and from 0.08 to 0.12 mm in Borings SS-134A. The fines content (percent finer than .074 mm) is approximately 30 percent in Boring SS-134 and ranges from 32 to 43 percent in Boring SS-134A. Based on the grain size distributions, the cyclic strength characteristic of the silty sands was estimated from the correlations for $D_{50} < 0.15$ mm in accordance with Seed and Idriss (1981).

The induced shear stress ratios below the ground surface in each boring were computed based on the procedure described in Section 3.2. Comparisons of the induced shear stress ratios with the cyclic stress ratios causing liquefaction are shown in Figures 7 and 8 for Borings SS-134 and -134A, respectively. The comparisons shown in Figures 7 and 8 indicate that the cyclic strength of the silty sand layer in Borings SS-134 and -134A is higher than the induced shear stress due to the SSE. The factor of safety defined in Section 3.2 (the ratio of the cyclic stress ratio causing liquefaction to the induced stress ratio due to earthquake shaking) is calculated to range from 1.1 to 1.6. The factor of safety of 1.1 was obtained for N = 3 in Boring SS-134. Redrilled Boring SS-134A showed that the N-value increased to 4 and the calculated factor of safety increased slightly. Based on the cyclic strength determined from the preliminary cyclic triaxial test results described in Section 3.3, the calculated factors of safety range from 1.5 to 1.6.

4.2.2 Borings SS-138 and SS-138A - The soil profile, blow counts and index properties obtained at Borings SS-138 and SS-138A are depicted in Figure C-18. The ground surface elevations at Borings SS-138 and SS-138A are at El. 727.2 and 726.7 feet, respectively. The bottom of the pipeline is located at El. 718.5 feet at this location. The soil profile depicted in Figure C-18 consists of a 7 feet thick fill of clayey gravel (GC) to silty clay (CL) and inorganic silt (MH) underlain by 17 feet of alluvial soils. The alluvial soils consist of 6 feet of sandy silt (ML) of low plasticity underlain by 11 feet of sandy to clayey silt (ML, CL) to silty sand (SM) of low plasticity. Comparisons between the two boring logs shown in Figure C-18 indicate that the alluvial soils below the water table in Boring SS-138 are more silty and clayey than those in Boring SS-138A. Below the alluvial soils is a weathered shale and shale bedrock. The water table elevations observed at the time of drilling at Boring SS-138 and inferred from a nearby piezometer for Boring SS-138A are at El. 711 and 710 feet, respectively, and are depicted in Figure C-18. The design water table is at El. 714.

The alluvial soils to be evaluated against liquefaction potential are the sandy silt to silty clay (ML-CL) and silty sand (SM) layers that are 10 feet thick below the design water table. The clay to sandy silt (CL-ML) encountered in Boring SS-138 has blow counts of 5 and is classified as a medium stiff clay and is considered non-liquefiable. Thus, the layers that were evaluated against liquefaction potential in this report are the silty sand (SM) and sandy silt (ML) encountered in these borings.

The grain size distribution of the samples of the silty sand taken from El. 714.4 to 710.4 in Boring SS-138 is shown in Figure 9. The D_{50} value of the silty sand in this depth range is equal to 0.074 mm and the fines content is approximately 50 percent. The grain size curve for the sample at El. 705 is not

available. Because of the plasticity of the soil (PI = 2.3), it is expected that the soil has a high fines content and is similar to the silty sand at the adjacent Boring SS-138A. The grain size distributions of the split-spoon samples of the silty sands taken in Boring SS-138A between El. 713.7 and 704.7 are shown in Figure 10. The grain size distributions shown in Figure 10 indicate that the silty sands encountered in this layer are very similar except the last sample between El. 705.7 to 704.7. Values of D_{50} for the samples above El. 705.7 range from 0.074 mm to 0.1 mm and the fines content ranges from 36 to 50 percent. The D_{50} value of the silty sand between El. 705.7 to 704.7 ranges from 0.15 mm to 0.18 mm and the fines content ranges from 21 to 31 percent.

Based on the procedure described in Section 3.2, the cyclic strength of the silty sand in Borings SS-138 and -138A was determined. Comparisons of the induced shear stress ratios with the cyclic stress ratios causing liquefaction are shown in Figures 11 and 12 for Borings SS-138 and -138A, respectively. The comparisons shown in Figures 11 and 12 indicate that the cyclic strength of the silty sand layer at Borings SS-138 and -138A is higher than the induced shear stress due to the SSE. The calculated factor of safety ranges from about 1.2 to 2.0. Based on the cyclic strength determined from the cyclic triaxial test results, the calculated factors of safety range from 1.6 to 1.9. Also shown in Figure 12 is the cyclic strength of the silts (ML) at El. 709.2 estimated conservatively using the correlations for silty sands. The calculated factor of safety for the silts is 1.5.

Boring SS-138A was also redrilled twice to investigate the SM layer with N = 4 at elevation 707. These redrills are identified as 138B and 138C. The soil profile, blow counts, and index properties for these borings are depicted in Figure C-19. In general, Borings SS-138B and -138C are offset 10 feet on either

side of 138A. The actual relationship is shown in Figure C-8. The companion borings show blow counts of 8 for SM-SC and 9 for SM material in Boring SS-138B and blow counts of 6 for SC and 4 for CL material in Boring SS-138C at about this same elevation. Thus, the low blow count SM layer in Boring SS-138A is not very extensive. The calculated factor of safety for the SM material in Boring SS-138B is 1.71. Thus, any possible adverse performance at Boring SS-138A would be very limited.

4.2.3 Borings SS-161 and SS-161A - The soil profile, blow counts and index properties obtained at Borings SS-161 and SS-161A are depicted in Figure C-23. The ground surface elevations at Boring SS-161 and SS-161A are at El. 732.4 and 732.9 feet, respectively. The bottom of the pipeline is located at El. 728 feet at this location. The soil profile depicted in Figure C-23 consists of 24 feet of alluvial soils underlain by a weathered shale and shale bedrock. The alluvial soils consist predominantly of silty clay (CL), clayey to sandy silt (CL-ML) and clayey sand (SC) with a thin layer of silty sand (SM) and gravel (GM-GP). The silty sand layer is approximately 4 feet thick and located between El. 720.5 to 715.5 feet in Boring SS-161 and between El. 722 and 718 feet in Borings SS-161A. The water table elevations observed at the time of drilling at Boring SS-161 and inferred from a nearby piezometer for Boring SS-161A are at El. 717.5 and 715, respectively, and are depicted in Figure C-23. The design water table is at El. 722 feet.

As discussed in Section 3.2, the silty clay (CL), clayey to sandy silty (ML-CL) and clayey sand (SC) are non-liquefiable except when the water content of the soils is higher than 90 percent of the liquid limit (LL) and the Atterberg limits plot above the A-line and the blow counts are low (less than 5). Thus, the two-foot layer of clayey soil with an N-value of 3 (CL-ML) situated between El. 716 and 714 feet encountered in Boring SS-161 is to be evaluated later using the results of

cyclic tests now being conducted (see Section 5). The layers that are evaluated against liquefaction potential in this report are the 4-foot thick layer of silty sand (SM) of and the 2-foot thick layer of sandy silt (ML) in these two borings.

The grain size distributions of the split-spoon samples of the silty sand taken in Boring SS-161 between 719.5 and 715.5 and in Boring SS-161A between El. 722 and 718 feet are depicted in Figures 13 and 14, respectively. The sample taken in Boring SS-161 contained approximately 20 percent of gravel size particles. The behavior of the soil-gravel matrix of the sample having a grain-size distribution similar to that shown in Figure 13 is controlled by the fine particle matrix but not by the gravel size particles. Thus, D_{50} should be determined from the grain size distribution of the fine particle matrix as shown in Figure 13. The D_{50} value determined for the fine particle matrix is equal to 0.17 mm and the fines content is about 30 percent. The values of D_{50} of the silty sand in Boring SS-161A range from 0.1 to 0.23 mm and the fines content ranges from 17 to 32 percent.

Comparisons of the induced shear stress ratios with the cyclic stress ratios causing a pore pressure ratio of 100 percent are shown in Figures 15 and 16 for Borings SS-161 and -161A, respectively. The comparisons shown in Figures 15 and 16 indicate that the cyclic strength of the silty sand layer in Borings SS-161 and -161A is higher than the induced shear stress due to the SSE. The calculated factors of safety range from 1.6 to 2.4. Based on the cyclic strength determined from the cyclic triaxial test results, the calculated factors of safety range from 1.6 to 1.8. Also shown in Figures 15 and 16 are the cyclic strengths of the silts (ML) in the borings estimated conservatively using the correlations for silty sands. The calculated factor of safety for the silts is about 1.2.

4.3 Results of Evaluation Along the ERCW Pipeline Route

Using the data of the SPT blow counts, the grain size distributions, and other index properties (the Atterberg limits, water content, etc.), the liquefaction potential of the alluvial soils encountered along the ERCW pipeline route due to the SSE has been evaluated. The results of this evaluation are summarized in Tables 2, 3, 4 and Figures 17, 18, 19 and 20. Table 2 summarizes all the locations of the silty sands (elevation and boring number) where the calculated factor of safety (defined as the ratio of the cyclic stress ratios causing a pore pressure ratio of 100 percent to the induced cyclic stress ratios due to earthquake shaking) is relatively low. The locations of these SPT samples are identified on the soil profiles along the ERCW pipeline route shown in Figures 17, 18 and 19 for Section A-A, the main plant area section and the Diesel Generation Building area section, respectively.

It is noted that the laminated blocky soils of the weathered shale are considered non-liquefiable. Thus, the weathered shale with the blow counts of 6 and 4 encountered in Boring SS-49 is considered non-liquefiable. In some of the borings where no grain size distributions are available because either no split-spoon samples were recovered (i.e. Borings SS-65, -143) or the test data were not available (i.e. Boring SS-135) the SPT and grain size data from the adjacent redrilled borings were used in the evaluation.

The results of the evaluation for the sands and silty sands summarized in Table 2 and Figures 17, 18 and 19 show that among all the SPT data obtained from a total of 70 boring locations spacing at approximately 100 feet apart along the ERCW pipeline route and a total of 464 SPT data points below the design water table, only three N-values in three borings result in a calculated factor of safety equal to or slightly less than one (i.e.

N=2 at El. 693.8 in Boring SS-50, N=2 at El. 711.5 in Boring SS-158, and N=2 at El. 712.1 in Boring SS-128). At two of these three boring locations (Borings SS-50 and -158), redrilling was made using the rotary drilling techniques in 1981. The results indicate that the N-value increased from 2 to 5 at the corresponding depth in Boring SS-50 and -50A. At Boring SS-158, the redrilling in Boring SS-158A encountered a clayey sand (SC) with an N-value equal to 3 at the corresponding depth where the N-value was 2. Thus, redrilling in 1981 removed the potential problem soils from considerations at the locations of Borings SS-50 and -158. Thus effectively only one two-foot layer in one boring (SS-128) of all 84 borings along the pipeline has a factor of safety of less than 1.0.

Eleven N-values in seven boring locations (SS-134, -134A, -65, -65B, -138A, -163, -84, -125 and -25) have a comparatively low calculated factor of safety, but still greater than one. These points are located at a few isolated locations along the pipeline route as summarized in Table 2 and Figure 20. Among these boring locations, redrilling at Boring SS-163 (redrilled boring is identified as SS-163A) reveals that the silty sands encountered in Boring SS-163 with N=3 and 4 (F.S. = 1.12 and 1.18, respectively) correspond to a silty sand (D_{50} = 0.08 mm) with N=4 (F.S. = 1.23) and a clayey sand (CL-SM) with N=5 in Boring SS-163A. Thus, there should not be liquefaction potential at Boring SS-163.

As discussed earlier, Boring SS-138A has two companion Borings SS-138B and -138C in addition to original Boring SS-138. The presence of these companion borings severely restricts the lateral extent of the questionable SM material in SS-138A.

It should be noted that in all borings identified to have low N-values with low factors of safety, only one low N-value is present at each boring location except in Boring SS-65B. In Boring

SS-65B, two successive drives resulted in low N-values (3 and 5). This indicates that the thickness of the silty sand with low N-values is less than two feet except at Boring SS-65B.

Borings SS-25, -82, -125, and -128 in the main plant area each contain only a single 2-foot layer of SM material with rather low factors of safety. Boring SS-25 is located beneath the Diesel Generator Building. This material was removed so the Diesel Generator Building could be founded on 1032 stone (an engineered granular fill). The remaining three boring locations were not redrilled in 1981. It is possible, based on the experience of Borings SS-158, -161, and -163 that redrilling would have increased the factor of safety sufficiently to eliminate these borings from consideration also. Nevertheless, these borings are located in a level area where slope failure is impossible. Thus the only possible consequence of liquefaction (either partial or complete) in this area would be induced settlement due to the cyclic mobility of the soils as the excess pore water pressure is dissipated. Such settlements could range from approximately 1% of layer thickness for partial liquefaction (excess pore water pressure < 100% of the confining pressure) to no more than 5% of layer thickness for complete liquefaction. Considering the thickness of the questionable SM material to be 2 feet (the sampling interval for the continuous splitspoon sampling performed), the resulting settlement could range from 0.02 to 0.1 feet or 0.2 to 1.0 inches. The utilities in this area are fully capable of withstanding movements of this amount. In addition, the excess pore water pressure developed within a single 2-foot thick layer at a depth of 15 to 20 feet would not have surface manifestations. It would cause a slight rise in the ground water level as the excess pore water pressure induces water to migrate upwards. Further, the fairly thick relatively impervious alluvial clay layer at the ground surface would also inhibit the migration of ground water.

The results of the evaluation for the sands and silty sands are summarized in Table 2. Using the empirical evaluation procedure based on blow counts, the listed SM materials have low factors of safety. If in addition these same SM materials are evaluated using the cyclic test data, which are still preliminary, the factors of safety shown in the last column of Table 2 are obtained. These factors of safety range from 1.44 to 1.85. The cyclic test results are directly applicable for those borings in the vicinity of the two test pits (SS-134, -134A, -65, -65B, and -138A). Assuming that the test results are also applicable in the main plant area, the listed factors of safety for Borings SS-25, -82, -125 and -128 are obtained. This indicates for both regions that the SM materials possess adequate factors of safety against liquefaction. This conclusion is preliminary since the cyclic test data are still preliminary. This conclusion will be re-evaluated upon completion of the cyclic testing.

For the silts (ML) encountered along the pipeline route, a preliminary evaluation was made by using cyclic strengths conservatively estimated based on the correlations for silty sands. The layers with a calculated factor of safety less than 1.2 are summarized in Table 3. The factors of safety calculated for the silts (ML), identified in Table 3, range from 1.06 to 1.19. Actual factors of safety are expected to be higher than those summarized in Table 3 because the cyclic strengths of silts are likely to be much higher than that of silty sand material. Among those areas identified in Table 3, Borings SS-140 and -143A, which are located on the south leg of the pipeline route, contain the silt material with factors of safety of 1.17 and 1.19, respectively. Thus, liquefaction of the silt material is not expected in this section of the pipeline. Similarly, for the silt having a computed factor of safety of 1.17 in Boring SS-161, liquefaction is not likely to occur. Boring SS-27 is beneath the Diesel Generator Building. The material in this boring has been removed. Borings SS-101 and -84 each contain

only a single two-foot layer of silt material with factors of safety of 1.06 and 1.09, respectively. These borings are located in a level area where slope failure is impossible. Thus, as discussed for silty sand the only possible consequence of liquefaction or cyclic mobility is the potential settlement due to dissipation of excess pore pressure. As discussed above for the silty sand material, the resulting settlements are tolerable.

For the clayey soils encountered (silty clay, CL and clayey sand, SC) the criteria suggested by Seed and Idriss (1981) were used. The locations of the clayey soils for which the blow counts were low and the water content was greater than 0.9 LL are identified in Table 5. The cyclic behavior of these soils will be evaluated after the result of the cyclic tests described in Section 5 are available.

4.4 Summary

The liquefaction evaluation of the soils along the ERCW pipeline was conducted according to the procedures given in Seed and Idriss (1981). This procedure differentiates between clean sands and silty or dirty sands. The results of this evaluation are presented and discussed in Section 4.2 and 4.3. Based on these results we draw the following general conclusions.

1. The soils, while similar in profile in various lengths of the pipeline, are not continuous in detail. Most conditions are very localized.

2. The sandy soil (SM) at the site is relatively dirty or silty with silt contents ranging up to 50 percent. The cyclic performance of these silty sands is better than for a clean sand when both materials have the same blow counts or other similar characteristics. This

increased cyclic performance is accounted for in the Seed and Idriss (1981) procedure.

3. The silts (ML) along the pipeline were conservatively evaluated treating them as a silty sand. Based on this and the resulting factors of safety, we concluded the silts are non-liquefiable.

4. The clays (CL and SC) will be evaluated after the results of the cyclic simple shear tests described in Section 5.2 are available. However, we tentatively conclude the clays are non-liquefiable.

5. The redrilling program of 1981, in which 10 selected borings were repeated using rotary drilling techniques and drilling mud, yield higher blow counts than did the earlier investigations using hollow stem augers without drilling fluid. In many cases this redrilling showed sufficiently increased insitu cyclic strength (as inferred from blow counts) to remove earlier problem soils from consideration. Borings SS-50, -158, and -163 fall in this category.

6. On the east and north legs of the pipeline, the evaluation shows all soils to possess ample factors of safety against potential liquefaction.

7. On the south leg of the pipeline, borings 134, 65, and 138 contain SM material with relatively low factors of safety (but still greater than 1.0) when using the N-value approach. These three borings are spaced over approximately 500 feet of the pipeline with seven intermediate and companion borings containing SM materials with higher factors of safety. Thus the materials with a low factor of safety are widely

scattered and localized. We tentatively conclude that these soils are not subject to liquefaction. A cyclic testing program, discussed in Section 5.1, to confirm this conclusion is currently in progress on undisturbed samples taken from testpits 1 and 2 in this general vicinity. Based on the preliminary cyclic test results, these same SM materials have factors of safety ranging from 1.44 to 1.61.

8. In the main plant area, Borings SS-25, -84, -125, and -128 contain SM material with a relatively low factor of safety. While these zones may have been eliminated by redrilling which was not done, the consequences of a postulated worst case behavior in this area are tolerable. Such a postulation yields induced settlements ranging from 0.2 to 1.0 inches. The utilities in this area are fully capable of withstanding movements of this amount.

5.0 ONGOING STUDIES

The results of the liquefaction evaluation given in the previous section have pointed to several potentially questionable areas. These questions arise, not due to the soils and their characteristics, but rather due to a lack of established and documented data against which to compare them. We have already begun several studies in these areas to remove any potential concerns. These consist of cyclic triaxial shear tests of silty sands, cyclic direct simple shear tests of clays, and additional analytical studies.

5.1 Cyclic Triaxial Tests of Silty Sands

Two testpits were opened in December, 1981. These are described in detail in Appendix D. Block samples were taken of selected sands from each pit - one from pit 1 and two from pit 2. This material will undergo cyclic triaxial testing. The objective of the testing program is to evaluate the cyclic strength of the silty sand materials. This is a confirmatory testing program and is intended to supplement the previous evaluation. As presently planned, each block sample will yield 4 specimens for testing. All tests are performed under isotropic conditions with a confining pressure of 1 tsf. Of the planned 12 tests, four have been completed. Preliminary results indicate that the samples possess adequate cyclic strength. The material tested from all testpits classifies as SM with fines content ranging from 30 to 40% and D_{50}'s of about 0.10 mm. Complete details are given in Table 5. The block sample material was selected on the basis of being the loosest sands within each test pit. The material from test pit 1 is a red to dark brown silty sand. It was taken at elevation 706 about 1 feet above the bottom of the test pit and about 2 feet below the ground water level. The materials from test pit 2 are dark brown in color and were taken from elevation 706 and 707, respectively, about 1 and 2 feet

above the bottom of the test pit and about 2 and 1 feet below the ground water level.

5.2 Cyclic Direct Simple Shear Tests of Clays

A third block sample of a clay material was also taken from testpit 2. The clay material will be subjected to cyclic direct simple shear tests. The objective of this testing is to document the performance of the clays encountered at Watts Bar. Based on Seed and Idriss (1981), certain clays may undergo liquefaction. The rules or criteria for evaluating this possibility are fairly simple but not definitive. To remove any possible concerns, the wet clays encountered at Watts Bar will be tested. Currently four cyclic tests are anticipated. The block samples were taken January 11, 1982. However, due to the recent foul weather in the East and Northeast, the specimens were delivered to the testing laboratory February 1. It is anticipated that testing will be completed in three weeks. Cyclic direct shear testing was chosen instead of cyclic triaxial testing to avoid having to correct the triaxial test results to field conditions. This correction is well established for sands but not for clay.

5.3 Other Studies

Static cone penetration tests were performed along and adjacent to the ERCW pipeline near the Intake Pumping Station and in the vicinity of the two testpits. This program is described in detail in Appendix E. The data are currently being reduced. The purpose of the CPT program was to aid in establishing foundation conditions adjacent to the pipeline. The cone was selected primarily because it is much faster than the standard penetration test. The primary aim was to define zones of hard, medium, and soft soils and to yield a top of weathered shale or basal gravel profile. Such a profile would be needed if any

remedial measures are required or if other analytical studies are warranted.

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TABLE 1

DESIGN GROUND WATER LEVEL

Boring	Finished Grade	Design W.T.	Boring	Finished Grade	Design W.T.
49	712	701	146	741	711
49A	712	701	147	742	712
131	714	702	92	731	705
50	717	704	93	712	702
50A	717	704	149	706	700
132	719	706	94	710	700
51	722	707	150	709	702
133	725	716	95	714	706
134	727	716	151	718	709
134A	726	716	96	720	710
135	727	716	152	720	711
135A	727	716	97	720	712
65	725	716	153	720	712
65B	727	716	154	720	713
136	727	715	155	720	713
137	727	715	99	720	714
138	727	714	156	720	715
138A	727	714	157	724	717
139	728	712	158	728	717
140	727	712	158A	728	717
87	725	712	101	726	718
141	725	712	159	732	720
88	721	711	160	733	722
142	722	708	161	732	722
143	723	706	161A	733	722
143A	723	706	162	734	722
144	729	711	163	737	722
90	728	713	163A	738	722
145	737	712	80	741	722

TABLE 1

DESIGN GROUND WATER LEVEL
(cont'd)

Boring	Finished Grade	Design W.T.
164	741	722
165	741	722
166	741	722
84	741	722
167	740	720
168	740	720
169	741	720
170	741	720
125	727	718
126	729	718
127	728	718
128	727	718
129	725	718
130	725	718
82	728	718
86	725	718
25	742	720
26	742	720
27	742	720
28	742	720
138B	727	714
138C	727	714
143B	723	706
143C	722	706

TABLE 2

SUMMARY OF LIQUEFACTION EVALUATION OF SILTY SANDS (SM) ALONG ERCW PIPELINE ROUTE

Boring No.	Elev. (ft)	Blow Counts	Soil Type	L.L.	PI	W.C. %	D_{50} mm	Fines[1] Content %	F.S.[2] N-value	F.S.[2] Lab. Tests	Remark
SS-50	693.8	2	SM	NP	NP	31.5	0.087	47.	0.89	1.54	(3)
SS-134	710.5	3	SM	NP	NP	29.3	0.148	26.	1.07	1.61	(4)
SS-134A	709.5	4	SM	23.0	1.0	30.0	0.105	35.	1.13	1.54	(4)
SS-65B	709.2	3	SM	25.0	1.0	33.1	0.10	38.	1.04	1.57	(4)
SS-65B	707.2	5	SM	25.0	1.0	32.5	0.10	34.	1.18	1.52	(4)
SS-65	706.0	5	SM	28.9	3.5	28.2	0.14	34.	1.10	1.44	(4)
SS-138A	707.2	4	SM	25.0	2.0	28.1	0.09	44.	1.16	1.61	(4)
SS-158	711.5	2	SM	22.7	2.5	32.2	0.088	44.	0.97	1.61	(3)
SS-163	717.0	3	SM	27.2	3.3	31.1	0.097	45.	1.12	1.72	(3)
SS-163	715.0	4	SM	29.7	4.7	33.5	0.09	43.	1.18	1.65	(3)
SS-84	713.4	2	SM	24.8	2.2	30.1	0.11	42.	1.01	1.69	(5)
SS-128	712.1	2	SM	NP	NP	23.7	0.22	16.	0.6	1.55	(5)
SS-125	714.4	2	SM	NP	NP	29.0	0.13	8.	1.09	1.65	(5)
SS-25	715.6	2	SM	NP	NP	29.2	0.13	31.	1.14	1.85	(5)

Notes:

(1) Percent finer than 0.074mm.

(2) Factor of safety is defined as a ratio of cyclic stress ratio causing a pore pressure ratio of 100 percent or ± 2.5 percent strain to induced cyclic stress ratio.

(3) Superseded by more recent data.

(4) Pending results of cyclic testing.

(5) Eliminated by settlement analysis.

TABLE 3

SUMMARY OF LIQUEFACTION EVALUATION OF SILTS (ML) ALONG ERCW PIPELINE ROUTE

Boring No.	Elev. (ft)	Blow Counts	Soil Type	L.L.	PI	W.C. %	D_{50} mm	(1) Fines Content %	(2) F.S.
SS-140	708.7	3	ML	--	--	17.4	0.073	50.	1.17
SS-143A	709.0	2	ML	39.0	11.0	21.6	0.03	73.	1.19
SS-101	712.5	3	ML	24.7	2.0	31.9	0.072	53.	1.06
SS-161	712.4	5	ML	25.7	2.3	30.9	0.076	51.	1.17
SS-84	711.4	3	ML	24.5	1.3	31.4	0.07	52.	1.09
SS-27	713.1	3	ML	23.1	2.9	24.5	0.07	51.	1.11

Notes: The cyclic strength of silts used in the evaluation is estimated based on empirical correlations developed for silty sands. This approach is very conservative for these materials.

(1) Percent fines than 0.074mm

(2) Factor of safety is defined as a ratio of cyclic stress ratio causing a pore pressure ratio of 100 percent to induced cyclic stress ratio.

TABLE 4

SUMMARY OF CLAYEY SOILS ALONG ERCW PIPELINE ROUTE REQUIRED FURTHER EVALUATION BASED ON CYCLIC TEST RESULTS

Boring No.	Elev. (ft)	Blow Counts	Soil Type	L.L.	PI	W.C. %	D_{50} mm	(1) Fines Content
SS-137	706.9	3	CL	34.7	10.7	33.9	0.048	67.
SS-143	691.1	2	CL	31.2	15.5	--	--	--
SS-143A	701.0	3	SM-SC	21.0	5.0	21.2	0.093	45.
SS-143A	699.0	4	CL	25.0	8.0	24.9	0.065	55.
SS-143C	698.5	2	CL	23.0	8.0	23.7	0.04	62.
SS-161	714.4	3	CL-ML	36.8	13.2	35.8	--	--
SS-165	716.7	3	SM-SC	30.7	8.1	33.3	--	--
SS-165	714.7	2	SM-SC	30.7	8.1	34.4	--	--
SS-167	711.7	2	CL	31.0	15.2	34.1	--	--
SS-168	713.6	1	CL	25.5	9.0	28.4	--	--
SS-127	714.2	2	CL	26.3	8.3	32.2	--	--
SS-127	712.2	0	SM-SC	23.3	4.4	36.1	--	--
SS-125	712.4	1	CL	25.6	8.3	33.7	--	--
SS-27	715.1	2	CL	29.3	12.4	31.8	--	--

TABLE 5

SUMMARY OF PRELIMINARY CYCLIC TRIAXIAL TEST RESULTS OF SILTY SANDS

Sample	Soil Classification	Gradation (%) Sand	Silt	Clay	Specific Gravity G_s	Grain Sizes (mm) D_{10}	D_{30}	D_{60}	C_u	Confining Pressure tsf	Cyclic Stress Ratio $\frac{\sigma_{dp}}{2\sigma_{3c}}$	No. of Cycles Required to Cause $\sigma_u=\sigma_{3c}$	$\pm 2.5\%$ Strain	$\pm 5\%$ Strain
1-A-1	SM	57	27	16	2.73	.0014	.0283	.1090	76.	1.0	0.295	18	27	39
1-A-2	SM	67	21	12	2.74	.0012	.0629	.1287	100	1.0	0.355	5	6	10
2-A-1	SM	66	25	9	2.76	.0061	.0619	.1231	20.	1.0	0.192	430	455	475
2-A-2	SM	69	22	9	2.74	.0057	.0693	.1240	22.	1.0	0.30	11	11	14

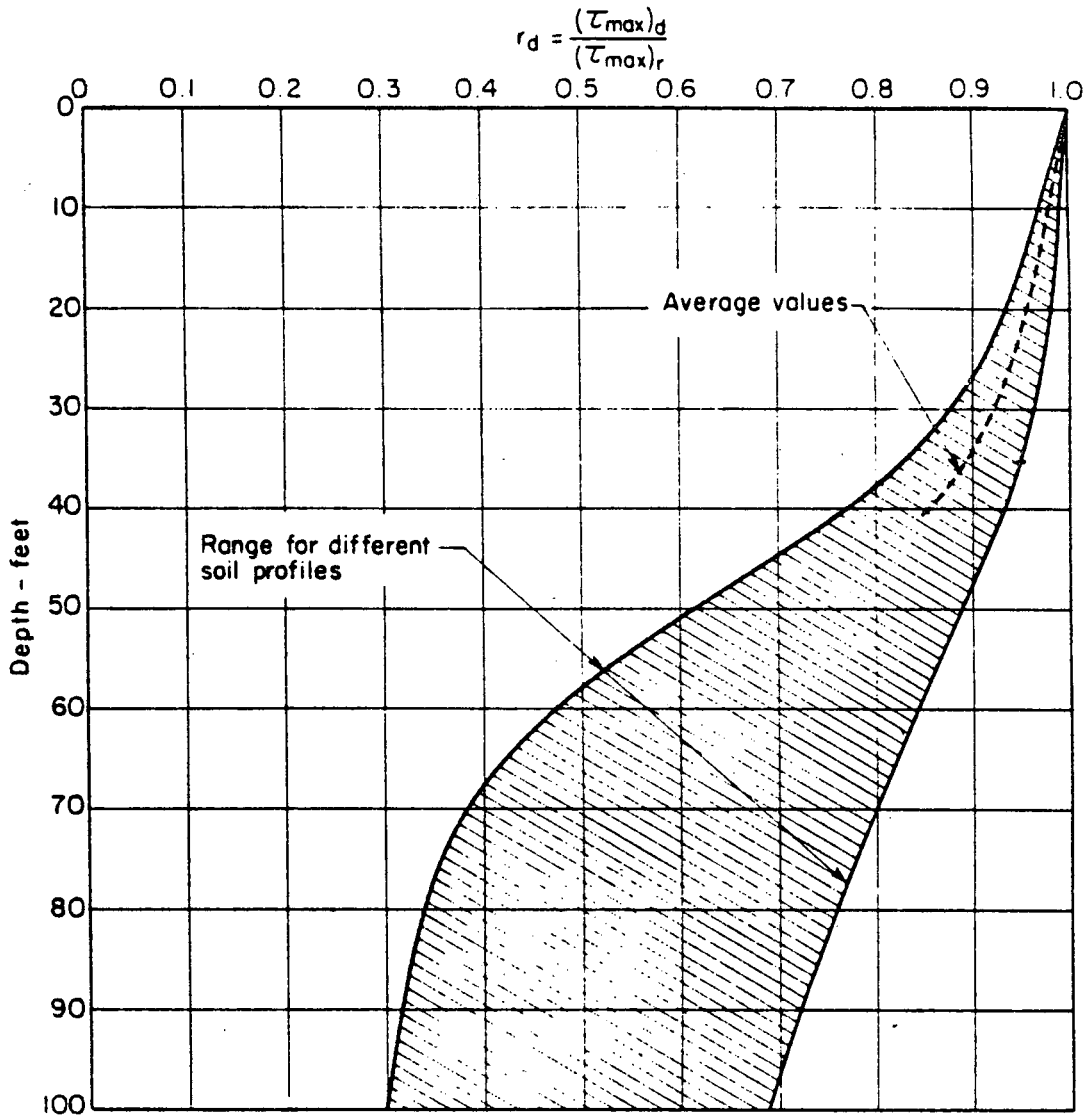


Figure 1 — Range of Values of r_d for Different Soil Profiles
(After Seed and Idriss, 1970)

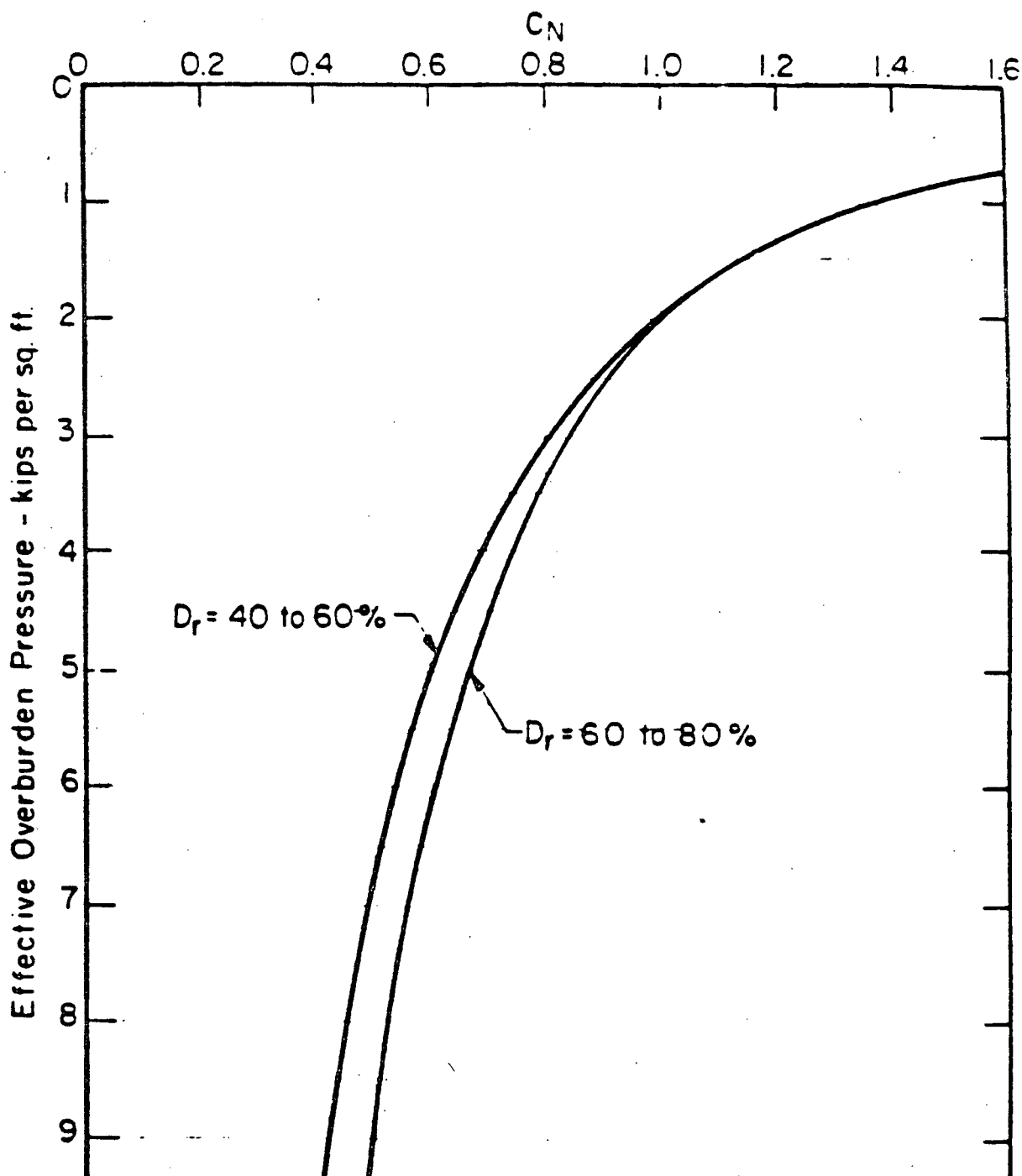


Figure 2 – Recommended Curves for Determination of C_N Based on Averages for W.E.S. Tests (After Seed and Idriss, 1981)

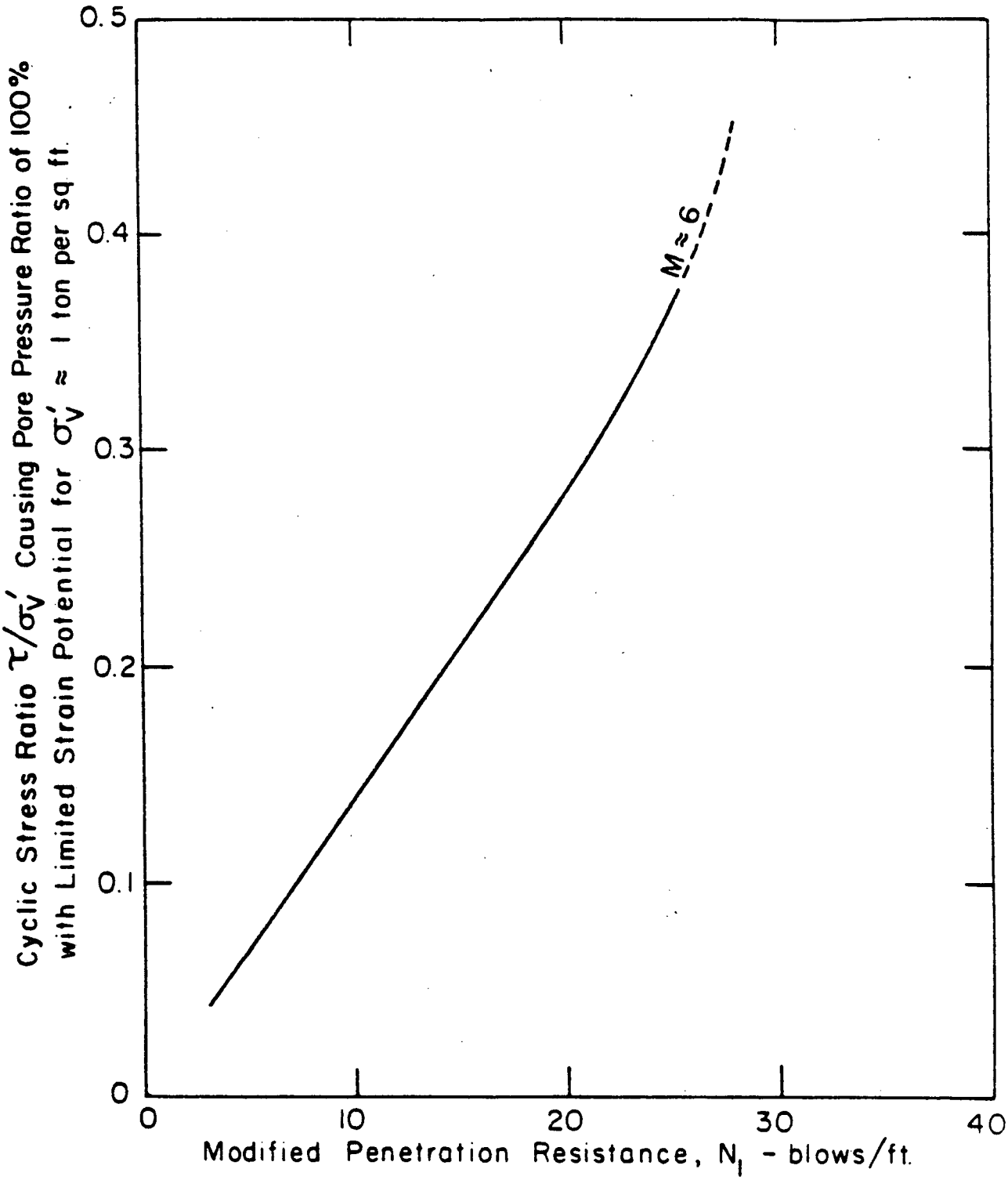


Figure 3 – Chart for Evaluation of Liquefaction Potential for M = 6 Earthquake (From Seed and Idriss, 1981)

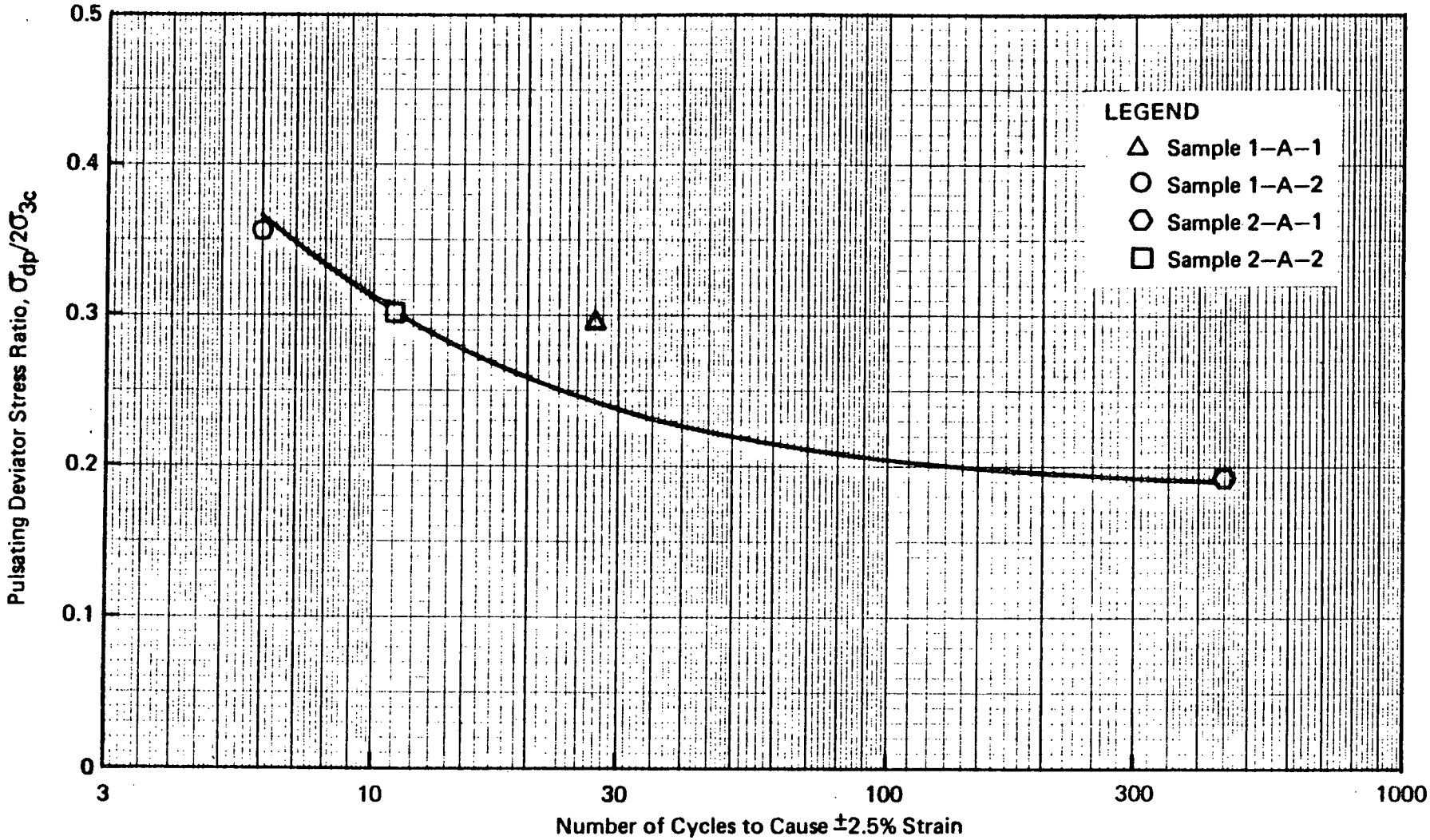


Figure 4 — Preliminary Results of Cyclic Triaxial Tests on Silty Sands
Watts Bar Nuclear Plant, ERCW Pipeline Route

BORING NO.	SAMPLE NO.	ELEV. (ft)	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
SS–134	GR4	711.7–709.7	————	NP	NP	SM
SS–134	GR5	709.7–707.7	- - - - - -	NP	NP	SM

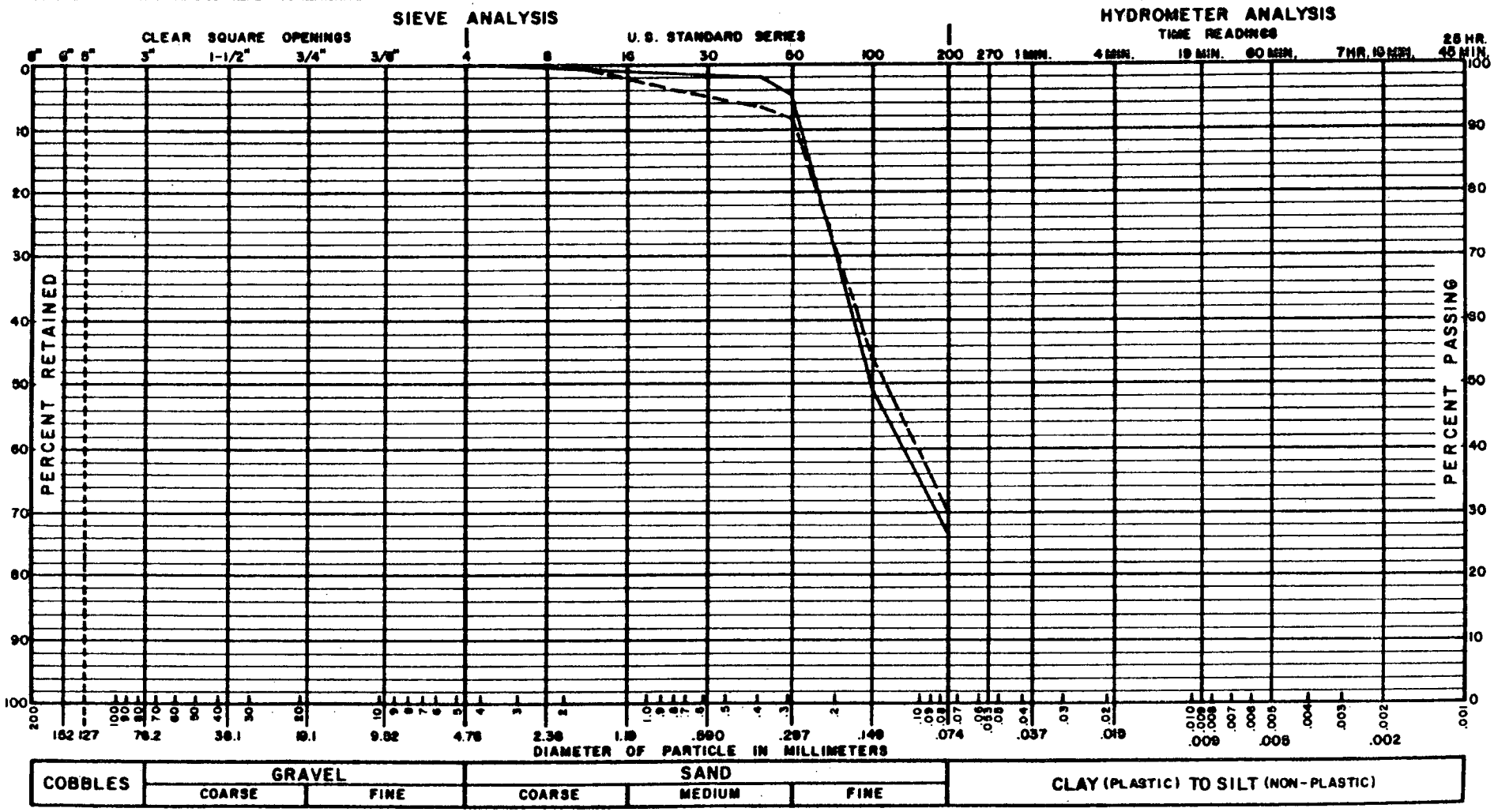


Figure 5 – Grain Size Distribution Curves for Silty Sand Samples from Boring SS–134

BORING NO.	SAMPLE NO.	ELEV. (ft)	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
SS–134A	8A	710.5–709.9	————	23	1	SM
SS–134A	8B	709.9–709.4	— — —	NP	NP	SM
SS–134A	9A	708.0–707.4	– – – – –	24	2	SM
SS–134A	9B	707.4–707.0	.....	24	1	SM
SS–134A	10A	706.5–706.3	—·—·—	NP	NP	SM

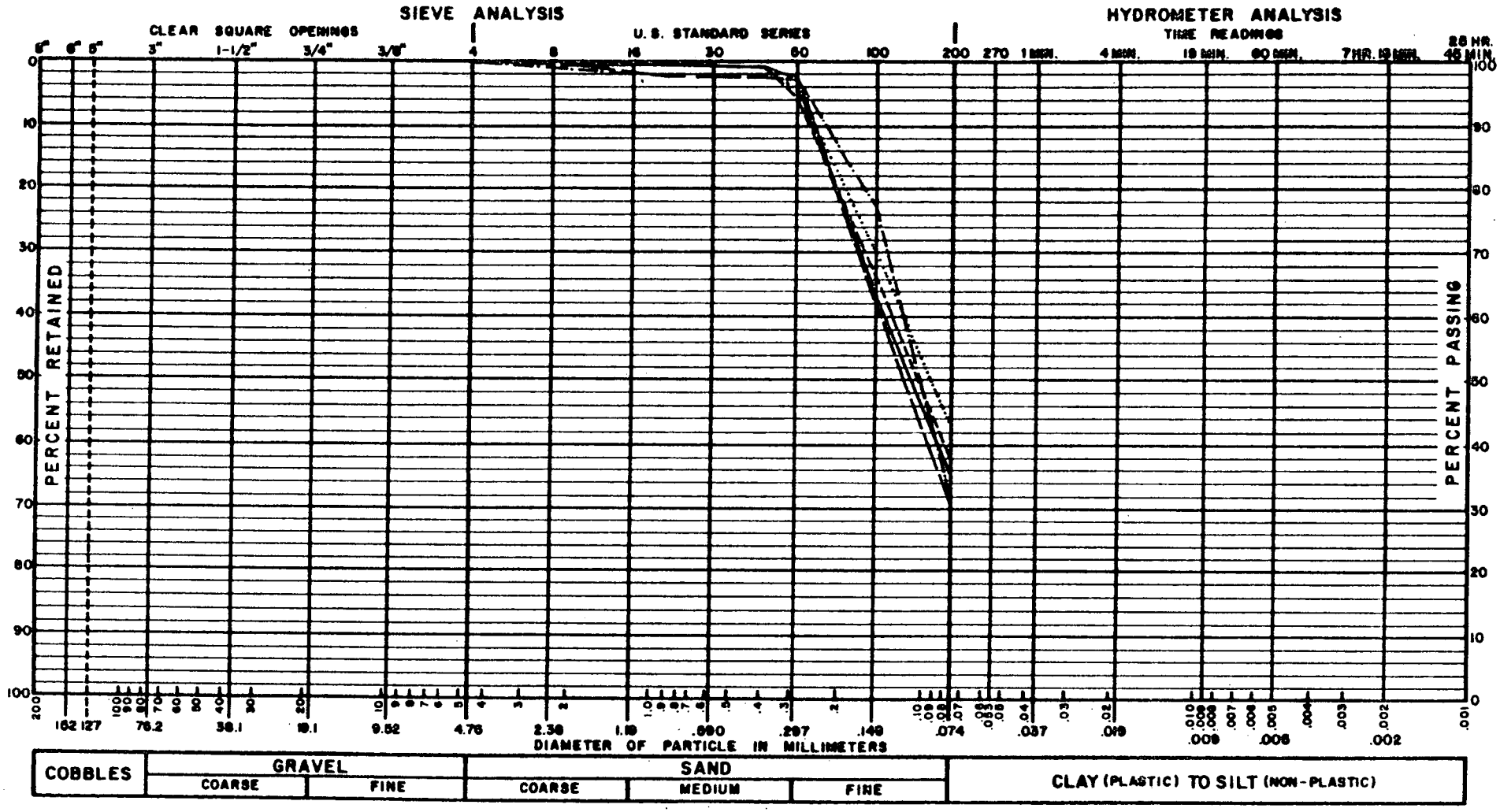
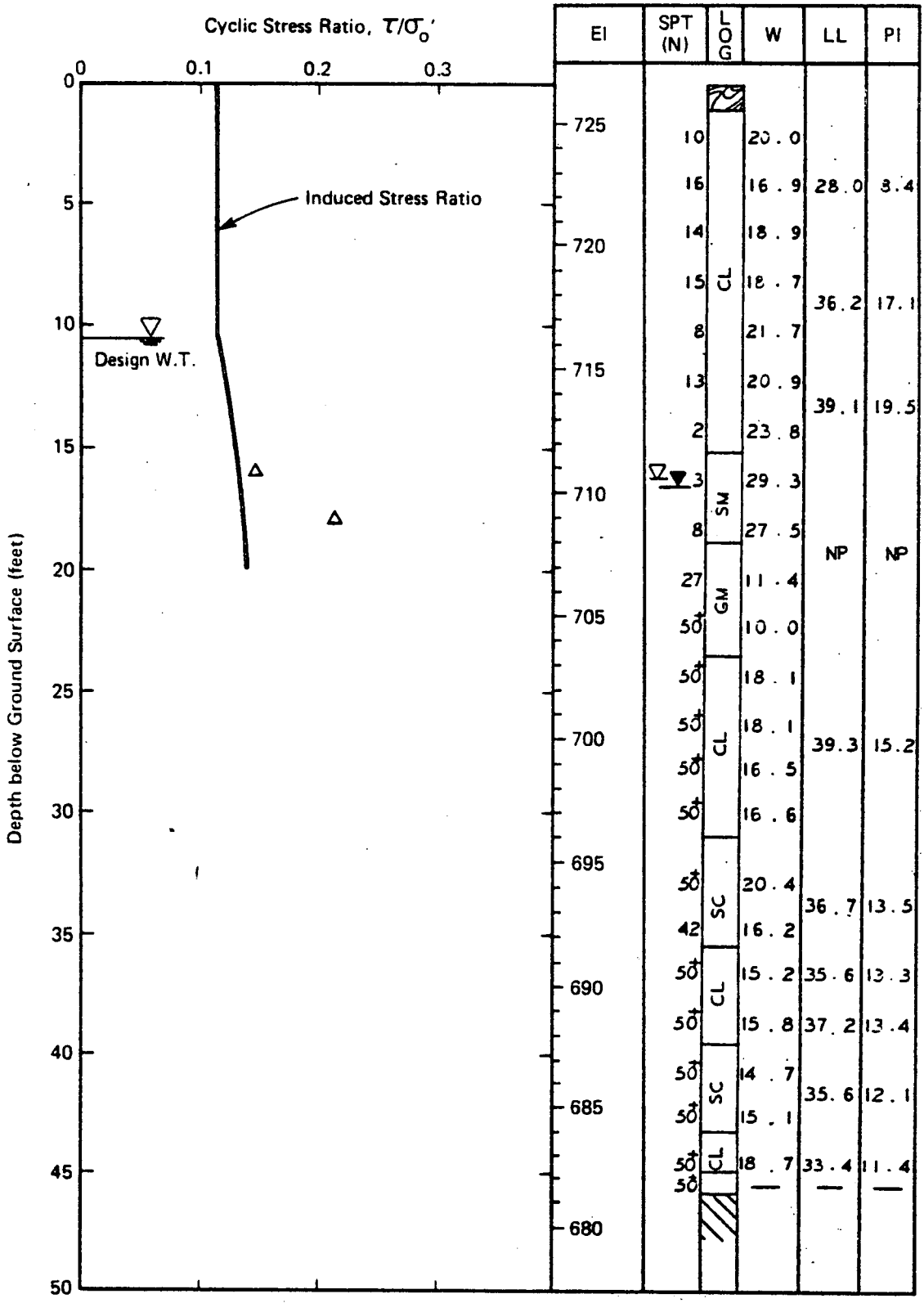


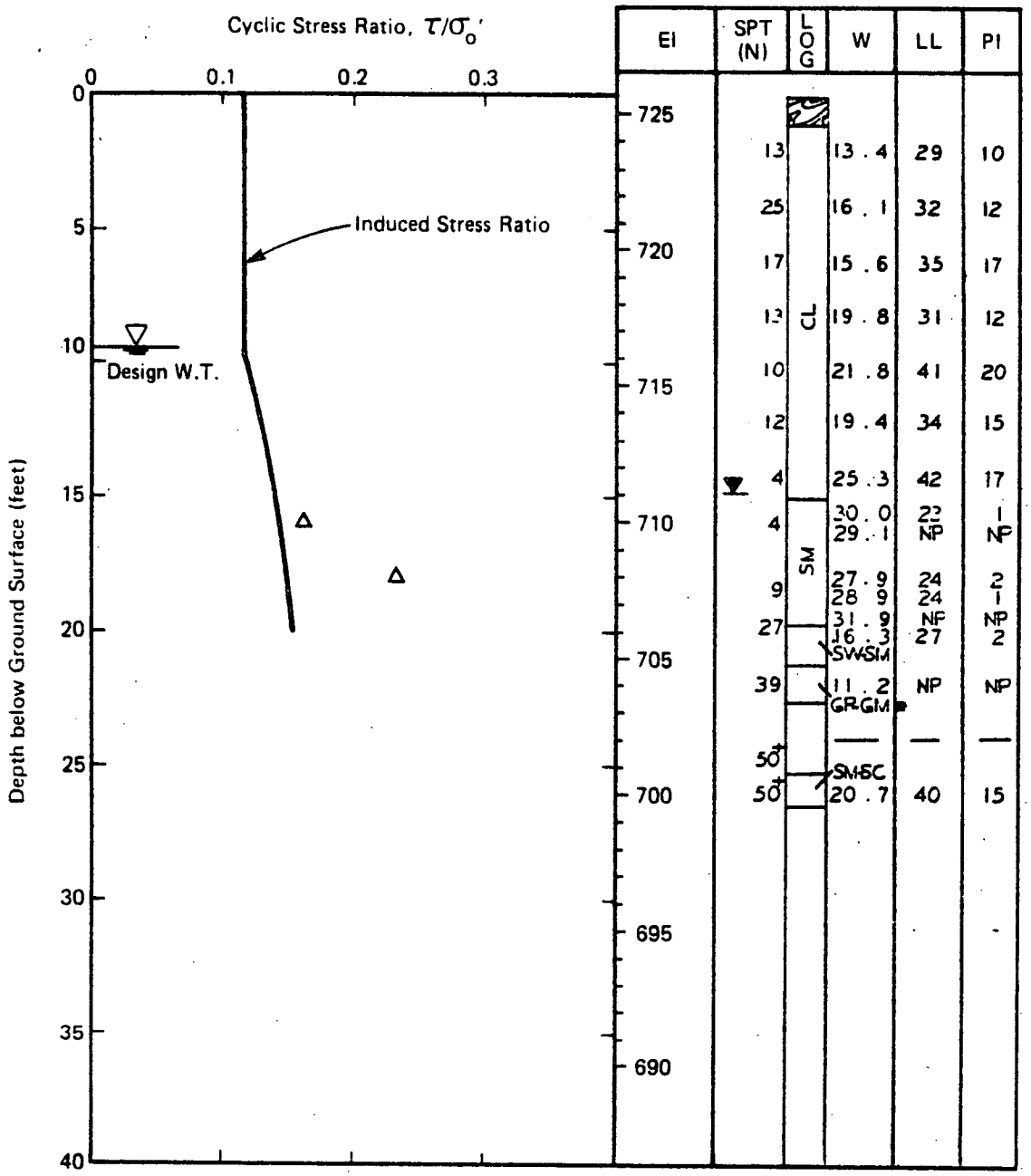
Figure 6 – Grain Size Distribution Curves for Silty Sand Samples from Boring SS–134A



LEGEND

▲ Estimated cyclic strength (cyclic stress ratio causing pore pressure ratio of 100%) based on N–values.

Figure 7 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Pore Pressure Ratio of 100 Percent – Boring SS–134



LEGEND

△ Estimated cyclic strength (cyclic stress ratio causing pore pressure ratio of 100%) based on N–values.

Figure 8 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Pore Pressure Ratio of 100 Percent – Boring SS–134A

BORING NO.	SAMPLE NO.	ELEV. (ft)	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
SS-138	GR6	714.4-710.4	————	28.1	2.5	SM

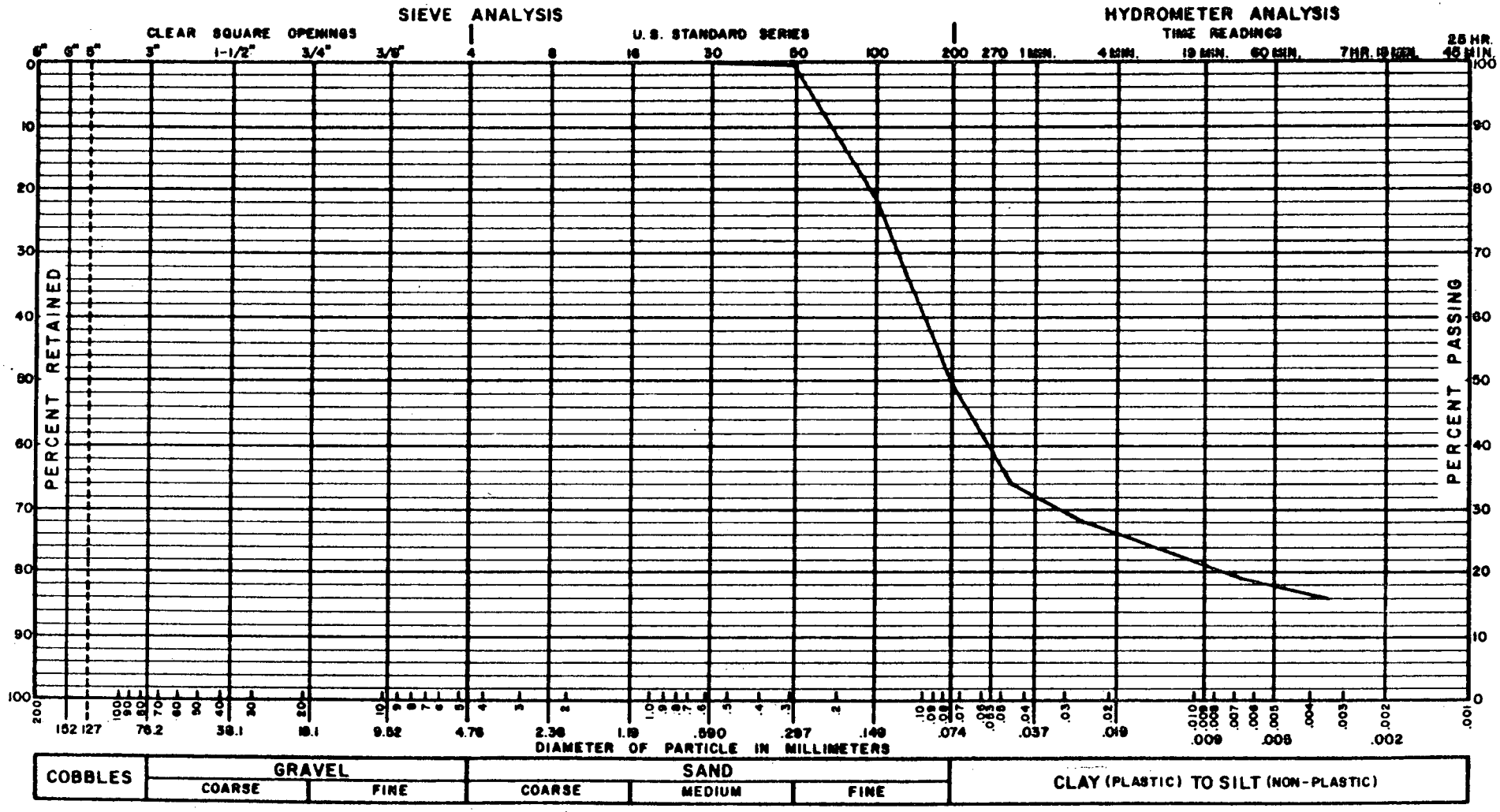


Figure 9 — Grain Size Distribution Curve for Silty Sand Sample from Boring SS—138

BORING NO.	SAMPLE NO.	ELEV. (ft)	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
SS–138A	6A	713.7–712.7	————	29	3	SM
SS–138A	7A	711.7–710.7	— — —	NP	NP	SM
SS–138A	8A	709.7–708.7	– – – – – –	29	1	SM
SS–138A	9A	707.7–707.2	.....	28	2	SM
SS–138A	9B	707.2–706.7	—·—·—·—	25	2	SM
SS–138A	10A	705.7–705.2	—··—··—	22	1	SM
SS–138A	10B	705.2–704.7	— – – —	NP	NP	SM

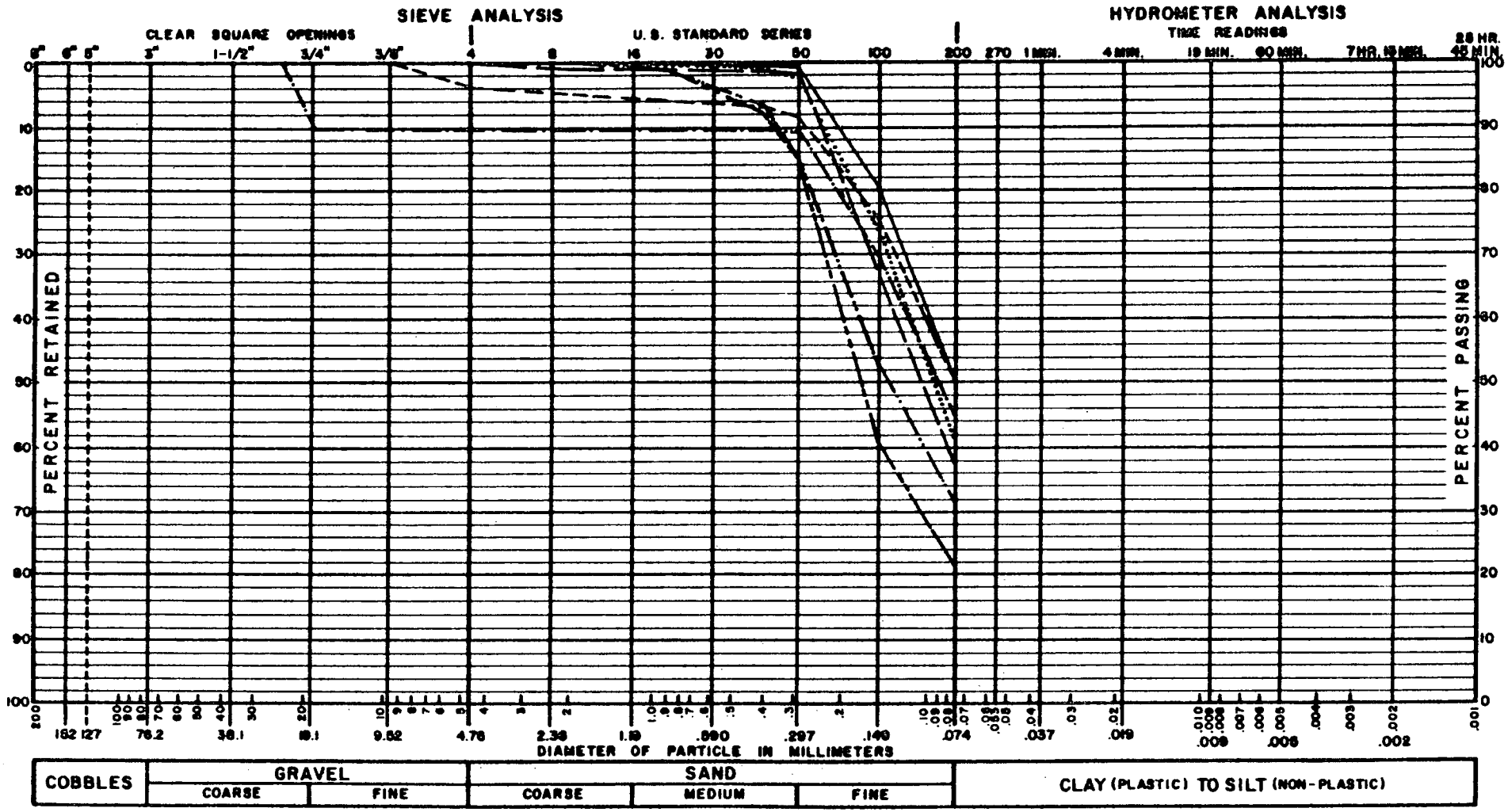
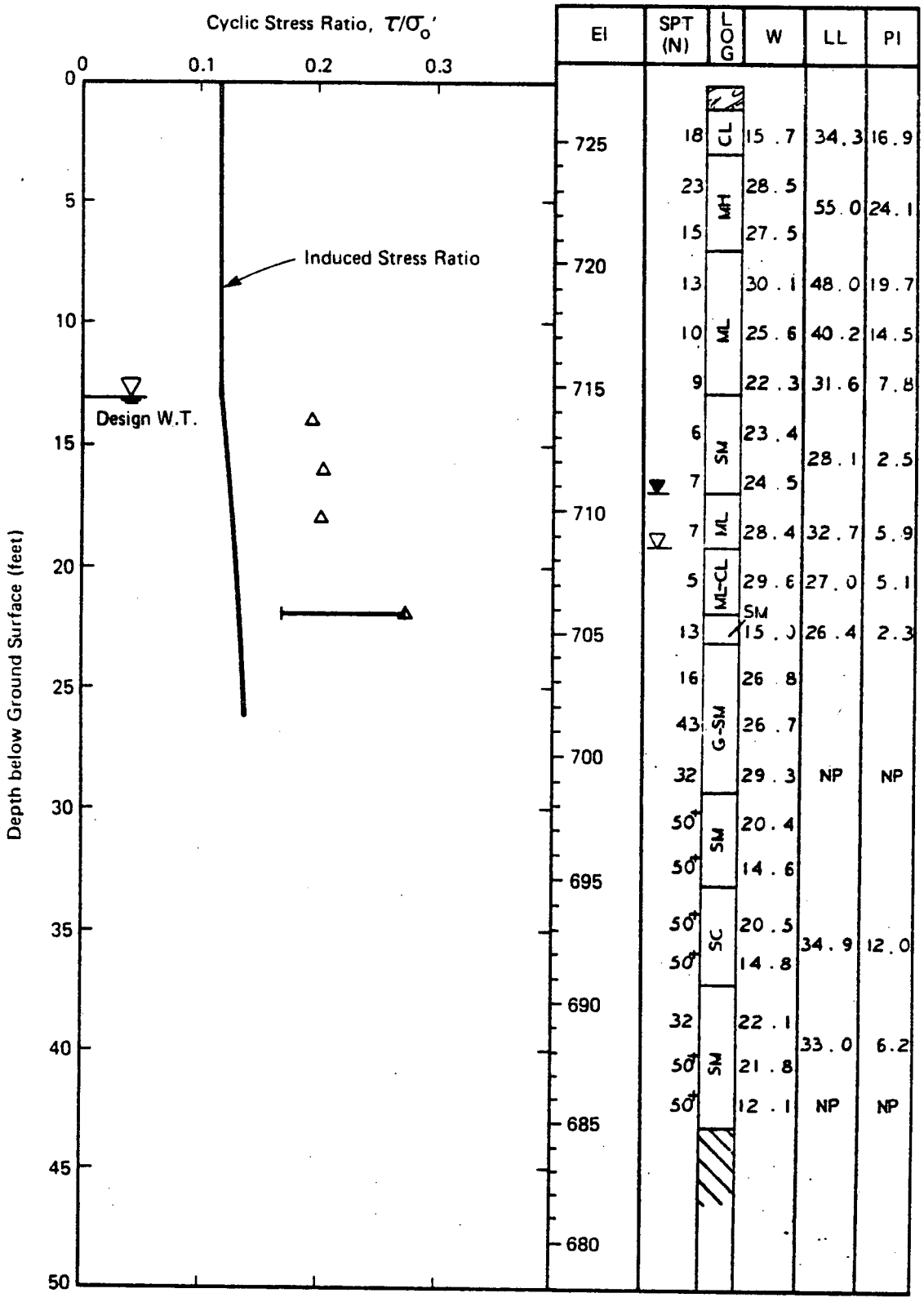


Figure 10 – Grain Size Distribution Curves for Silty Sand Samples from Boring SS–138A

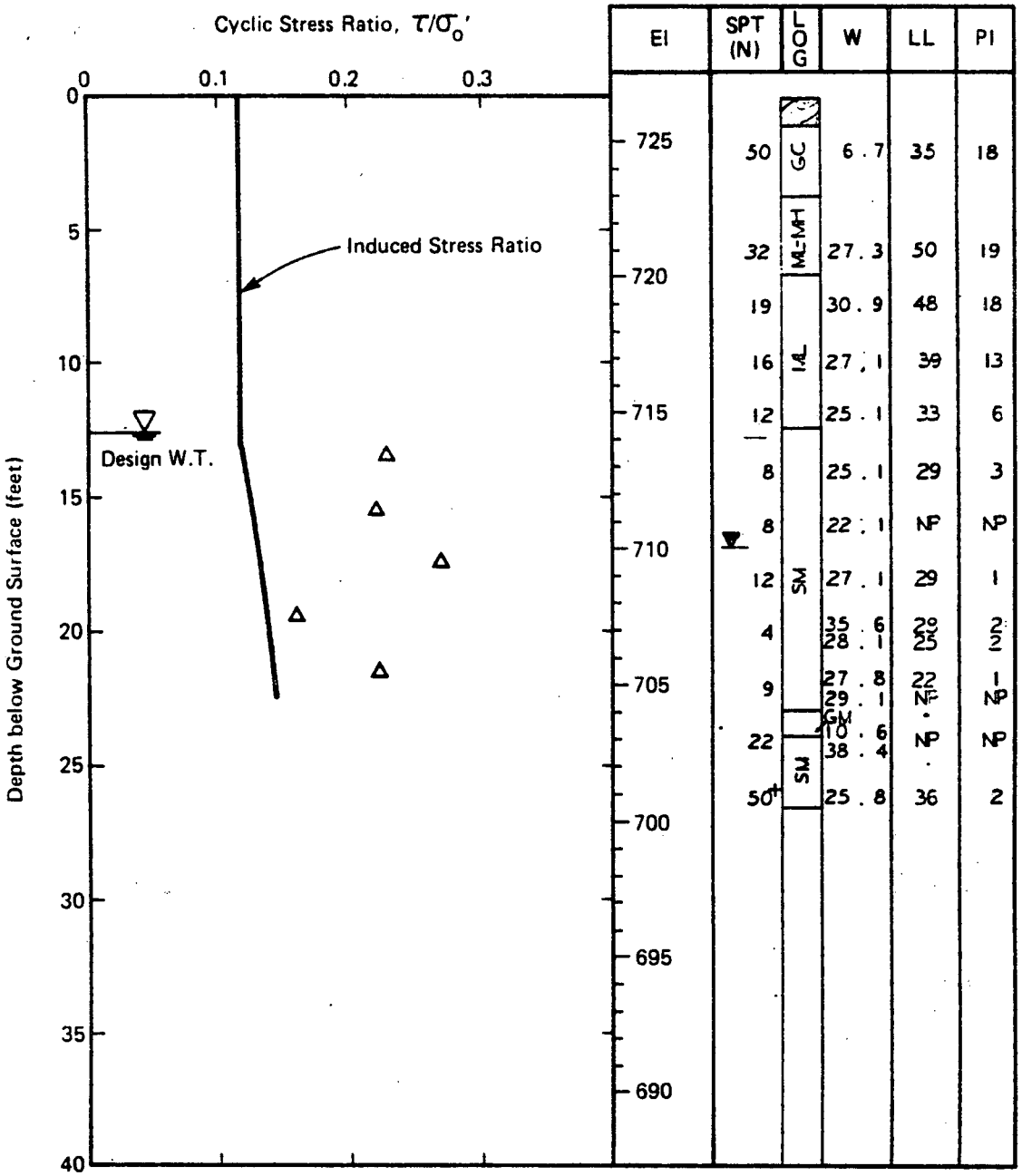


LEGEND

△ Estimated cyclic strength (cyclic stress ratio causing pore pressure ratio of 100%) based on N–values.

Possible range of cyclic strength

Figure 11 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Pore Pressure Ratio of 100 Percent – Boring SS–138



LEGEND

△ Estimated cyclic strength (cyclic stress ratio causing pore pressure ratio of 100%) based on N–values.

Figure 12 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Pore Pressure Ratio of 100 Percent – Boring SS–138A

BORING NO.	SAMPLE NO.	ELEV. (ft)	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
SS–161	GR7	719.6–717.6	————	NP	NP	SM
SS–161	Fine Particle Matrix (Removing sizes >#4)		— — —			

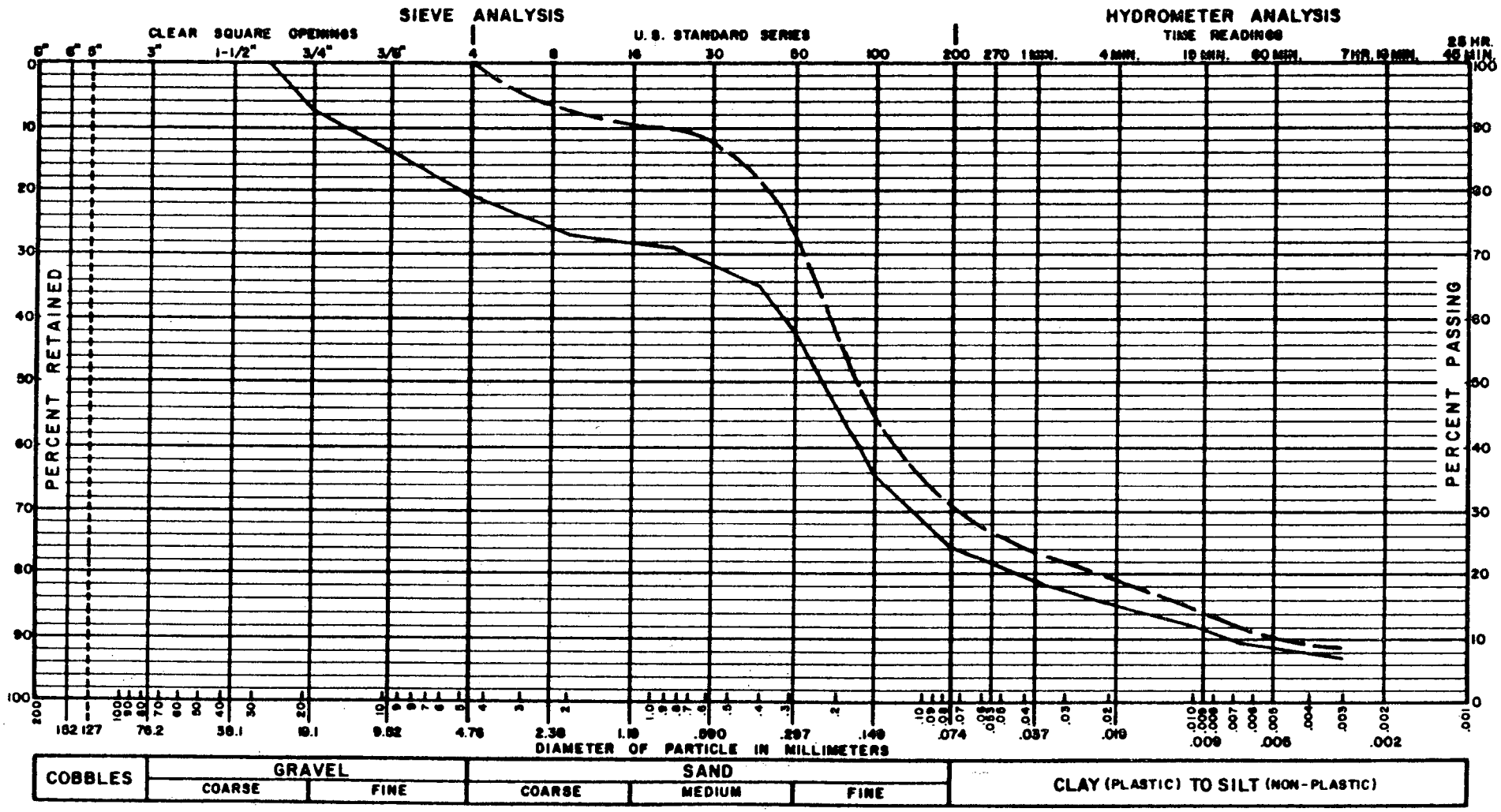


Figure 13 – Grain Size Distribution Curve for Silty Sand Sample from Boring SS–161

BORING NO.	SAMPLE NO.	ELEV. (ft)	SYMBOL	LIQUID LIMIT	PLASTICITY INDEX	UNIFIED CLASSIFICATION
SS-161A	6A	721.4–720.4	————	26	2	SM
SS-161A	7A	719.9–719.4	— — —	NP	NP	SM
SS-161A	7B	719.4–718.9	- - - - - -	NP	NP	SM

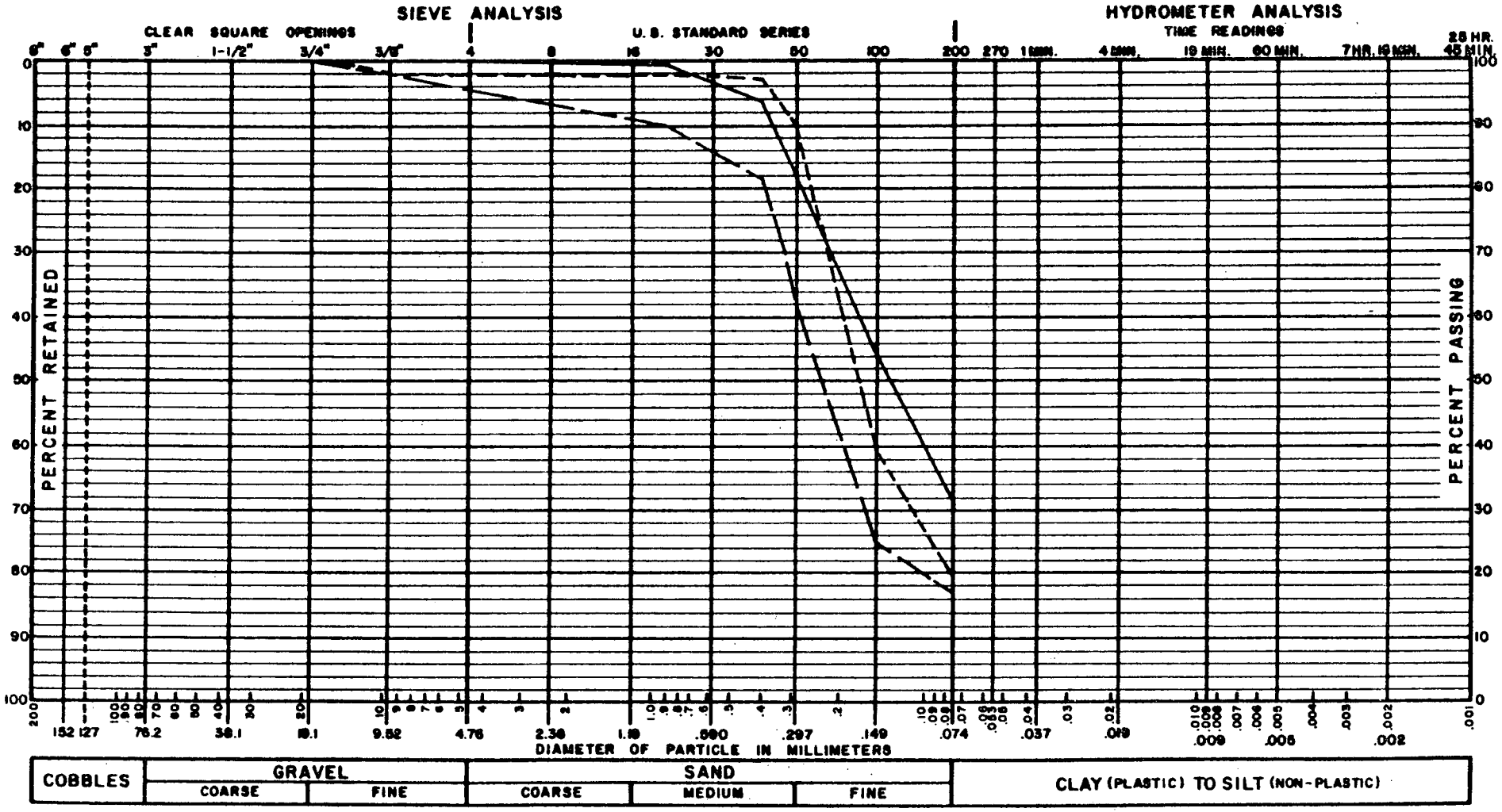
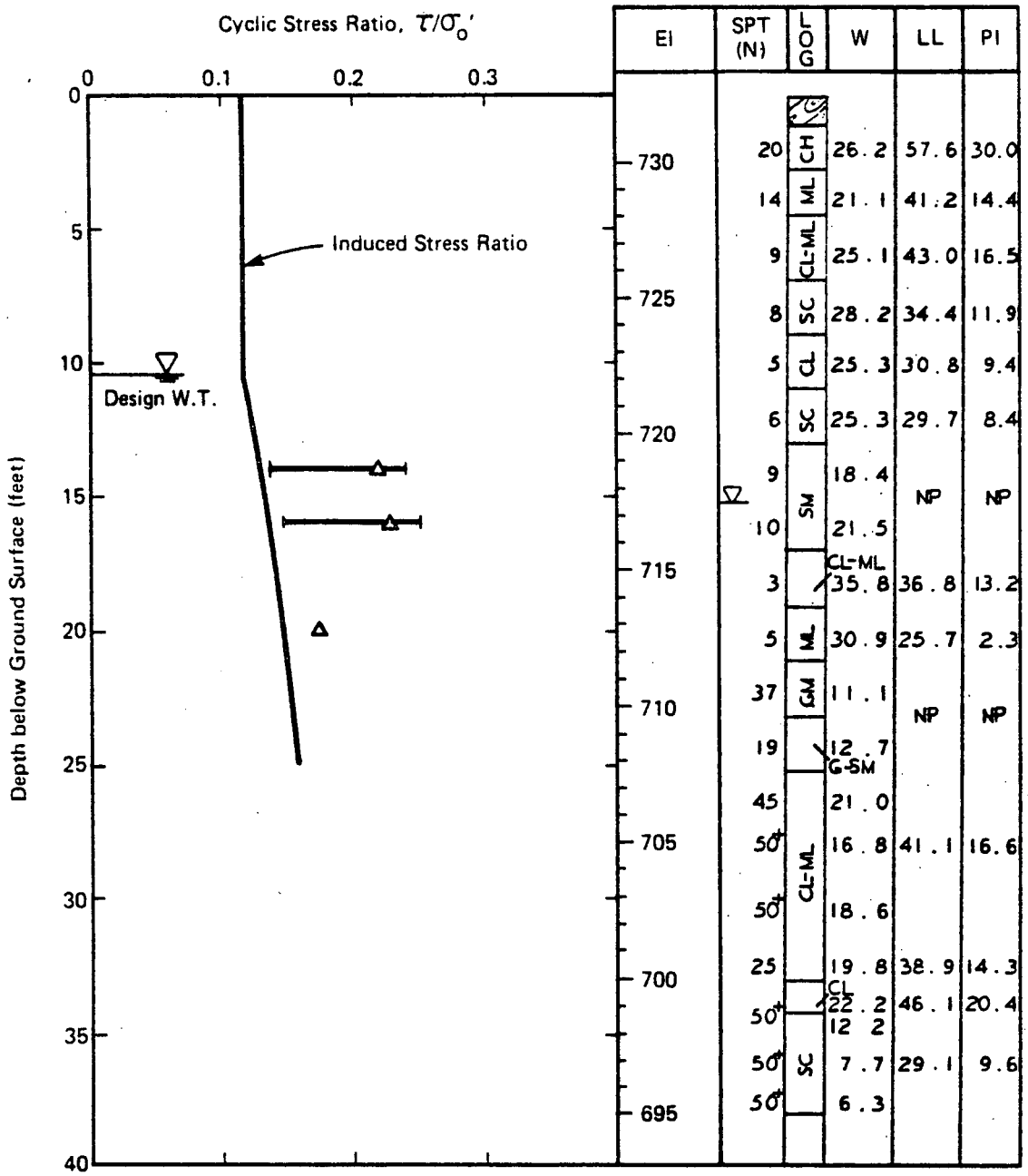


Figure 14 – Grain Size Distribution Curves for Silty Sand Samples from Boring SS–161A

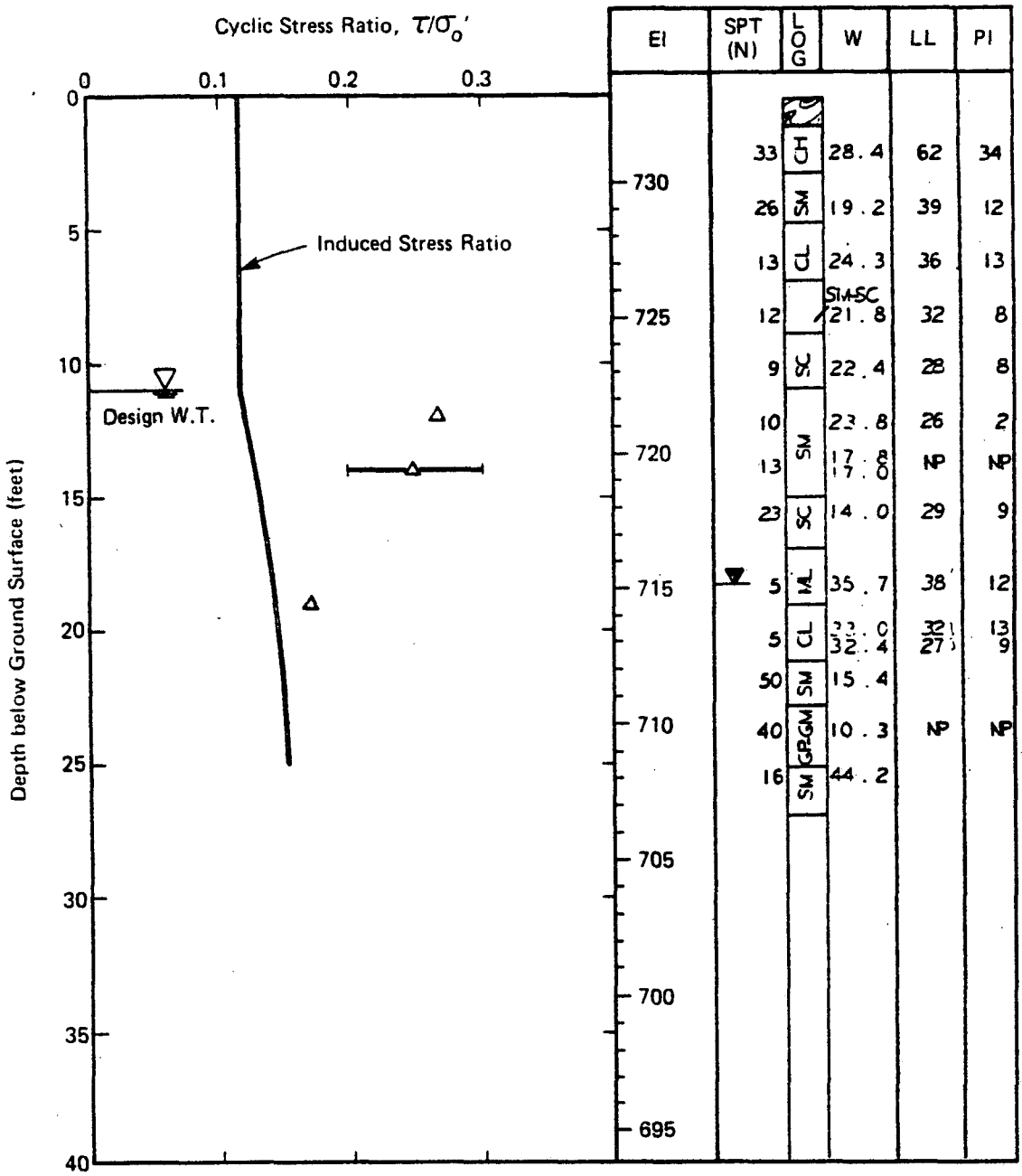


LEGEND

△ Estimated cyclic strength (cyclic stress ratio causing pore pressure ratio of 100%) based on N–values.

Possible range of cyclic strength

Figure 15 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Pore Pressure Ratio of 100 Percent – Boring SS–161



LEGEND

△ Estimated cyclic strength (cyclic stress ratio causing pore pressure ratio of 100%) based on N–values.

Possible range of cyclic strength

Figure 16 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Pore Pressure Ratio of 100 Percent – Boring SS–161A

APPENDIX A

ERCW PIEZOMETER DESIGN, INSTALLATION, AND DEVELOPMENT

A series of eight piezometers were installed along the ERCW pipeline route to establish the ground water level. The piezometers, designated P-1 through P-8, were installed between October 22, and November 2, 1981, at the locations shown in figure C-1. The piezometers served two purposes. Their primary purpose was to establish the ground water level at the time of soil sampling operations. The soil sampling, described in appendix C, used drilling mud which interferes with the determination of ground water in the drill hole. Their second purpose was to determine the ground water level along the entire pipeline route (drilling was not required along the entire route) and its fluctuation with time.

The details of the piezometer design are shown in figure A-1. The piezometers consist of 2-inch-diameter PVC pipe founded on rock and are composed of alternating 5-foot segments of slotted and nonslotted pipe. The piezometers were installed by making a boring with an 8-inch hollow-stem auger advanced to refusal. The PVC pipe was inserted down the interior of the auger (3-3/8-inch inside diameter) after the auger interior was cleaned. The auger was then removed and the boring backfilled with coarse sand to within approximately 5 feet of grade. A 6-inch layer of bentonite was then placed followed by a bentonite-soil mixture to grade.

The piezometers were developed by flushing with water to remove the fines. Backflushing continued until the water was reasonably clear. Compressed air was then used to blow the water out of the piezometer. This was continued until the hole was dry but not more than 2 hours. Details of the installation and development are given in table A-1.

The piezometers were read daily for a week after installation and then biweekly. Details of the ERCW piezometer ground water levels, rainfall, and lake level readings are summarized in table A-2. In addition, previously existing monitoring wells in the LLRW and general plant areas were read on the same schedule between October 26, and November 19, 1981. These readings are summarized in table A-3.

Table A-1

WATTS BAR NUCLEAR PLANT

ERCW PIEZOMETERS

INSTALLATION DETAILS

Location	P-1	P-2	P-3	P-4	P-5	P-6	P-7	P-8
Coordinates								
N-S	S1811.4	S1352.7**	S1337.7	S842.2	S344.4	N312.2	N657.0*	N561.2
E-W	E885.1	E875.1	E1452.8	E1868.4	E1862.4	E1799.4	E1156.2	E758.9
Surface El	712.0	724.8	727.3	730.2	735.3	719.3	722.0	734.2
Refusal El	677.0	691.8	687.3	685.2	700.3	687.8	697.0	702.2
Top of Pipe El	713.2	726.8	729.1	732.1	--	721.8	724.3	736.2
Bottom of Pipe El	677.0	691.8	687.3	685.2	--	687.8	697.0	702.2
Pipe Length, ft	36.2	35.0	41.8	46.9	--	34.0	27.3	34.0
Pipe Stickup, ft	1.2	2.0	1.8	1.9	--	2.5	2.3	2.0
Development time								
Flushing, min	15	20	30	15	--	20	20	20
Air blowout, min	120	120	180	75	--	90	120	120
Initial water readings								
1 h, el	695.0	708.5	707.7	707.0	dry	702.4	710.1	--
24 h, el	696.2	708.5	708.5	707.2	dry	702.4	710.0	715.1
Installation								
date began	10/27	10/29	10/22	10/23	10/26	10/26	11/2	10/30
date completed	10/28	10/29	10/22	10/23	10/26	10/27	11/2	10/30

*Offset 7' south from surveyed location (N664.0)
**Offset 4' north from surveyed location (S1356.7)

Table A-2

WATTS BAR NUCLEAR PLANT

ERCW - PIEZOMETERS

WATER LEVEL READINGS

Date 1981	P-1	P-2	P-3	P-4	P-6	P-7	P-8	Rainfall in.	Lake Level
Oct-22	--	--	--	--	--	--	--	0.00	--
23	--	--	--	--	--	--	--	0.55	--
24	--	--	--	--	--	--	--	0.30	--
25	--	--	--	--	--	--	--	trace	--
26	--	--	707.7	707.0	--	--	--	0.90	679.34
27	--	--	708.5	707.2	702.4	--	--	0.70	--
28	695.7	--	708.5	707.1	702.4	--	--	0.00	679.93
29	696.2	708.5	708.6	707.1	702.3	--	--	0.00	679.33
30	696.1	708.5	708.7	707.1	702.3	--	--	0.00	680.32
31	--	--	--	--	--	--	--	0.00	--
Nov 1	--	--	--	--	--	--	--	0.00	--
2	696.1	708.5	708.8	707.3	702.4	710.1	715.1	0.00	680.35
3	696.1	708.5	708.8	707.2	702.3	710.0	715.1	0.00	678.89
4	696.1	708.6	708.8	707.3	702.5	710.2	715.1	0.00	679.19
5	696.3	708.9	708.8	707.3	702.5	710.4	715.2	0.28	679.90
6	696.4	709.5	708.8	707.3	702.5	710.5	715.1	0.22	678.46
7	--	--	--	--	--	--	--	0.00	--
8	--	--	--	--	--	--	--	0.00	--
9	696.2	708.6	709.2	707.2	702.4	710.4	715.2	0.00	678.52
10	--	--	--	--	--	--	--	0.00	--
11	--	--	--	--	--	--	--	0.00	--
12	--	--	--	--	--	--	--	0.00	--
13	696.1	710.4	709.7	707.2	702.5	710.3	715.2	0.00	678.53
14	--	--	--	--	--	--	--	0.00	--
15	--	--	--	--	--	--	--	0.00	--
16	696.2	710.7	710.1	707.7	702.4	710.3	715.1	0.00	677.40
17	--	--	--	--	--	--	--	0.50	--
18	--	--	--	--	--	--	--	0.00	--
19	696.0	710.8	710.3	707.8	702.5	710.4	715.2	0.00	678.98
20	695.9	710.8	710.3	707.8	702.5	710.4	714.5	0.00	677.46
21	--	--	--	--	--	--	--	0.00	--
22	--	--	--	--	--	--	--	0.00	--
23	695.9	710.9	710.4	708.0	702.4	710.3	715.0	0.00	678.63
24	--	--	--	--	--	--	--	0.48	--
25	695.9	711.0	710.5	708.1	702.5	710.5	715.1	0.00	677.99
26	--	--	--	--	--	--	--	--	--
27	--	--	--	--	--	--	--	--	--
28	--	--	--	--	--	--	--	0.60	--
29	--	--	--	--	--	--	--	0.00	--
30	696.0	711.1	710.1	708.4	702.4	710.8	715.1	0.35	678.40

Table A-2

(Continued)

WATTS BAR NUCLEAR PLANT

ERCW - PIEZOMETERS

WATER LEVEL READINGS

Date 1981	P-1	P-2	P-3	P-4	P-6	P-7	P-8	Rainfall in.	Lake Level
Dec 1	--	--	--	--	--	--	--	1.89	--
2	--	--	--	--	--	--	--	0.00	--
3	695.9	711.4	711.1	708.5	702.7	711.2	715.2	0.00	679.85
4	--	--	--	--	--	--	--	0.00	--
5	--	--	--	--	--	--	--	0.00	--
6	--	--	--	--	--	--	--	0.00	--
7	--	--	--	--	--	--	--	0.00	--
8	695.9	711.4	711.3	708.8	702.5	711.0	715.2	0.00	679.41
9	--	--	--	--	--	--	--	0.00	--
10	695.9	711.5	711.3	708.9	702.5	710.9	715.3	0.00	679.53
11	--	--	--	--	--	--	--	0.00	--
12	--	--	--	--	--	--	--	--	--
13	--	--	--	--	--	--	--	--	--
14	695.8	711.0	711.3	709.0	702.5	710.9	715.3	0.72	678.48
15	--	--	--	--	--	--	--	--	--
16	--	--	--	--	--	--	--	--	--
17	695.8	711.0	711.4	709.1	702.6	711.2	715.3	0.20	679.67
18	--	--	--	--	--	--	--	0.00	--
19	--	--	--	--	--	--	--	0.00	--
20	--	--	--	--	--	--	--	0.00	--
21	695.7	711.0	711.3	709.1	702.5	711.1	715.3	0.00	679.20
22	--	--	--	--	--	--	--	1.05	--
23	--	--	--	--	--	--	--	0.00	
24	695.6	711.2	711.4	709.3	703.1	711.7	715.3	0.00	679.7
25	--	--	--	--	--	--	--	--	--
26	--	--	--	--	--	--	--	--	--
27	--	--	--	--	--	--	--	--	--
28	695.7	711.2	711.7	708.8	702.8	711.7	715.5	0.24	680.9
29	--	--	--	--	--	--	--	0.00	--
30	695.7	711.4	711.7	708.9	702.7	711.6	715.5	0.00	679.1
31	--	--	--	--	--	--	--	--	--

Table A-3

WATTS BAR NUCLEAR PLANT

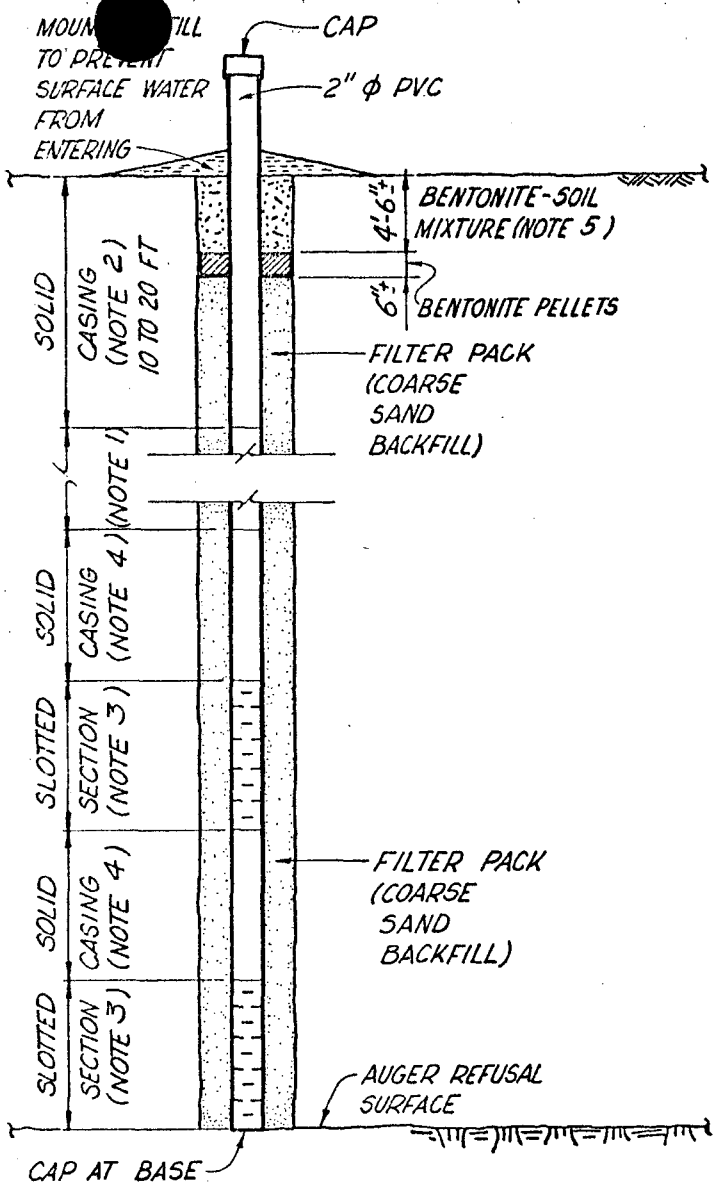
GENERAL

WATER MONITORING WELLS

WATER LEVEL READINGS

ASSOCIATED WITH ERCW STUDY

Date 1981	General Plant Area					LLRW Area		
	2	3	4	5	6	1	2	3
Oct 26	680.9	682.3	719.5	724.5	711.8	720.0	719.4	720.4
27	681.0	682.4	720.0	724.6	711.8	720.4	719.8	720.6
28	681.0	682.4	720.3	724.7	711.8	720.4	719.9	720.7
29	681.0	682.4	720.2	724.9	711.9	720.5	719.9	720.9
30	680.9	682.3	720.2	725.0	711.8	720.5	719.8	720.9
31								
Nov 1								
2	680.5	682.0	720.1	725.1	711.8	720.5	719.6	721.0
3	680.4	681.8	720.1	725.3	711.8	720.5	719.5	721.0
4	680.4	681.8	720.1	725.0	711.8	720.3	719.5	721.0
5	680.5	681.9	720.3	725.2	711.9	720.7	719.8	721.1
6	680.6	681.8	720.5	725.2	712.0	720.9	720.1	721.2
7								
8								
9	680.5	681.8	720.5	727.3	711.9	720.8	719.9	721.3
10								
11								
12								
13	680.3	681.7	720.2	725.1	711.9	720.5	719.6	721.2
14								
15								
16								
17								
18								
19	679.7	681.0	720.4	725.1	711.9	720.7	719.8	721.2



PIEZOMETER DETAIL
(NTS)

NOTES:

1. THE PIEZOMETERS SHALL BE INSTALLED IN SOIL BORINGS EXTENDED TO TOP OF ROCK (REFUSAL).
2. THE PIEZOMETER SHALL BE MADE OF ALTERNATING SECTIONS OF SLOTTED AND SOLID SECTIONS OF 2" ϕ PVC PIPE. A REMOVEABLE CAP SHALL BE PROVIDED ON THE TOP TO PREVENT RAINFALL FROM ENTERING THE PIEZOMETER.
3. THE SLOTTED SECTIONS SHALL CONSIST OF SLOTS SPACED ALONG A 5 FOOT LENGTH OF THE PIPE. THE SLOTS ARE TO BE AS FOLLOWS. ON OPPOSITE SIDES OF THE PIPE 2 TRANSVERSE SLOTS APPROXIMATELY 1½ INCHES BY 1/16 INCH ARE TO BE CUT. THE SLOTS ARE TO BE SPACED ON 3 INCH CENTERS WITH ALTERNATING SLOTS ROTATED APPROXIMATELY 90° DEGREES. THE LAST SLOTTED SECTION IS TO BE NO CLOSER THAN 10 TO 20 FT. OF GRADE.
4. THE SOLID SECTIONS SHALL CONSIST OF SECTIONS OF PVC PIPE WITHOUT SLOTS. THE LENGTH OF THESE SECTIONS IS DEPENDENT ON THE DEPTH OF THE BORING. IF THE BORING IS GREATER THAN 35 FEET DEEP THE SOLID SECTION IS TO BE 10 FEET LONG. FOR BORINGS LESS THAN 35 FEET DEEP THE SOLID SECTION IS TO BE ONLY 5 FEET. LONG.
5. THE PIEZOMETER SHALL BE BACKFILLED AS FOLLOWS. BACKFILL WITH COARSE SAND TO WITHIN APPROXIMATELY 5 FEET OF GRADE. ADD A 6-INCH LIFT OF BENTONITE PELLETS AND THEN A BENTONITE-SOIL MIXTURE TO GRADE. THE BENTONITE AND THE BENTONITE-SOIL LAYERS SHOULD BE TAMPED AS THEY ARE PLACED TO SEAL THE BORING. MOUND UP THE SOIL AT THE SURFACE OF THE BORING TO PREVENT WATER FROM ENTERING THE BORING.
6. EACH PIEZOMETER SHALL BE DEVELOPED AS FOLLOWS. BACK FLUSH THE PIEZOMETER WITH CLEAN WATER TO REMOVE FINES BY INSERTING AN (APPROX.) 1" LINE TO THE BOTTOM OF THE PIEZOMETER. THE WATER LINE NEEDS TO BE REASONABLY STIFF TO KEEP THE END OF THE LINE AT THE BOTTOM OF THE HOLE DURING BACKFLUSHING THE BACKFLUSHING SHOULD BE CONTINUED UNTIL THE WATER IS REASONABLY CLEAR. THEN USING COMPRESSED AIR PUMPED TO THE BOTTOM THROUGH A 1 INCH LINE BLOW THE WATER OUT OF THE PIEZOMETER. THIS SHOULD BE CONTINUED UNTIL THE HOLE IS DRY OR NOT MORE THAN 2 HOURS.

WATTS BAR NUCLEAR PLANT
FIGURE A-1

APPENDIX B

SEASONAL HIGH GROUND-WATER LEVELS

The basis for the projected seasonal high ground-water levels used in evaluating soil liquefaction potential beneath the ERCW pipeline is described in this appendix.

General Site Conditions

Ground-water monitor wells and piezometers at the plantsite are shown on figure B-1. Eight soil piezometers, P1 through P8, were installed in late October and early November 1981 along the pipeline specifically for the liquefaction analysis. Approximately one month of daily to twice weekly water-level measurements are available for the ERCW piezometers (except for P5 which was dry during this period) as shown in figure B-2. Ground-water level measurements for the same period are available for three monitor wells, A1 through A3, recently installed in the vicinity of the proposed low-level radwaste storage facility (figure B-2). Approximately nine years of ground-water level data exists for monitor wells, B1 through B6, as shown on figures B-2 and B-3. The long-term water level records for these latter wells form the basis for estimating seasonal high water levels in the new ERCW piezometers.

The water table generally slopes from the upland areas in the northern part of the site toward the reservoir, following site topography in a subdued manner (refer to figure B-1). Examination of long-term records of ground-water levels, precipitation, and reservoir stage indicates that the water table in the northern portion of the site fluctuates in response to rainfall, whereas reservoir stage appears to largely control water table fluctuations in areas closer to the reservoir. The amplitudes of seasonal fluctuations in up-gradient control wells, B4 and B5, are substantially greater than those of the down-gradient wells, B1, B2, and B3, located near the reservoir. In the absence of long-term water level data in the immediate vicinity of the ERCW pipeline, it is assumed that the amplitude of seasonal water table fluctuations in this area lies somewhere between the amplitudes observed in the up-gradient and down-gradient control wells.

Methods

Two methods of estimating seasonal high ground-water levels at the ERCW piezometers were used. Both methods involve adding to the current water level in each piezometer an increment determined by linear interpolation of historic seasonal high water levels observed in selected up-gradient and down-gradient control wells. Water table elevations measured on November 13, 1981, were selected as the current or baseline water table elevations. Maximum water levels recorded in the control wells in March 1979 (March 8), were considered representative of seasonal high water table conditions.

In the first method, the seasonal high water level at each ERCW piezometer is computed as,

$$y_p = ca'/a + d$$

where y_p is the estimated seasonal high ground-water level at piezometer p; c is the difference between the March 8, 1979, water levels at the up-gradient and down-gradient control wells; a is the difference between the November 13, 1981, water levels at the piezometer and the down-gradient control well; a' is the difference between the November 13, 1981, water levels at the up- and down-gradient control wells; and d is the March 8, 1979 water level at the down-gradient control well. Basically, using this method, all piezometers lying on the same (November 13, 1981) water table contour increase in response to precipitation or reservoir changes by the same amount. The amount of increase at a piezometer is proportional to its current water table elevation in relation to current water table elevations at the control wells. Implicit in this method is the assumption that water level fluctuations in areas of relatively low water table elevation are more closely related to reservoir stage, regardless of distance from the reservoir. Likewise, water table fluctuations in areas of relatively high water table are related more to precipitation events, regardless of location.

Four pairs of up-gradient and down-gradient control wells are used in method 1, and results were averaged. Wells B4 and B5 are used as up-gradient control points, and wells B2 and B3 are used as the down-gradient control points. Since the B3-B5 well pair lies closer to the ERCW pipeline area, additional weight was given to results using these wells in the averaging process.

The second method of estimating seasonal high water levels at the ERCW piezometers is similar to the first, except that in linearly distributing the observed water level increases between control wells, the incremental increase assigned to each ERCW piezometer is a function of its relative position between control wells. Only control wells, B3 and B5, are used in this analysis, since they represent the only well pair constituting a line which traverses the ERCW pipeline area. Each piezometer location is projected orthogonally to the line between B3 and B5 to determine its relative position between the two control wells. The seasonal high water level for each piezometer is then computed as,

$$y_p = (y_u - y_d)\ x/x_t + y_d + y_o$$

where y_u is the difference between the March 8, 1979, and November 13, 1981, water levels at the up-gradient control well (B5); y_d is the difference between the March 8, 1979, and November 13, 1981, water levels at the down-gradient control well (B3); x is the distance between well B3 and the normal projection of piezometer p onto the line between B3 and B5; x_t is the total distance between wells B3 and B5, and y_o is the November 13, 1981, water table elevation at piezometer p. The line between B3 and B5, over which the interpolations are made, trends roughly normal to the reservoir shoreline. Thus, the second method implicitly assumes that the influence of the reservoir on ground-water levels is inversely proportional to distance from the reservoir.

The geohydrologic assumptions inherent in both methods are admittedly oversimplified. Factors such as the hydraulic properties of geologic materials and land slope also influence water table responses and are not directly accounted for by either method. However, the relative importance

of each controlling factor cannot be determined with the available information. In view of these limitations, the approaches that have been adopted appear reasonable and justifiable.

Results

The projected seasonal high ground-water levels at the ERCW piezometers estimated by methods 1 and 2 are presented in the table B-1. Note that since piezometer P5 was dry during the measurement period, the November 13, 1981, water level was assumed to be at elevation 699 feet or 1 foot below the bottom of the piezometer for purposes of estimating a seasonal high water level at this location. As a result of the simplifications and uncertainties associated with these analyses, and the fact that the results obtained by both methods are basically similar, we recommended simply averaging the seasonal high water levels predicted by these two methods. The recommended design ground-water levels at the ERCW piezometers are given in the last column of the table B-1 and are plotted in profile along the pipeline route on figure B-4. A projected seasonal high water table contour map is shown in figure B-5 for purposes of estimating water table elevations at intermediate points along the pipeline.

TABLE B-1

Summary of Estimated Seasonal High Ground Water Elevations

Well	Range of Water Table October 26 to December 30, 1981	Method 1					Method 2	Recommended Design G. W. Levels**
		B3-B5	B2-B5	B3-B4	B2-B4	Weighted Mean*		
P1	695.7 - 696.4	702.3	701.6	700.1	699.3	701.1	701.5	701
P2	708.5 - 711.5	718.0	717.6	713.6	713.2	716.1	716.2	716
P3	707.7 - 711.7	717.2	716.9	712.9	712.6	715.4	715.3	715
P4	707.0 - 709.3	714.5	714.1	710.5	710.1	712.7	713.1	713
P5	dry	705.5	704.9	702.8	702.1	704.2	705.4	705
P6	702.3 - 702.8	709.3	708.8	706.1	705.5	707.8	709.3	709
P7	710.0 - 711.7	717.9	717.5	713.2	713.5	716.0	717.8	717
P8	714.5 - 715.5	723.2	723.0	717.8	718.1	721.1	722.7	722

*B3-B5 estimate weighted double because it lies closer to ERCW piezometers.

**Average of Method 1 weighted mean and Method 2 results.

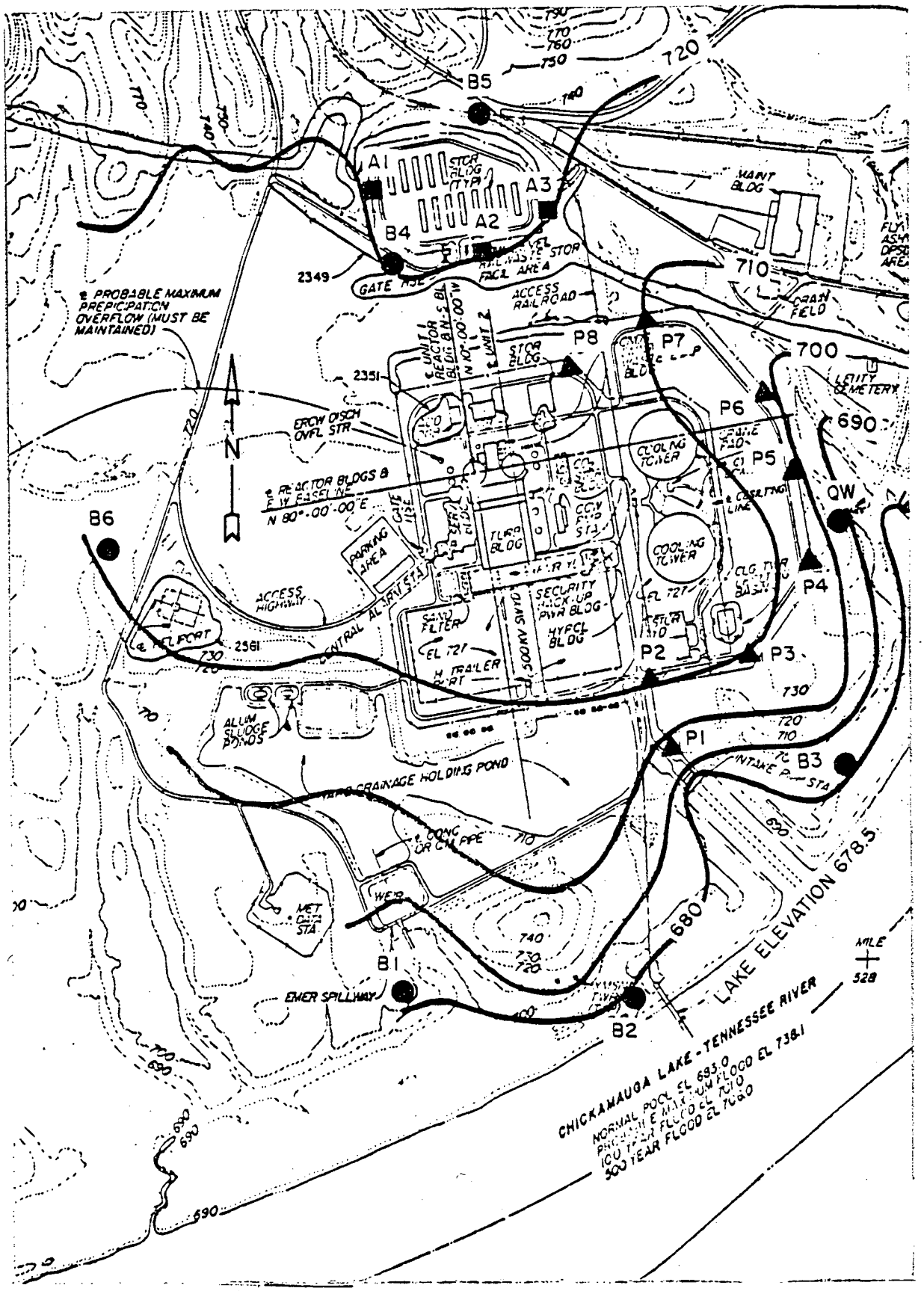


Figure B-1: Well Locations and Water Table Elevation on Nov. 13, 1981

WR2 85-103.1

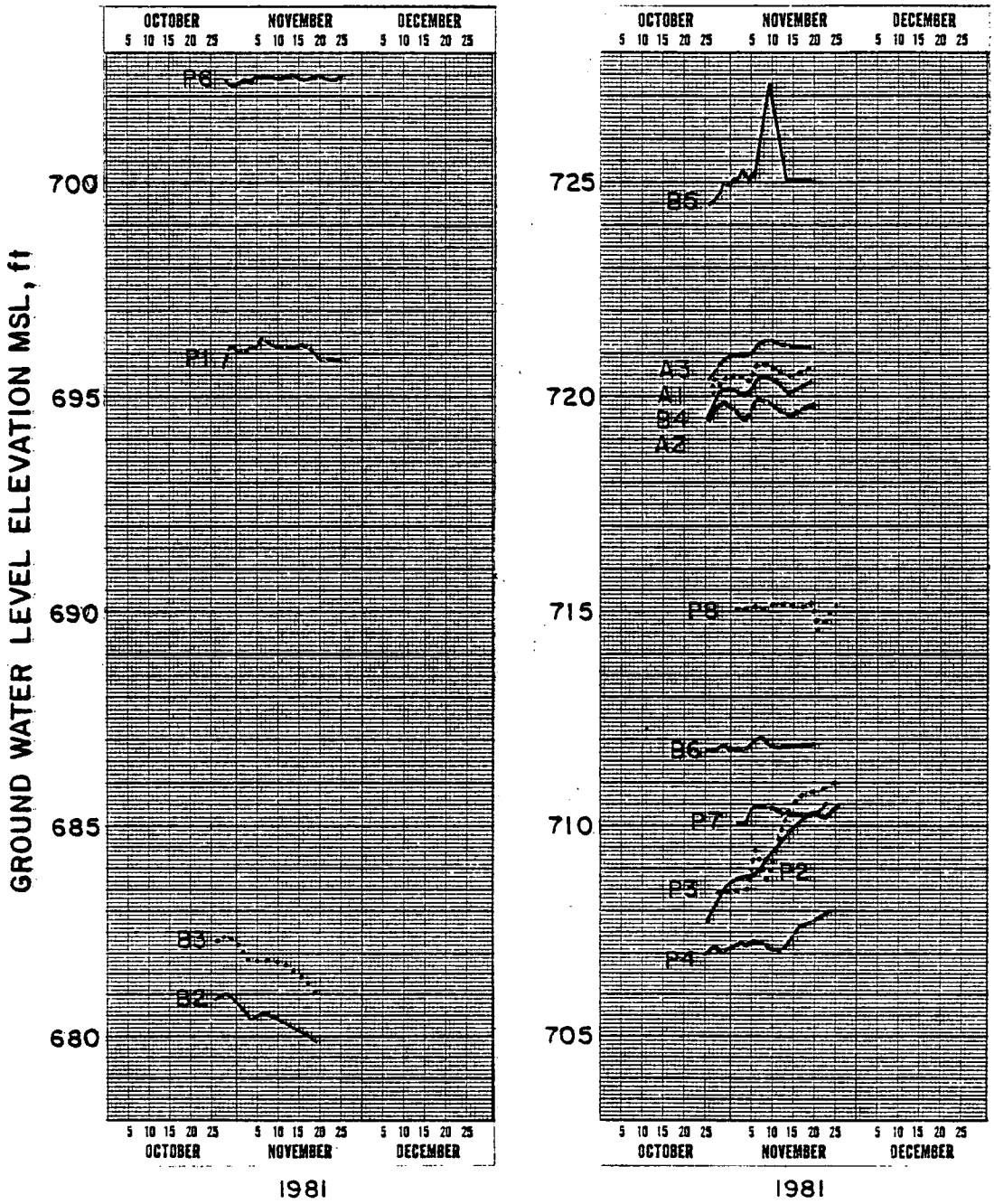


Figure B-2 : Well Hydrographs

WR2 8J-103.2

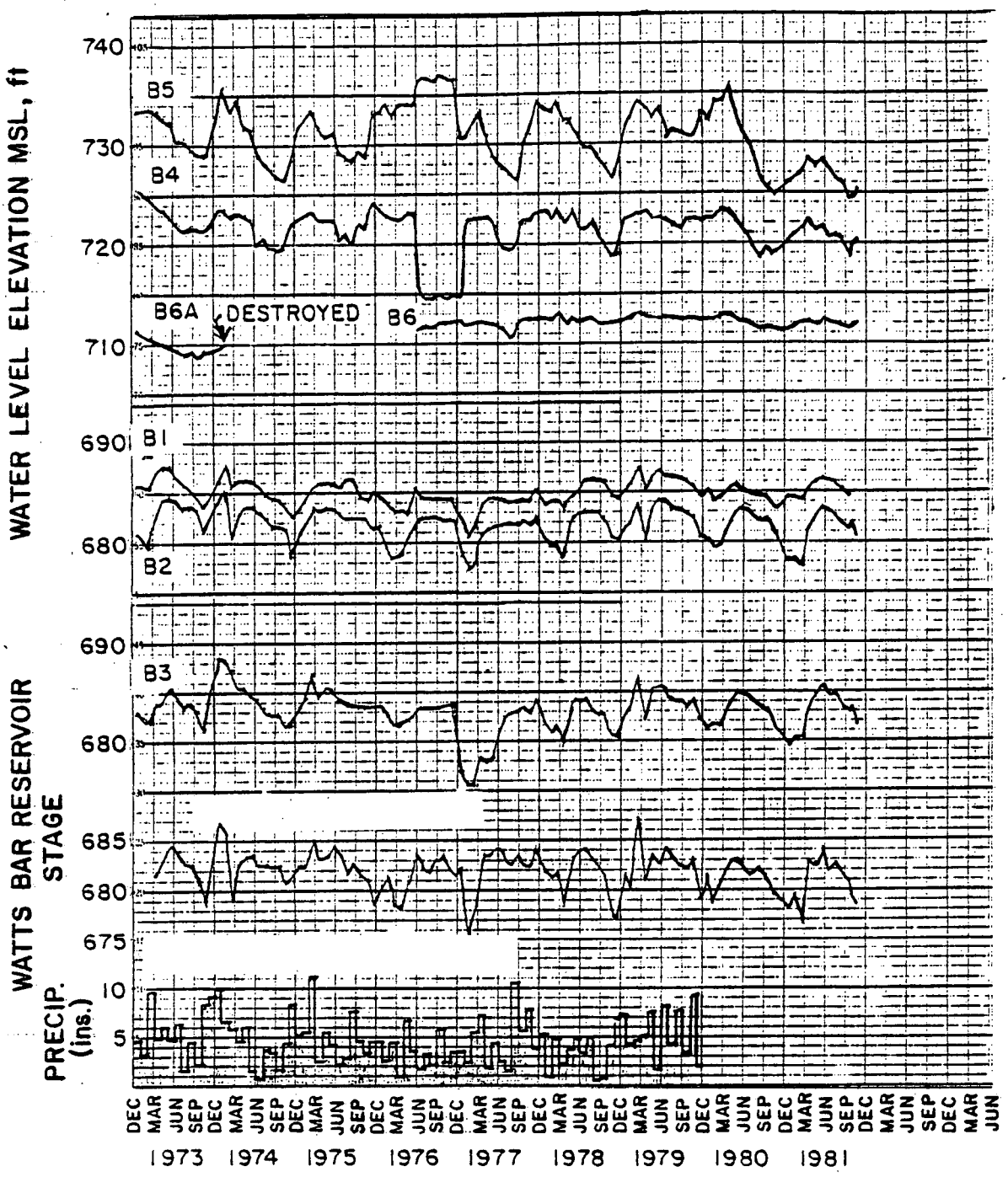


Figure B-3 : Long Term Ground Water Level Records

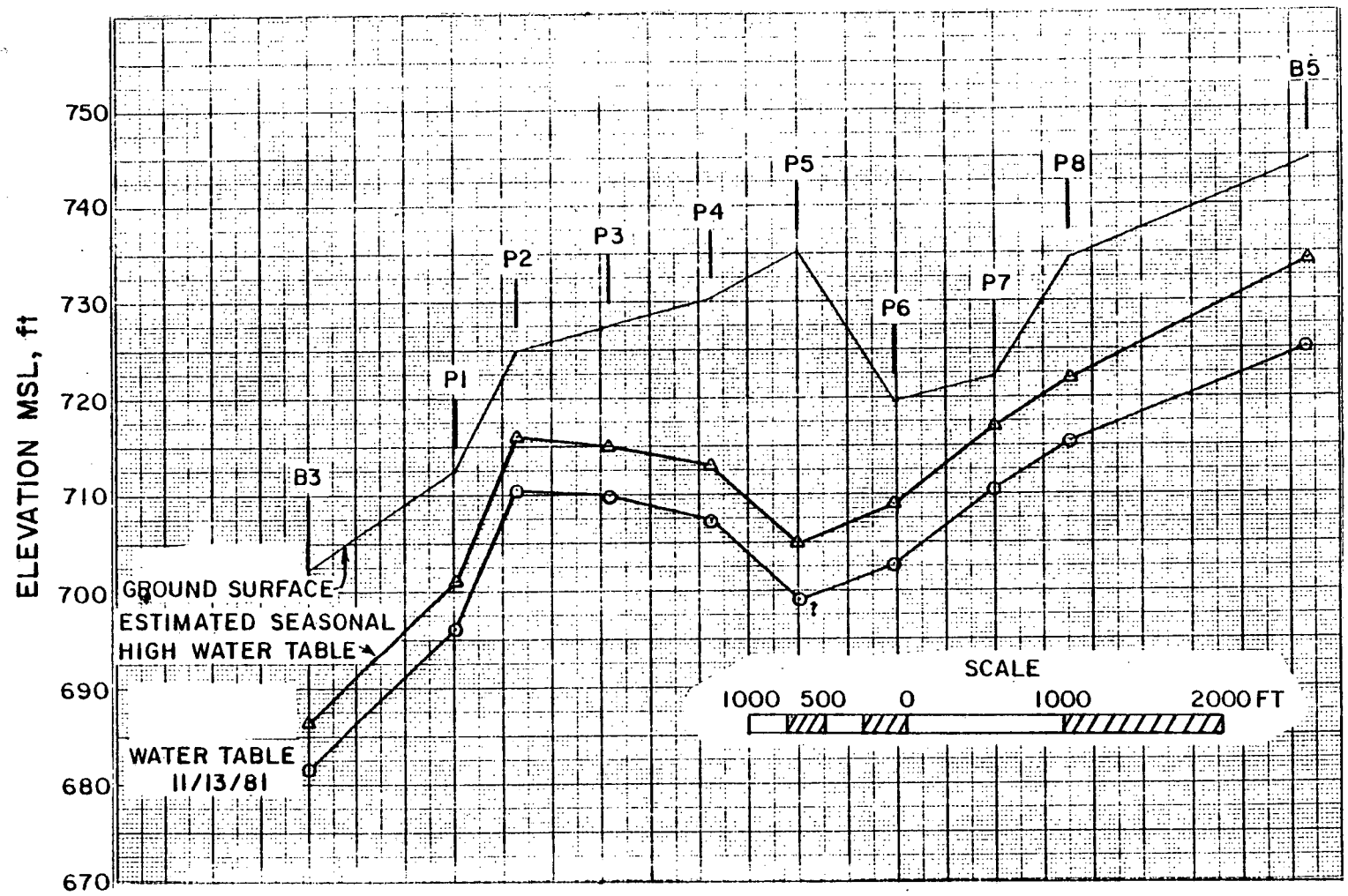


Figure B-4: Water Table Profiles

APPENDIX C

DESCRIPTION OF SOIL DRILLING AND REDRILLING AND SAMPLING PROGRAM

The soils along the ERCW pipeline route have been sampled in three major investigation programs. These were performed in 1975, 1979, and 1981. All sampling locations are shown in figure C-1.

The first investigation was carried out between July 24, and August 19, 1975, using two Mobile Drills. It consisted of 45 standard penetration (split-spoon) and 14 undisturbed borings. These borings are identified as 49 through 101. This investigation was completed prior to construction of the pipeline.

The second investigation was carried out between May 3, and July 3, 1979. It consisted of 41 standard penetration test borings. Three drills, CME models 55 and 75 and a Mobile model B-50 were used. These borings are identified as 131 through 170. This investigation was completed after construction of the pipeline.

A third investigation was carried out between November 4, and 24, 1981, using a Mobile B-61. It consisted originally of 10 standard penetration test borings. Later 4 additional borings were requested in the vicinity of borings 138A and 143A. This was a redrilling program for previously drilled holes to determine the impact of a different drilling procedure. These holes are identified by letters A, B, and C, following the original boring number and range from 49A to 163A.

The first two investigations were performed by dry drilling techniques utilizing hollow stem augers to advance and case the hole. To resolve questions surrounding this sampling technique, the third investigation was performed utilizing rotary drilling equipment and drilling mud. A description of the drilling procedures and a comparison of the results are given below.

Comparison of Boring Procedures

The dry drilling techniques utilized in the 1975 and 1979 investigations conformed to the procedures specified by ASTM D 1586. Hollow-stem augers having diameters of either 3-3/8 inches or 6 inches were mechanically advanced by rotary drilling equipment in 2-foot increments to the desired sampling depths. After the auger was cleared in place of any accumulated soil intrusion, the split-spoon sampler was inserted through the stem to the test elevation. The sampler was driven 18 inches into the soil by a 140-pound hammer falling 30 inches, hoisted using a single wrap of rope on the drill rig cathead. The number of blows required to drive the sampler each 6 inches of the 18-inch total was recorded. The number of blows required to drive the sampler the final 12 inches was reported as the standard penetration resistance, N. The auger is then advanced the 18 inches of the drive plus 6 inches more and the process repeated. Water or drilling mud was not used to counteract hydrostatic uplift below the water table.

The 1981 investigation was conducted using procedures specified in attachment C-1. Specifically, sample borings were advanced with a Mobile B-61 drill using rotary methods. Revert drilling fluid was maintained at a marsh funnel viscosity 40. The drilling mud level was maintained at the ground surface. Drag bits were equipped with baffles to deflect the drilling fluid upward. Upon achieving the desired depth, the auger was withdrawn, the hole cleared of auger cuttings as appropriate, and the split-spoon sampler inserted to the test elevation. Driving proceeded as previously described except the hoist rope was wrapped twice on the drill rig cathead. This procedure is also in conformance with ASTM D 1586.

Examination of adjacent borings performed using dry and mud drilling techniques indicates little significant difference between the data associated with each such as sample retrival and disturbance. Overall, the modification in procedure resulted in an increase in blow counts. Since the mud-drilling procedure was closely adherred to, it is reasonable to conclude that at this site the dry procedure also yields results satisfactory for engineering purposes. A tabular summary of the differences between the two procedures is given in attachment C-2.

Soil Profile

Subsoils along the drilled ERCW alignment shows a six-tiered profile which can be generalized as follows:

1. Terrace alluvial clays or silts

2. Terrace alluvial silty sands

3. Terrace alluvial gravels

4. Laminated residuum (N 30)

5. Weathered shale (N 30)

6. Bedrock

Exceptions to this profile occur locally where the profile is either more complex or simple. This profile was based on data obtained through all three investigations (1975, 1979, and 1981). However, comparison of these additional 1981 borings, which are approximately 5 feet from the 1979 borings, shows variations in soil types over a short distance. (See the graphic logs for details.)

The southern portion of the line, inclusive of borings SS-49A through SS-138A, exhibit a profile of apparent continuity. From the surface, cohesive clay, silt fill, or alluvium overlie sandy silt which grades downward into alluvial silty sand. Locally, a thin basal alluvial gravel rests on the residual weathered shale in which the borings were discontinued. From borings SS-143A through SS-158A, the profile is more complex, including more cohesive fine-grained soils. Borings SS-161A and SS-163A on the north portion of the line have a profile similar to the southern portion, but with only a relatively thin sand layer.

Based on number of blows, N, the surficial cohesive fill and alluvial soils typically show stiff to very stiff consistencies and the sand loose to

medium relative densities. Overall, the modifications in procedure resulted in an increase in blow counts as compared to the companion borings drilled using dry procedures.

Due to the use of the revert drilling fluid, water level readings in the 1981 SPT borings were not realistic. Therefore, interpolated water levels based on measurements taken in the adjacent or surrounding piezometers are shown on the graphic logs. Ground-water levels varied from elevation 693.0 at SS-49A to elevation 715.0 at SS-161A and SS-163A. From SS-134A through SS-138A, water levels were near elevation 711.0.

All sampling locations are shown in figure C-1. Graphic logs of borings 49 to 101 are shown in Watts Bar FSAR figures 2.5-196 to 2.5-201. Graphic logs of borings 131 to 170 are shown in Watts Bar FSAR figures Q362.36-2 to -9. Condensed graphic logs of the redrilled holes are shown in figure C-2. In general, the redrill locations are offset approximately 5 feet from the original boring. Figures C-3 to C-12 give the exact relationship of each boring where redrilling was performed. A comparison of the original and redrill boring graphic logs are given in figures C-13 to C-24. Borings 138A and 143A were redrilled to confirm selected portions of the soil profile. These repeated redrills are designated as 138B, 138C, 143B, and 143C and are generally offset 10 feet on each side of 138A and 143A. The graphic logs of these borings are given in figures C-19 and C-21. Figure 16 shows a profile of the ERCW pipeline. The redrilled borings have been inserted in the profile adjacent to the original boring. Figures C-17 and 18 show a generalized profile of the borings in the vicinity of the main plant area and the Diesel Generator Building.

ATTACHMENT C-1

RECOMMENDED PROCEDURES AND GUIDELINES FOR STANDARD PENETRATION TESTING WATTS BAR NUCLEAR POWER PLANT

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 guideline, 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 inches.

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 be driven for the full 18 inches. A record of the blows for each 6 inches 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). Jars shall be labeled to identify the location and position of the sample pieces in the sampler.

RECORD KEEPING

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-inch sheet of paper showing the entire sample length. Information to be shown thereon includes:

1. Total length of drive of the sampler (usually 18 inches).

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 inches of drive.

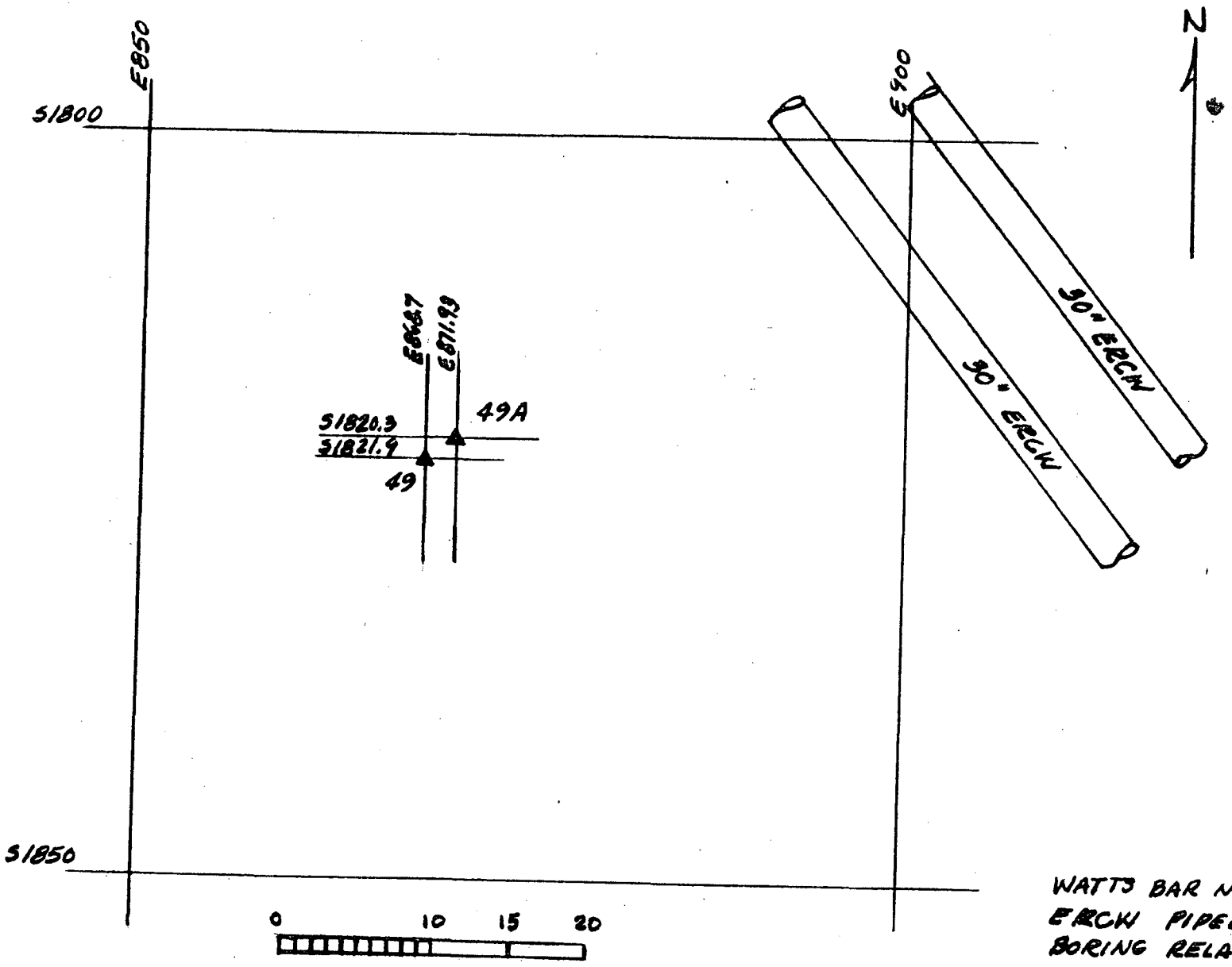
5. The description and classification of the sample along its length (different segments may have different description and classifications if changes in soil type occur in the sample).

6. Identification of the jars containing the pieces of the sample.

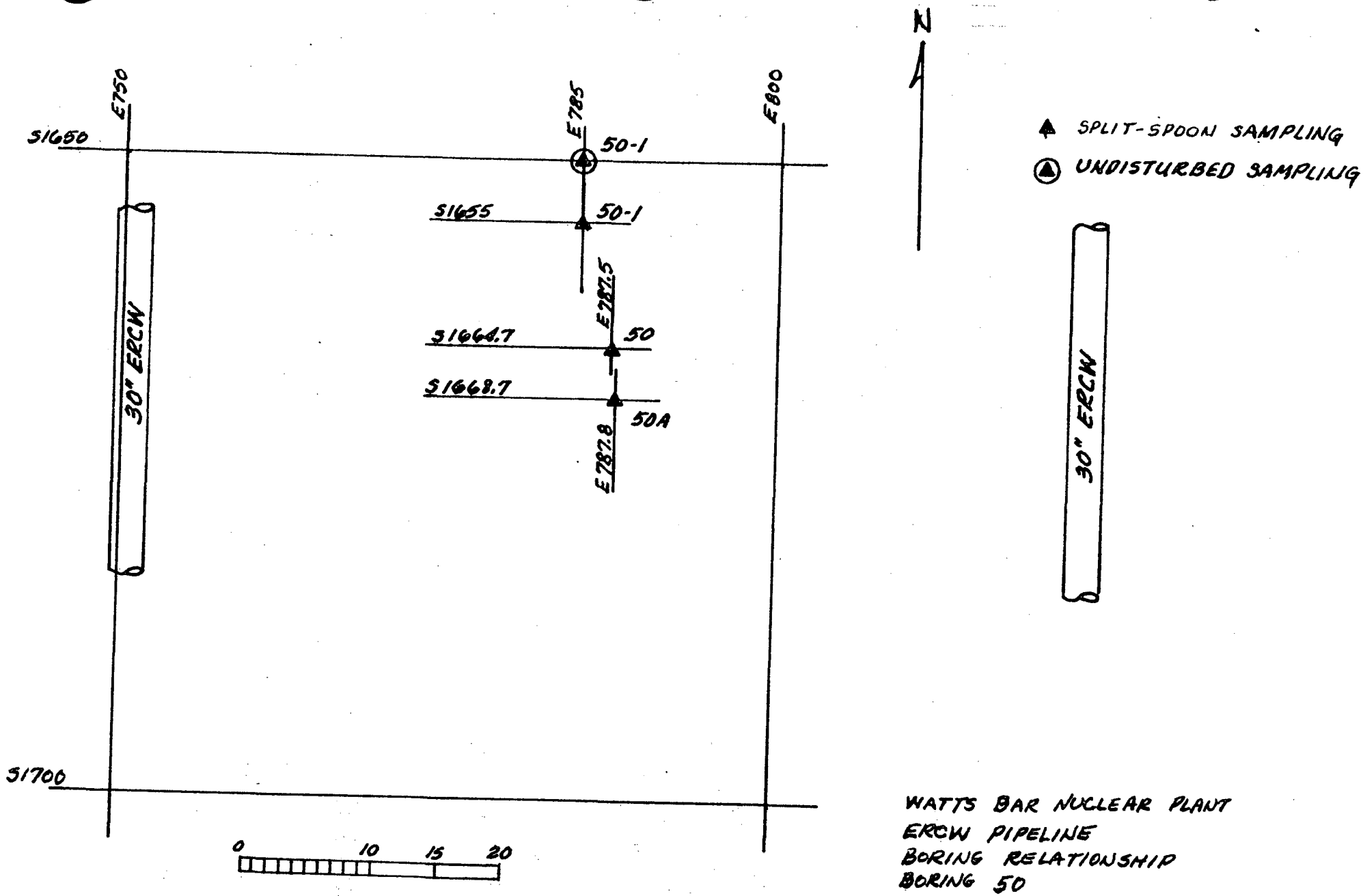
ATTACHMENT C-2

DIFFERENCES IN PROCEDURES AND TECHNIQUES BETWEEN INITIAL DRILLING AND REDRILLING

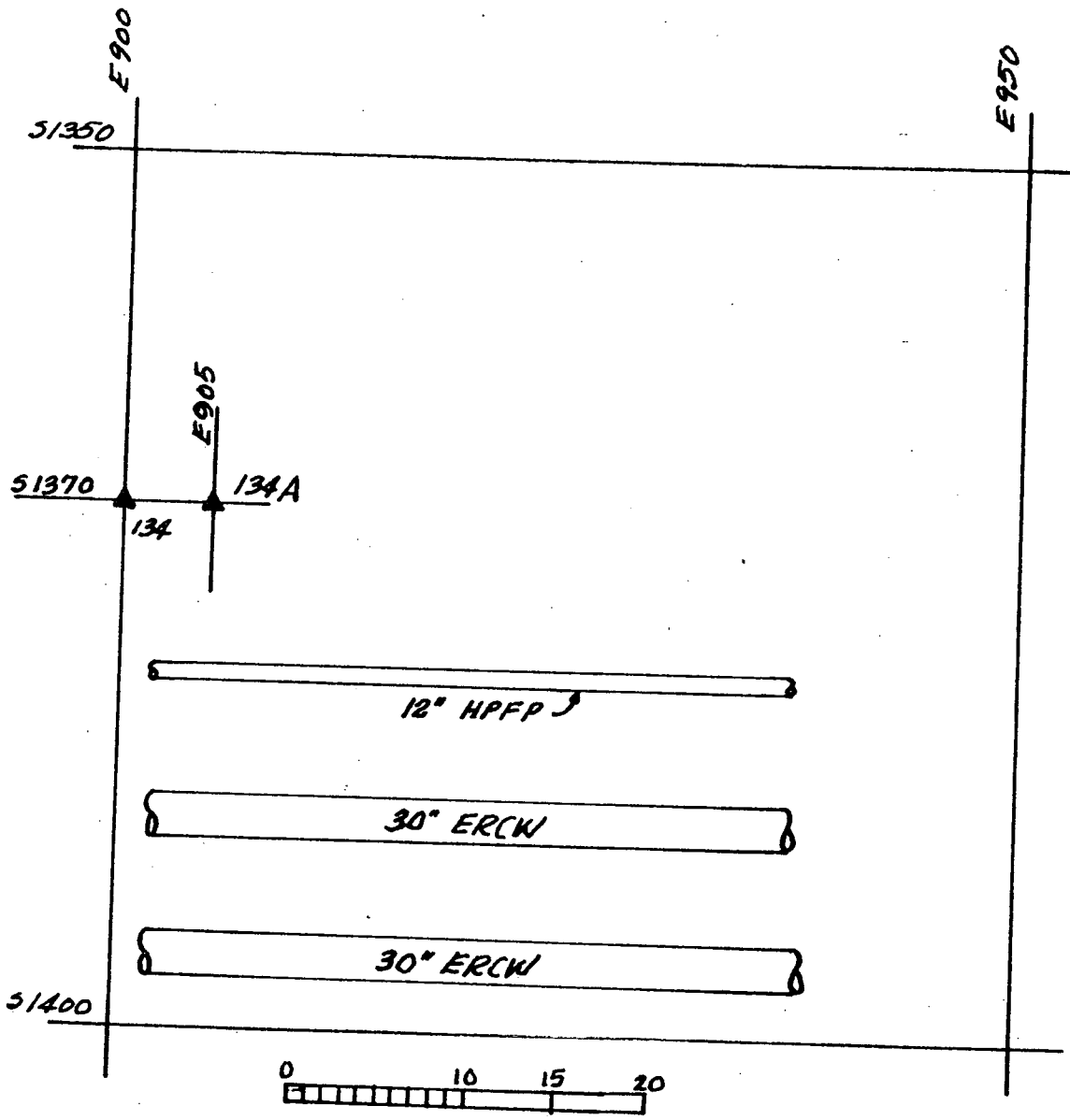
Item	Initial Drilling	Redrilling
Drilling	Hollow-stem auger using dry methods (acts as casing)	Rotary drilling with drilling mud (no casing)
Hole Diameter	6-7 inches	4-5 inches
Advancement	Hollow-stem auger	Fishtail bit with deflectors
Sampling	Split-spoon (per ASTM D 1586) with one wrap on cathead	Split-spoon (per ASTM D 1586) with two wraps on cathead



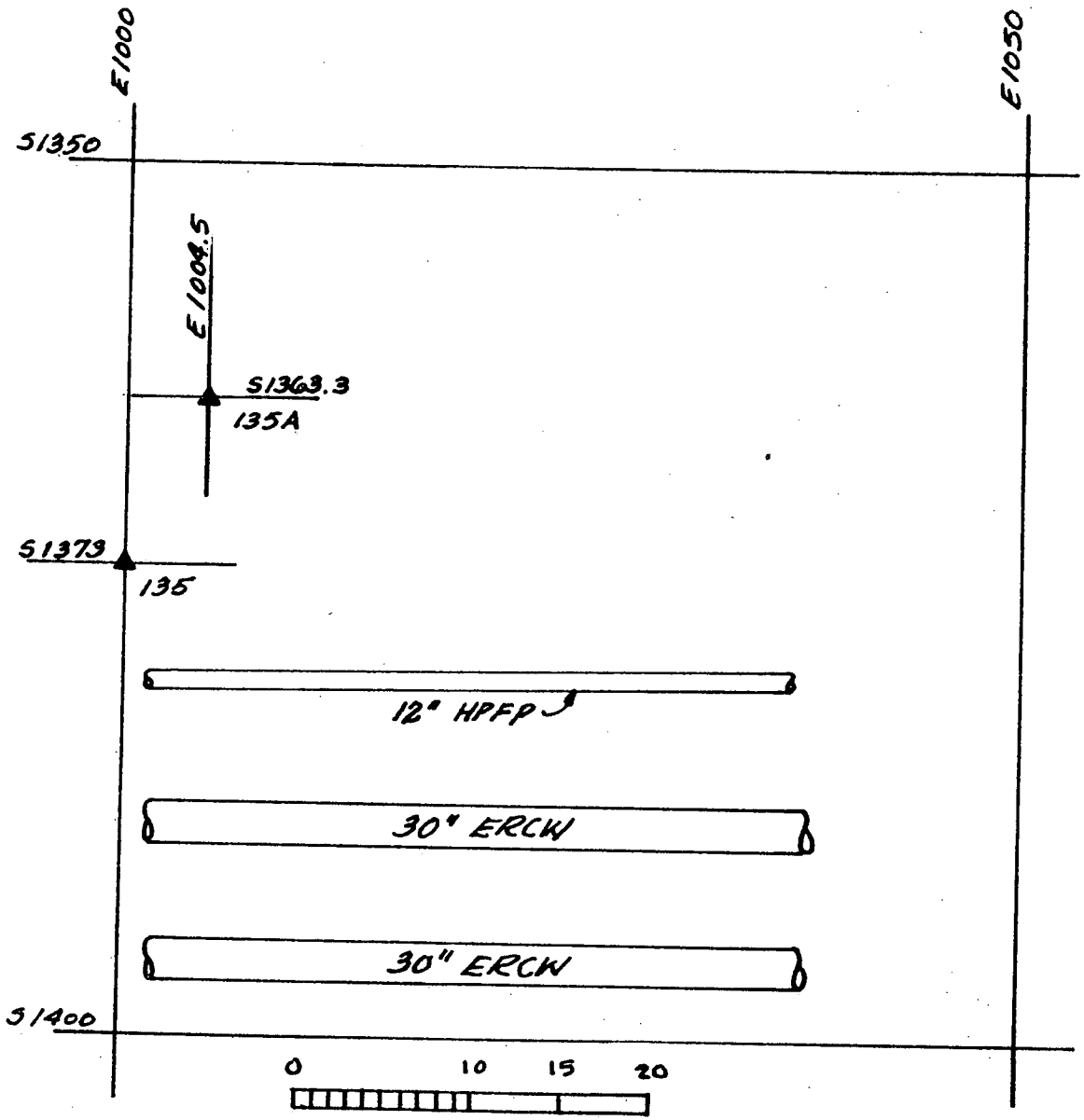
WATTS BAR NUCLEAR PLANT
ERCW PIPELINE
BORING RELATIONSHIP
BORING 49
FIGURE C-3



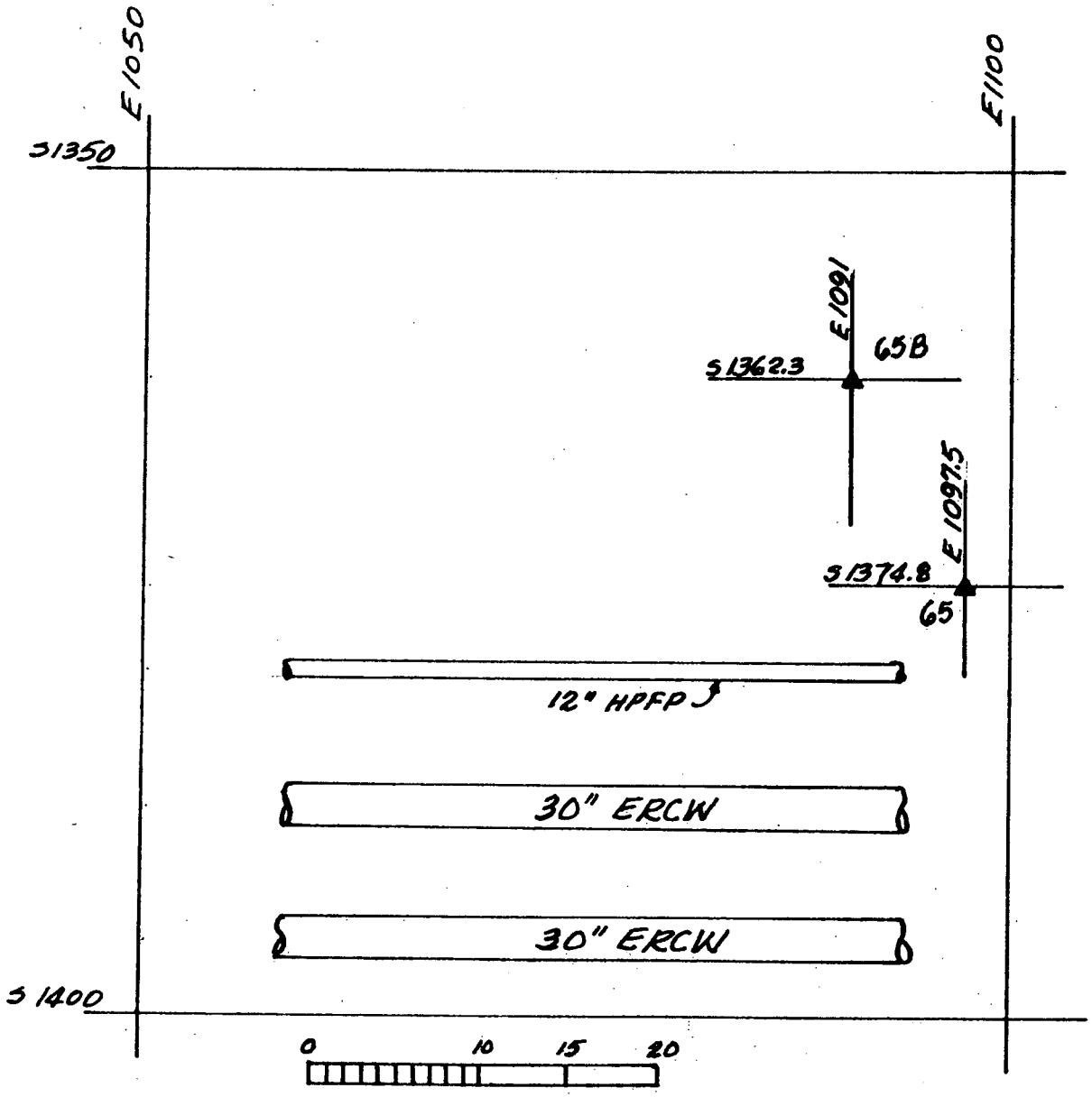
WATTS BAR NUCLEAR PLANT
ERCW PIPELINE
BORING RELATIONSHIP
BORING 50
FIGURE C-4



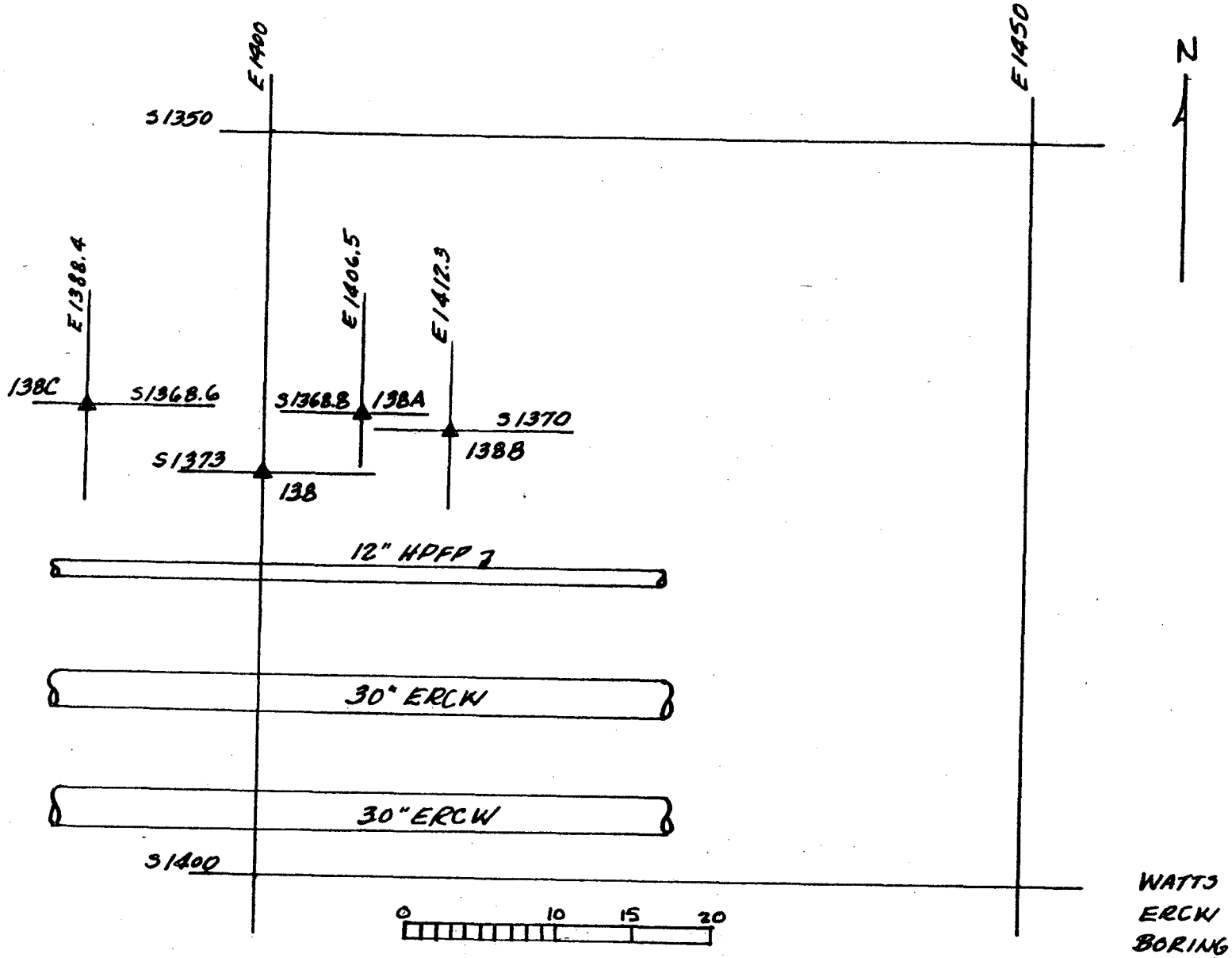
WATTS BAR NUCLEAR PLANT
ERCW PIPELINE
BORING RELATIONSHIP
BORING 134
FIGURE C-5



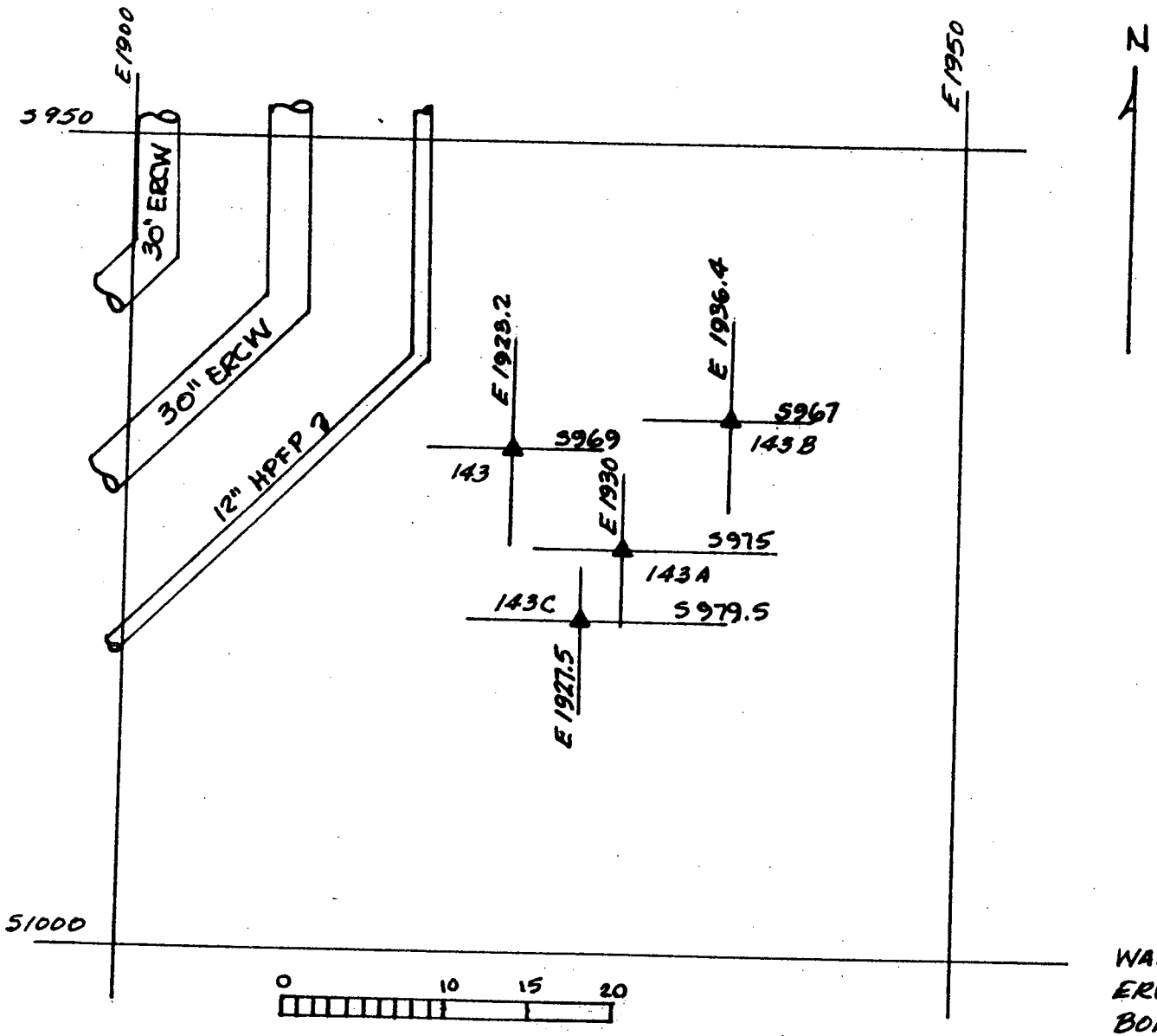
WATTS BAR NUCLEAR PLANT
ERCW PIPELINE
BORING RELATIONSHIP
BORING 135
FIGURE C-6



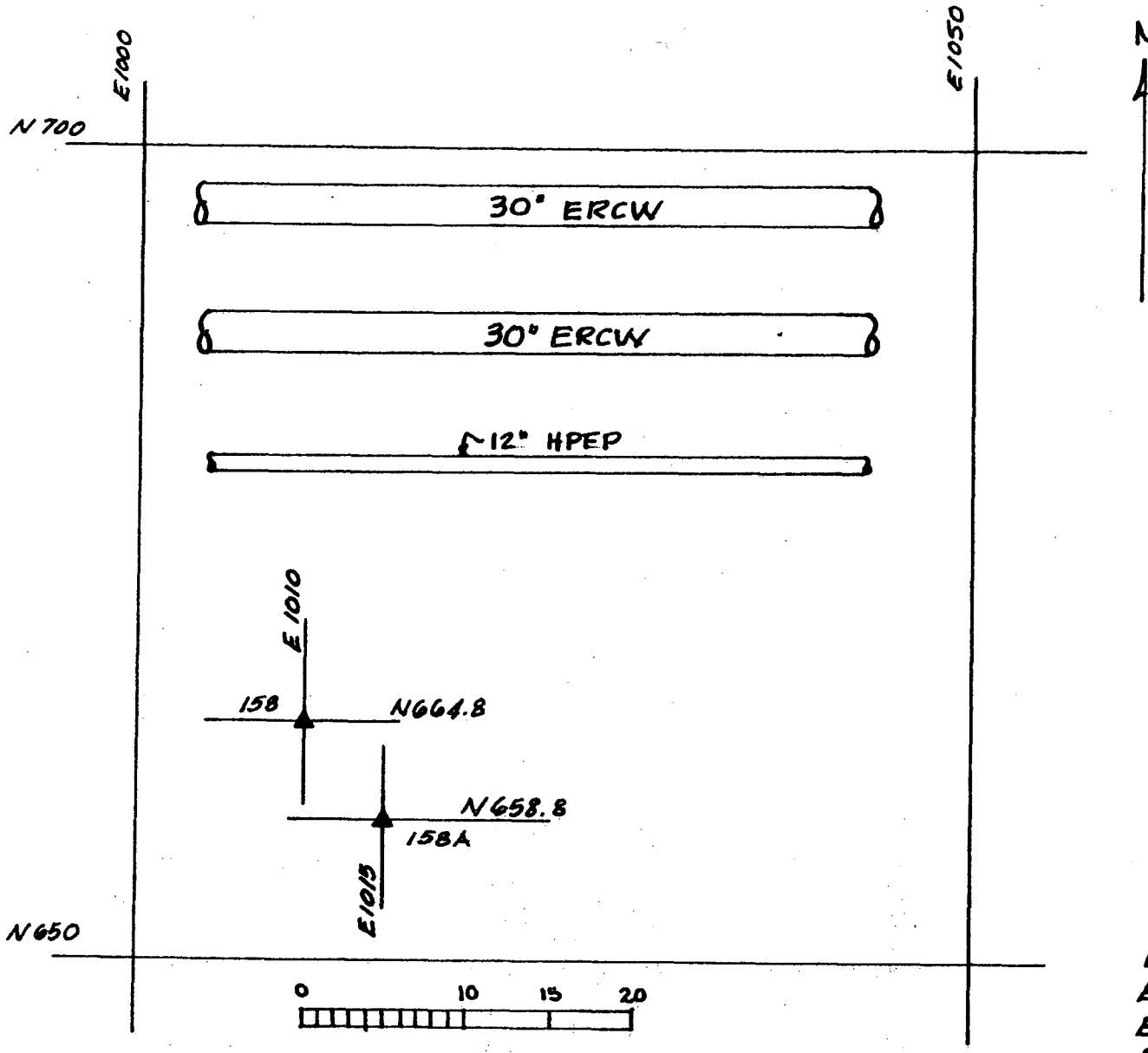
WATTS BAR NUCLEAR PLANT
ERCW PIPELINE
BORING RELATIONSHIP
BORING 65
FIGURE C-7



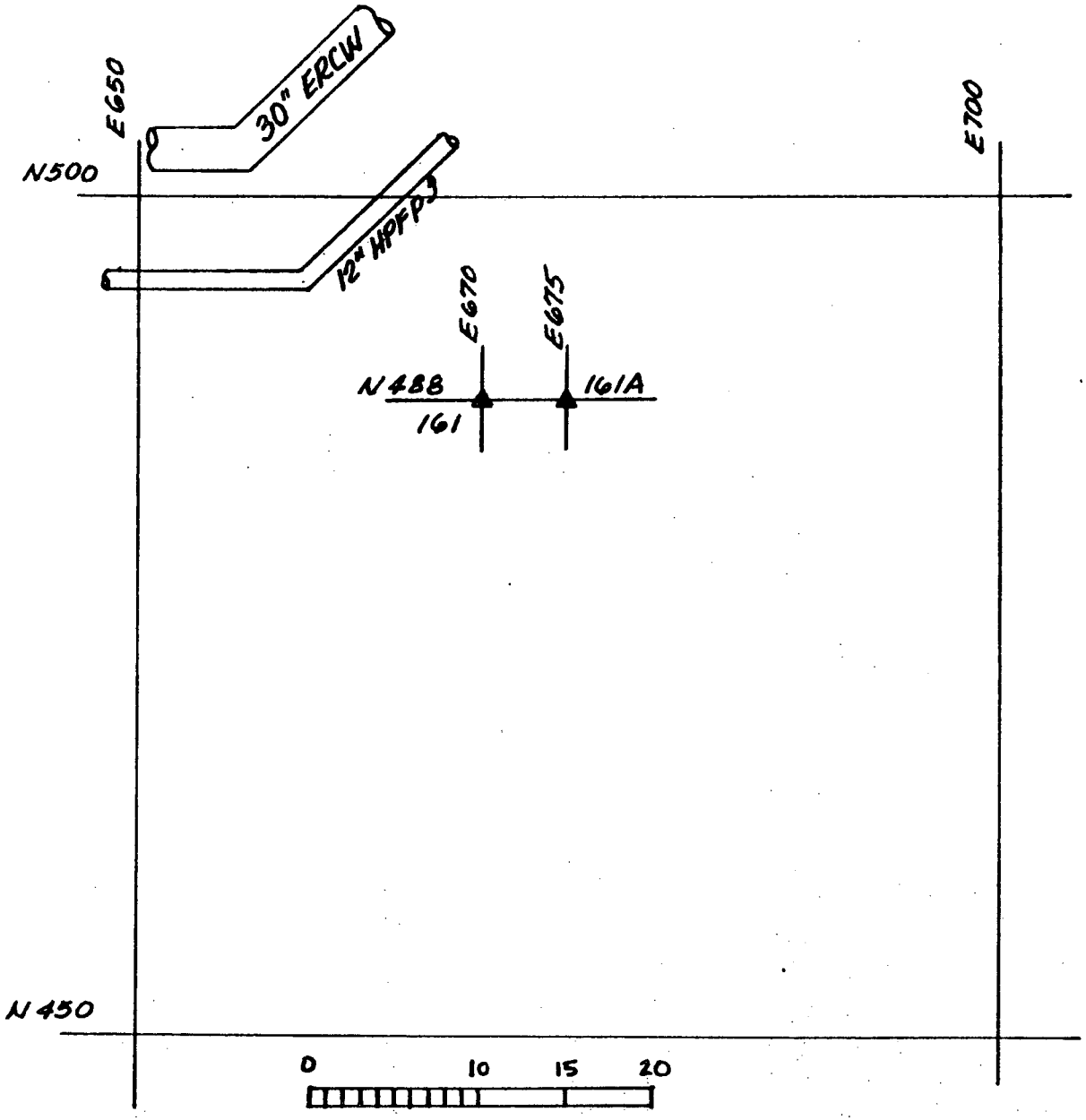
WATTS BAR NUCLEAR PLANT
ERCW PIPELINE
BORING RELATIONSHIP
BORING 138
FIGURE C-8



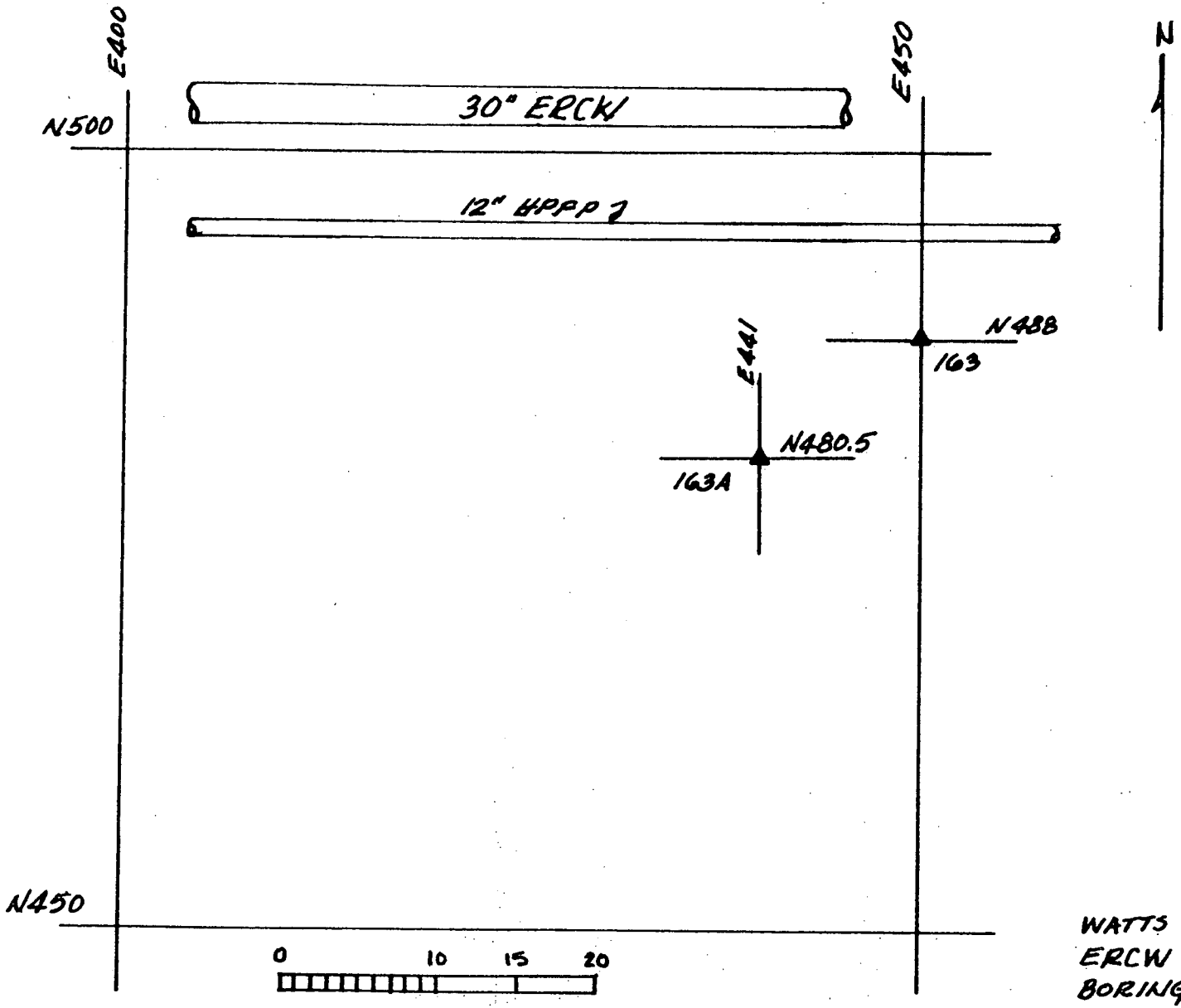
WATTS BAR NUCLEAR PLANT
ERCW PIPELINE
BORING RELATIONSHIP
BORING 143
FIGURE C-9



WATTS BAR NUCLEAR PLANT
ERCW PIPELINE
BORING RELATIONSHIP
BORING 158
FIGURE C-10



WATTS BAR NUCLEAR PLANT
ERCW PIPELINE
BORING RELATIONSHIP
BORING 161
FIGURE C-11



WATTS BAR NUCLEAR PLANT
ERCW PIPELINE
BORING RELATIONSHIP
BORING 163
FIGURE C-12

WATTS BAR NUCLEAR PLANT ERCW
SOIL PROFILE

Figure C-13

Boring SS-49	Boring SS-49A	Prepared by JLB
Station 1821.9S Range 868.7E	Station 1820.3S Range 871.93E	Checked by HPM
Surface El 716.9	Surface El 711.7	
Date Drilled 7-7-75 to 7-7-75	Date Drilled 11-16-81 to 11-18-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
715	30	MH	23.6	56.0	22.1						
	27		27.2	60.9	25.1						
	30		26.8	53.1	21.4						ROADBED GRAVEL
710	24	SM	15.4	29.4	5.4	17	CL-ML	21.1	32	8	
	23	SC	20.0	36.2	11.9	14	SM	21.4	30	6	
705	19	SM	21.2	36.0	11.6	9		24.6	29	3	
	18		26.7	34.0	10.2	5		26.5 21.6	28	2	
	13	SM-SC	25.1	28.3	6.5	5		26.5 29.0	NP	NP	ALLUVIUM
700	14	ML	26.1	28.8	5.3	6		29.9	23	1	
	12		26.8			5		31.8 32.4	NP 29	NP 4	
695	9	ML-CL	31.9	27.4	7.0	6	ML	28.3 28.0	22 22	1 3	
	11		29.1			CL 5		27.8 28.7	NP 28 23	NP 17 1	
690	6	SM	29.0	NP	NP	6	SM	30.0	NP	NP	
	4		28.0			17		31.3 21.2	NP	NP	
	31		25.3			50+	SM-SC	18.9	37	13	WEATHERED SHALE DISCONTINUED
685	50	SC	13.7								
	50		14.8	37.5	14.9						
	50		12.7								
680	50+		13.5								
	50+	GM	13.5	—	—						
675											

WATTS BAR NUCLEAR PLANT ERCW
SOIL PROFILE

Figure C-14

Boring SS-50	Boring SS-50A	Prepared by JLB
Station 1664.7S Range 787.5E	Station 1668.7S Range 787.8E	Checked by HPM
Surface El 721.8	Surface El 717.2	
Date Drilled 7-23-75 to 7-23-75	Date Drilled 11-13-81 to 11-19-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
720	25	CL	18.0	40.5	17.8						
	18		22.5	43.3	18.5						
	23		21.3								ROADBED GRAVEL
715	13	ML	20.0	44.2	16.0	22	CL-ML	19.9	43	16	FILL
	16		24.2			16	ML	19.8	42	15	
710	25		23.5	46.4	16.8	22		21.5	41	13	
	17		22.1	42.8	13.6	25		20.9			
	14		22.8	37.5	11.3	22		24.4	38	11	
705	10		22.1			20		25.3	35	9	ALLUVIUM
	10	SM	22.4	34.1	7.6	14	SM	25.5	NP	NP	
700	8	CL	25.6	36.3	12.0	11		28.8 26.9	27 NP	2 NP	
	5	SM	28.2			13		27.4 28.8	26 NP	2 NP	
	8		29.1			9		33.5 34.5	NP	NP	
695	2		31.5	NP	NP	5		38.4 34.9	29	3	
	10	G-SM	23.7			23	SM ML	25.1	NP 37	NP 10	
690	27		19.6			50	ML SM-SC	20.1	35	12	WEATHERED SHALE
	50+	SC	15.4	34.0	10.9						DISCONTINUED
	50+	CL	15.2	36.9	13.5						
685	50+	G-SC	12.0	31.7	10.4						
	50+		14.9								
680	30		15.2	29.3	8.1						
	50+		17.1	32.7	12.2						
	50+		5.1	25.5	7.8						
675	50+		7.3	—	—						
670											

WATTS BAR NUCLEAR PLANT ERCW
SOIL PROFILE

Figure C-15

Boring SS-134	Boring SS-134A	Prepared by JLB
Station 1370.0 S Range 900.0E	Station 1370.0 S Range 905.0E	Checked by HPM
Surface El 726.5	Surface El 725.5	
Date Drilled 6-6-79 to 6-7-79	Date Drilled 11-6-81 to 11-9-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
725	10	CL	20.0			13	CL	13.4	29	10	FILL
	16		16.9	28.0	3.4	25		16.1	32	12	
720	14		18.9			17		15.6	35	17	
	15		18.7	36.2	17.1	13		19.8	31	12	ALLUVIUM
	8		21.7			10		21.8	41	20	
715	13		20.9	39.1	19.5	12		19.4	34	15	
	2		23.8			4		25.3	42	17	
710	3	SM	29.3			4	SM	30.0	23	1	
								29.1	NP	NP	
	8		27.5	NP	NP	9		27.9	24	2	
								28.9	24	1	
	27	GM	11.4			27		31.9	NP	NP	
								16.3	27	2	
								SW-SM			
705	50+		10.0			39		11.2	NP	NP	
								GR-GM			
	50+	CL	18.1					—	—	—	WEATHERED SHALE
	50+		18.1			50+		SM-SC			
700				39.3	15.2	50+		20.7	40	15	
	50+		16.5								DISCONTINUED
	50+		16.6								
695	50+	SC	20.4								
				36.7	13.5						
	42		16.2								
690	50+	CL	15.2	35.6	13.3						
	50+		15.8	37.2	13.4						
	50+	SC	14.7								
685				35.6	12.1						
	50+		15.1								
	50+	CL	18.7	33.4	11.4						
	50+		—	—	—						
680											

WATTS BAR NUCLEAR PLANT ERCW
SOIL PROFILE

Figure C-16

Boring SS-135	Boring SS-135A	Prepared By JLB
Station 1373.0 S Range 1000.0E	Station 1363.3 S Range 1004.5E	
Surface Elev 726.9	Surface Elev 726.5	Checked By 4PM
Date Drilled 5-30-79 To 6-1-79	Date Drilled 11-9-81 To 11-10-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
725	7	CL	—	30.9	12.2	19	CL	16.1	28	7	FILL
	13		19.4	32.7	9.7	20		16.6	33	13	
	13		19.3	37.8	19.6	21	CL-ML	19.8	40	15	
720	21	ML	—	48.0	19.6	19	ML	24.7	41	13	
	14		26.7	46.5	16.5	19		26.7	41	12	
715	12		26.3	42.2	13.8	13	SM	24.3	31	3	ALLUVIUM
	11	SM	23.6	34.1	8.7	7		22.8			
	12		20.1	30.0	4.4	7		24.3	NP	NP	
710	8		—	—	—	5		34.2			
	8		—	— NP	— NP	8		27.0 / 32.1	22 / 27	1 / 2	
	8		25.3				ML	— / 32.1	39 / 25	7 / 2	
705	8		—			7	SM	30.9	NP	NP	
		CL	—	32.3	11.8	50+		16.7	—	—	
	22		28.9	44.5	15.8	36+	GM	30.1	46	14	
		G-SM									
700	26	SM	25.7	43.5	16.7						DISCONTINUED
	50+		20.4	38.9	12.7						WEATHERED SHALE
695	48		21.3	38.6	12.4						
	43		23.3	37.9	10.5						
	50+	CL	17.9	34.3	11.3						
690	50+		—	35.1	13.1						
	50+	SC	15.6	34.1	12.0						
	50+	CL	12.3	30.7	10.7						
685	50+		20.6	36.1	16.1						
	50+		11.2	28.5	8.7						
680											

WATTS BAR NUCLEAR PLANT ERCW
SOIL PROFILE

Figure C-17

Boring SS-65	Boring SS-65B	Prepared by JLB
Station 1374.8S Range 1097.5E	Station 1362.3S Range 1091.0E	Checked by HPM
Surface El 726.0	Surface El 727.2	
Date Drilled 7-25-75 to 7-25-75	Date Drilled 11-13-81 to 11-13-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
725						11	SC	14.1	28	8	FILL
	50+	GCL	15.4	36.3	15.6	20	CL	12.5	32	14	
	35	CL	12.9	35.6	14.1	25	MH	28.3	55	8	ALLUVIUM
720	24	MH	28.2	50.7	17.7	18		28.9	51	15	
	21		24.9			12	ML	29.1	42	12	
	13	ML	24.5	40.4	13.0	14		26.7	35	6	
715	16		29.2	46.1	15.6	9	SM	25.7	29	2	
	12	SM	21.5	33.1	6.6	6		27.5			
710	10		15.7	NP	NP	3		33.1 / 32.9	25 / NP	1 / NP	
	7		23.7	30.1	5.1	5		32.5	25	1	
	5		28.2	28.9	3.5	7		27.1 / 30.8	26 / 25	2 / 1	
705	8		—	—	—	37		22.7 / 10.7	NP	NP	
	20	GM	13.5	32.5	9.0			21.9	44	14	WEATHERED SHALE
700	18	SM	24.8	46.4	18.2						DISCONTINUED
	16	G-SM	23.8								
	16		24.7	43.4	15.9						
695	14		25.5								
	11	ML	40.7	47.1	13.4						
690	30		30.8	42.2	13.9						
	48	SC	19.8	34.4	11.2						
	50+	G-SC	14.3								
685				36.6	12.0						
	16		19.1								
	41		22.6								
680	45		17.1	33.8	10.5						
	50+		15.4								
				32.0	10.5						
	50		15.3								
675											

WATTS BAR NUCLEAR PLANT ERCW
SOIL PROFILE

Figure C-18

Boring SS-138	Boring SS-138A	Prepared by JLB
Station 1373.0S Range 1400.0E	Station 1368.8S Range 1406.5E	Checked by HPM
Surface El 727.2	Surface El 725.7	
Date Drilled 6-9-79 to 6-11-79	Date Drilled 11-12-81 to 11-12-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
725	18	CL	15.7	34.3	16.9	50	CL	6.7	35	18	FILL
	23	MH	28.5	55.0	24.1						
	15		27.5			32	ML-MH	27.3	50	19	
720	13	ML	30.1	48.0	19.7	19	ML	30.9	48	18	ALLUVIUM
	10		25.6	40.2	14.5	16		27.1	39	13	
715	9		22.3	31.6	7.8	12		25.1	33	6	
	6	SM	23.4	28.1	2.5	8	SM	25.1	29	3	
	7		24.5			8		22.1	NP	NP	
710	7	ML	28.4	32.7	5.9	12		27.1	29	1	
	5	ML-CL	29.6	27.0	5.1	4		35.6 28.1	26 25	2 2	
705	13	SM	15.0	26.4	2.3	9		27.8 29.1	22 NP	1 NP	
	16	G-SM	26.8			22	GM	10.6 38.4	NP	NP	WEATHERED SHALE
	43		26.7			50+	SM	25.8	26	2	
700	32		29.3	NP	NP						DISCONTINUED
	50+	SM	20.4								
695	50+		14.6								
	50+	SC	20.5	34.9	12.0						
	50+		14.8								
690	32	SM	22.1	33.0	6.2						
	50+		21.8								
	50+		12.1	NP	NP						
685											
680											

WATTS BAR NUCLEAR PLANT ERCW
SOIL PROFILE

Figure C-19

Boring SS-138 B	Boring SS-138 C	Prepared by JLB
Station 1370.0S Range 1412.3E	Station 1368.6S Range 1388.4E	
Surface El 726.6	Surface El 726.6	Checked by HPM
Date Drilled 12-8-81 to 12-8-81	Date Drilled 12-11-81 to 12-11-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
725											
720											SEE SS-138A
715											
710	8	SM	24.7	27	3	8		SM-SC 27.5	27	4	
	9		36.2	34	5	6	SC	34.1	31	11	ALLUVIUM
	8	SM-SC	30.0	27	5	4	CL	32.8	31	12	
705	7		32.5	26	5	50	GC	17.1	29	8	
	19	SM	29.5	40	12	35	SM	35.8	39	9	WEATHERED SHALE
											DISCONTINUED
700											

WATTS BAR NUCLEAR PLANT ERCW
SOIL PROFILE

Figure C-20

Boring SS-143	Boring SS-143A	Prepared by
Station 969.0S Range 1923.2E	Station 975.0S Range 1930.0E	JLB
Surface El 723.1	Surface El 723.0	Checked by
Date Drilled 6-14-79 to 6-14-79	Date Drilled 11-19-81 to 11-20-81	HPM

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
	10	CL	16.3	31.9	16.8	12	SC	15.0	33	16	FILL
720	24		16.4	30.5	12.7	20		13.3	31	13	
	13		15.9	35.3	17.0	13	CL	19.6	35	16	
715	10		20.9	35.4	16.1	12	CL-ML	21.8	33	10	
	9		19.4			9	SC	22.4	29	10	ALLUVIUM
	9	ML-CL	22.4	37.4	13.6	4	ML	36.5 30.1	41 43	20 13	
710	9		22.9	38.9	13.3	2		21.6	39	11	
	9		22.7	36.2	11.5	8	CL	37.2	36	19	
705	6	CL	21.8	39.3	18.2	1	SC	29.1	38	18	
	7		25.0			0		41.4	39	20	
	3		29.0	42.2	22.4	3	SM-SC	21.2	21	5	
700	4		25.6			4	CL	24.9	25	8	
	4		29.0	35.2	16.2	8	SM	43.1	37	11	
695	7		—	—	—	16	SC	33.8	34	12	
	9	G-SP-SM	13.5	NP	NP	31	SM	25.9	NP	NP	WEATHERED SHALE
	2	CL	—	31.2	15.5						DISCONTINUED
690	17		11.4	—	—						
	27	SM	21.2	29.9	2.6						
685	50	SC	16.3	30.8	9.7						
	50+		11.1								
	50+		13.9	29.8	10.9						
680	34	G-SC	15.2	29.6	9.2						
	50+	SC	14.0	28.0	7.6						
675											

WATTS BAR NUCLEAR PLANT ERCW
SOIL PROFILE

Figure C-21

Boring SS-143B	Boring SS-143C	Prepared by JLB
Station 962.0S Range 1939.9E	Station 972.6S Range 1930.2E	Checked by HPM
Surface El 723.3	Surface El 722.1	
Date Drilled 12-9-81 to 12-9-81	Date Drilled 12-11-81 to 12-11-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
720											SEE SS-143A
715											
	6	CL	22.3	33	15	9	CL	20.5	35	16	ALLUVIUM
710	3		32.3	36	15	4	CL-ML	31.0	37	13	
	4		22.4	26	11	6	CL	25.3	33	13	
	6		21.7	25	9	6		—	—	—	
705	4		21.1	24	8	1	CL	36.1	37	18	
	9		26.3	31	11	1		32.5	38	21	
700	4		22.5	24	8						
	7		30.0	32	11	2		23.7	23	8	
	21	SM	27.7	37	7	3	CL-ML	46.5	32	10	
695	32		25.6	36	7	53	SC	30.6	30	10	
						37	SM	20.4	34	8	WEATHERED SHALE
690											DISCONTINUED

WATTS BAR NUCLEAR PLANT EKOW

SOIL PROFILE

Figure C-22

Boring SS-158	Boring SS-158A	Prepared by JLB
Station 664.8N Range 1010.0E	Station 658.8N Range 1015.0E	Checked by HPM
Surface El 727.5	Surface El 727.6	
Date Drilled 6-26-79 to 6-26-79	Date Drilled 11-20-81 to 11-20-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
											GRAVEL ROADBED
725	8	CL	13.6			20	CL	16.1	26	9	FILL
	17		15.4	28.0	9.8	15		21.2	43	23	
	13		19.1			15		21.0	36	15	ALLUVIUM
720	11		17.3	26.9	11.0	19		23.2	41	18	
	9		18.4			15	ML	24.8	19	7	
715	7		23.0	34.0	12.2	8		25.2	32	8	
	3		27.6			4	CL	30.6	31	13	
	2	SM	32.2	22.9	2.5	3	SC	27.8	35	15	
710	39	G-SM	9.7	NP	NP	50+	GM	24.1	NP	NP	
	49	ML	21.5	28.0	2.8	23	SM	24.5	31	6	WEATHERED SHALE
705	28		24.7								DISCONTINUED
	50+	CL-ML	18.7	30.8	8.8						
	50+		11.3	26.6	5.4						
700	50+	CL	6.7	24.2	8.2						
695											

WATTS BAR NUCLEAR PLANT ERCW

SOIL PROFILE

Figure C-23

Boring SS-161	Boring SS-161A	Prepared by JLB
Station 488.0 N Range 670.0 E	Station 488.0N Range 675.0E	Checked by HPM
Surface El 732.4	Surface El 732.9	
Date Drilled 6-28-79 to 6-28-79	Date Drilled 11-23-81 to 11-23-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
730	20	CH	26.2	57.6	30.0	33	CH	28.4	62	34	
	14	ML	21.1	41.2	14.4	26	SM	19.2	39	12	
	9	CL-ML	25.1	43.0	16.5	13	CL	24.3	36	13	
725	8	SC	28.2	34.4	11.9	12	SM-SC	21.8	32	8	
	5	CL	25.3	30.8	9.4	9	SC	22.4	28	8	ALLUVIUM
720	6	SC	25.3	29.7	8.4	10	SM	23.8	26	2	
	9	SM	18.4	NP	NP	13		17.0	NP	NP	
	10		21.5			23	SC	14.0	29	9	
715	3	CL-ML	35.8	36.8	13.2	5	ML	35.7	38	12	
	5	ML	30.9	25.7	2.3	5	CL	33.0 / 32.4	32 / 27	13 / 9	
710	37	GM	11.1	NP	NP	50	SM	15.4			
	19	G-SM	12.7			40	GP-GM	10.3	NP	NP	
	45	CL-ML	21.0			16	SM	44.2			WEATHERED SHALE
705	50+		16.8	41.1	16.6						DISCONTINUED
	50+		18.6								
700	25		19.8	38.9	14.3						
	50+	CL	22.2 / 12.2	46.1	20.4						
	50+	SC	7.7	29.1	9.6						
695	50+		6.3								
690											

WATTS BAR NUCLEAR PLANT ERCW
SOIL PROFILE

Figure C-24

Boring SS-163	Boring SS-163A	Prepared by JLB
Station 488.0 N Range 450.0E	Station 480.5N Range 441.0E	Checked by HPM
Surface El 737.0	Surface El 737.5	
Date Drilled 6-28-79 to 6-28-79	Date Drilled 11-24-81 to 11-24-81	

El	SPT (N)	LOG	W	LL	PI	SPT (N)	LOG	W	LL	PI	REMARKS
											ASPHALT
											GRAVEL ROADBED
											CLAY FILL
735	19	CL	18.3	31.7	13.3						
	17		GP-GM 3.4	NP	NP						
	18	MH	25.2	54.0	20.9	25	MH	31.9	60	24	
730	10	CL-ML	24.7	40.0	15.3	23		SM-SC 22.6	37	13	
	9		27.7			22	SC	22.5	33	10	
	5	SM	19.7	NP	NP	15		18.8	30	9	ALLUVIUM
725	5	SM-SC	27.1			9	SM	25.8	31	6	
	5		28.4	30.4	7.1	7		28.9	31	7	
						11		SP-SM 28.2	NP 31	NP 10	
720	6		26.9					CL			
	3	SM	31.1	27.2	3.3	4	SM	36.3	30	3	
						5		CL 33.0 34.3 SM	31 31	9 5	
715	4		33.5	29.7	4.7	50+	SP-SM	16.2			
	17	G-SM	27.3	28.7	3.8	40		16.4			
	50+		7.8	NP	NP	50+		GP-GM 13.9	NP	NP	
710	50+		12.1 GP-GM			50+	SM	13.2 20.5	40	12	WEATHERED SHALE
	50+	ML	18.5	43.6	16.2						DISCONTINUED
	50+		21.2	37.3	9.0						
705	50+		2.7	—	—						
700											

APPENDIX D

DESCRIPTION OF TEST PITS

In support of the subsurface exploration along and adjacent to the ERCW yard piping route, two test pits were excavated at the locations indicated in figure C-1. Test pits were excavated by gradall during the week of December 7, 1981, and logged after excavation by TVA geotechnical engineers. Inspection by TVA geotechnical engineers and Woodward-Clyde Consultants took place on December 14, and 15, 1981. NRC representatives conducted inspection of the test pits on December 15, 1981. Selected block samples and soil tests were obtained and performed in the pits following the December 15 inspection.

Test pits had plan dimensions at the ground surface of approximately 40 feet by 50 feet and at the bottom of 10 feet by 10 feet. Total pit depths were approximately 22 feet each. The pits were extended below the water table elevation prior to installation of a dewatering system. This resulted in some washing of soil due to the seepage into the excavation. This ceased upon installation of the planned dewatering sumps in the northwest portion of the bottom of test pit No. 1 and in the southeast portion of the bottom of the test pit No. 2. Gravel was placed on the bottom of each test pit to provide a cleaner, stiffer surface to work from. Inspection by TVA, Woodward-Clyde, and NRC geotechnical engineers took place subsequent to installation of the sumps. Block samples were obtained from the east wall of test pit No. 1 and the north and west walls of test pit No. 2 while the dewatering system was in operation.

A generalized soil profile observed in test pit No. 1 consists of a surficial layer of either topsoil or 1032 stone underlain by a 9-foot to 10-foot thick zone of silty clay fill. Beneath this, interbedded silty and sandy clay alluvium is encountered, grading into sandy clayey silts and clayey silty sands with increasing depth. This trend of increasing particle size with depth is typical of alluvial profiles. Below a depth of 17 feet, stratified deposits of thinly-bedded sandy silts and silty sands

predominate and continue down to the test pit termination depth. Ground water was encountered in the pit during construction. A reliable water table depth measurement was not made, but it was approximately 4 feet above the base of the excavation.

The generalized soil profile for test pit No. 2 consists of a 2 foot to 5-foot thick surface zone of spoil material and boulder fill. Underlying this material, silty clay alluvium is encountered. These soils grade through sandy clayey silts into clayey silty sands with increasing depth. Below a depth of 17 feet, stratified deposits of thinly bedded sandy silts and silty sands predominate and continue down to the termination depth of about 21 feet. It should be pointed out that the north wall of test pit No. 2 exhibited a more pronounced interbedding of the silts and sands below 17 feet, and included thin clay strata within the profile. The improved definition apparently resulted from seepage through the wall into the pit. Ground water was also encountered in test pit No. 2 during construction. Again a reliable water table depth measurement was not made, but it was approximately 4 feet above the pit base.

The test pit logs are presented as tables D-1 and D-2 in this appendix. Logs of each wall of the pits are included. These should be consulted for a detailed description of the soil profile.

TABLE D-1

TEST PIT LOG

TEST PIT NO. 1

Ground Surface Elevation = 726.8

Depth From	Depth To	Description
		North Wall
0.0	1.0	Topsoil
1.0	9.0	Red tan silty clay (fill)
9.0	14.0	Light grey to yellow tan alternating thin beds of silty clay and sandy clay (alluvium)
14.0	17.8	Mottled silty sandy clay grading to clayey silty sand
17.8	22.3	Light to dark brown micaceous alternating thin beds of fine sandy silt and silty fine sand
22.3		Discontinued pit
		East Wall
0.0	1.0	1032 Stone
1.0	9.0	Tan to red tan silty clay (fill)-stratified by color
9.0	14.0	Light grey to yellow tan alternating thin beds of silty clay and sandy clay (alluvium)
14.0	18.3	Mottled silty sandy clay grading to clayey silty sand
18.3	21.3	Medium to dark brown micaceous alternating thin beds of fine sandy silt and silty fine sand
21.3		Discontinued pit

TABLE D-1 (Continued)

Ground Surface Elevation = 726.8

Depth From	Depth To	Description
		South Wall
0.0	1.0	1032 Stone
1.0	9.0	Tan to red tan silty clay (fill) - stratified by color
9.0	13.1	Light grey to yellow tan alternating thin beds of silty clay and sandy clay (alluvium)
13.1	19.1	Mottled silty sandy clay grading to clayey silty sand
19.1	20.6	Red tan to brown micaceous fine sandy silt - stratified by color
20.6	22.3	Yellow brown to dark brown micaceous alternating thin beds of fine sandy silt and silty fine sand
22.3		Discontinued pit
		West Wall
0.0	1.0	Topsoil
1.0	9.0	Tan to red tan silty clay (fill) - stratified by color
9.0	13.3	Light grey to yellow tan alternating thin beds of silty clay and sandy clay (alluvium)
13.3	19.1	Mottled silty sandy clay grading to clayey silty sand
19.1	21.4	Mottled micaceous clayey fine sandy silt
21.4	22.8	Tan to dark brown micaceous alternating thin beds of fine sandy silt and silty fine sand
22.8		Discontinued pit

TABLE D-2

TEST PIT LOG

TEST PIT NO. 2

Ground Surface Elevation = 725.6

Depth From	Depth To	Description
		North Wall
0.0	3.0	Gravelly clay (spoil material - fill)
3.0	10.0	Mottled silty clay (alluvium)
10.0	16.6	Mottled fine sandy clayey silt grading to clayey silty fine sand
16.6	18.6	Medium brown to tan micaceous alternating thin beds of fine sandy silt and silty fine sand
18.6	21.1	Tan to red tan to light grey alternating thin beds of fine sandy silt and clayey silt
21.1		Discontinued pit
		East Wall
0.0	2.0	Gravelly clay (spoil material - fill)
2.0	8.0	Mottled silty clay (alluvium)
8.0	15.1	Mottled clayey sandy silt grading to clayey silty fine sand
15.1	17.1	Medium brown to tan micaceous clayey silty fine sand
17.1	21.1	Medium brown to tan micaceous silty fine sand - stratified by color
21.1		Discontinued pit

TABLE D-2 (Continued)

Ground Surface Elevation = 725.6

Depth From	Depth To	Description
		South Wall
0.0	5.0	Gravelly clay (spoil material - fill)
5.0	10.0	Mottled silty clay (alluvium)
10.0	16.6	Mottled fine sandy silty clay grading to clayey silty fine sand
16.7	21.5	Tan to dark brown micaceous alternating thin beds of "dirty" poorly-graded sand and silty fine sand
21.5		Discontinued pit
		West Wall
0.0	2.0	Gravelly clay (spoil material - fill)
2.0	10.0	Mottled silty clay (alluvium)
10.0	16.6	Red to yellow tan fine sandy clayey silt grading to clayey silty fine sand
16.6	21.1	Yellow tan to dark brown micaceous alternating thin beds of fine sandy silt and silty fine sand
21.1		Discontinued pit

APPENDIX E

CONE PENETRATION TESTS

Static Cone Penetration Tests (CPT) were performed as part of the investigation of subsurface conditions along and adjacent to the Essential Raw Cooling Water (ERCW) yard piping route. Data from these tests will supplement information obtained from other methods of soil exploration and testing along the route and thereby aid in evaluating the foundation conditions. This appendix provides a brief discussion of the utility, methods, and equipment associated with the CPT, and outlines the scope of exploration undertaken using this technique.

The static (or quasi-static) CPT began development in Europe around 1920. In 1935, the Department of Public works in the Netherlands developed the device which has provided the basis for most modern versions of the CPT, generally referred to as the Dutch Cone (reference E-3). Essentially, the device consists of a rod tipped with a 60° cone having a projected area of 10 cm^2. As this is advanced through the soil deposit, tip resistance and any of several other parameters are measured, from which evaluation of the soil profile and certain engineering characteristics is made. The technique has seen increasing use in the United States since the 1960's, where it is generally used to supplement and enhance the data obtained from the Standard Penetration Test. In this regard, the CPT is well suited to rapid, detailed surveys of erratic soil deposits between boring locations and/or determining engineering characteristics of difficult to sample soils such as loose sands or soft clays and silts.

Static soundings conducted at Watts Bar Nuclear Plant were performed using the Suitcase Cone System developed by and leased from Woodward-Clyde Consultants. The system utilizes a conventional cone tip (60°, 10 cm^2) connected directly to a full-bridge strain gauge load cell as illustrated in figure E-1. This assembly was connected to EW-size drill rod for use with a conventional truck mounted Mobile B-61 drill rig as illustrated in figure E-2. During penetration, load-cell output in millivolts (mV) was transmitted through a shielded electrical cable threaded through sections

of drill rod and led out through a slotted rod coupling. The cable was connected through a junction box to a single channel strip chart recorder powered by a self-contained rechargeable battery. A range of scale settings (20, 50, and 100 mV) on the recorder enabled selection of the plot scale to enhance interpretation. This has provided a permanent and continuous record from which the tip resistance, q_c, will be scaled. No other parameters (side friction, pore pressure, etc.) could be measured using the furnished equipment.

The CPT's were performed by advancing the cone in 5 foot increments using the down crowd system of the drill rig. This corresponds to the standard rod length used by TVA and provided a convenient stroke length for the hydraulic down crowd advancing the cone. Depth intervals of 6 inches were recorded as ticks on the data trace of the strip chart using a manually operated switch. Insofar as possible, a constant advance rate of about 5 feet per minute was used. However, the actual penetration rate is clearly documented from the spacing of the depth event ticks on the strip chart together with the feed rate of the paper. Pre-augering through 5 to 10 feet of stiff fill clay was performed at sounding sites as appropriate, after which the cone could be continuously advanced by adding additional rod sections without withdrawing the portion already in the ground. Exceptions to this occurred when either side friction along the rods increased excessively or when the tip resistance became overly high prematurely in the engineer's opinion. In such cases, the cone was withdrawn and the sounding augered to the depth where cone penetration could be resumed. This was most necessary in spoil areas containing boulder fill. Care was taken to confirm that the water table was not penetrated by the augering process. The equipment was used to maximum depths of 35 feet. A more complete description of the equipment and test procedure is available in reference E-1 listed at the conclusion of this appendix.

The approximate locations of the cone tests performed at the plant are indicated on figure E-3. Of the 35 CPT locations indicated, only the 27 proirity soundings were made due to time limitations on TVA's use of the equipment. All soundings were made between December 14, and 18, 1981. The

soundings not performed are listed on figure E-3. Of the 27 CPT's performed, 3 were made adjacent to previous Standard Penetration Test boring numbers 134A, 65B, and 138A. These serve as site correlation tests and aid in evaluation of the CPT results.

The field records of cone tip resistance are now being reduced for interpretation by TVA geotechnical engineers. This consists of plotting curves of tip resistance, q_c, versus depth, from which the soil profile and consistency are inferred. Site specific correlations for these parameters are not yet complete; however, general guidelines illustrating the interpretive characteristics of the records may be stated as follows:

1) Sandy soils result in a "rougher" signature, i.e., numerous peaks and valleys occur along the general trend line.

2) Clays exhibit smoother signature, with peaks and valleys much less pronounced than in sands.

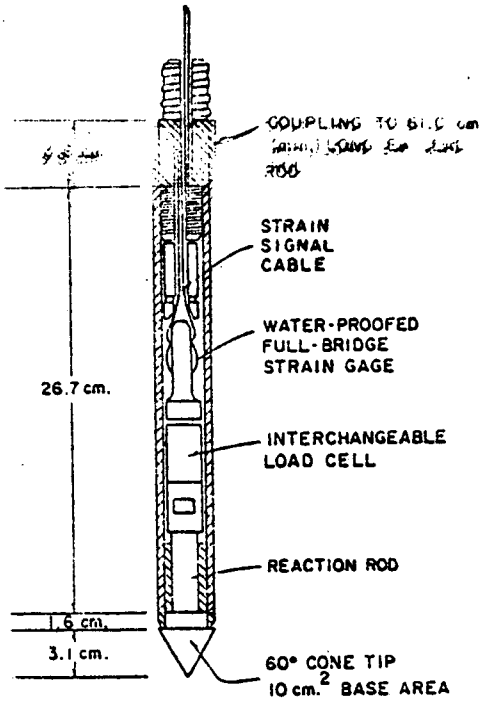
Figure E-4 taken from reference E-2 illustrates the typical trends, as well as the need for the site-specific correlation of soil type and consistency now in progress. The final logs will consist of the q_c versus depth plots with the various soil strata and consistencies delineated on each. These will be made available upon completion of the subsurface evaluation.

References:

E-1 Gardner, William S, and Nathan, Sreenivasan V. (1981), "The Suitcase Cone System", Cone Penetration Testing and Experience, Proceedings of a session at the ASCE National Convention, St. Louis, October 26-30, 1981.

E-2 U.S. Department of Transportation, Guidelines for Cone Penetration Test - Performance and Design, FHWA-TS-78-209, Washington, D.C.

E-3 Terzaghi, Karl, and Peck, Ralph B, Soil Mechanics in Engineering Practice, 2nd edition, New York, 1967.



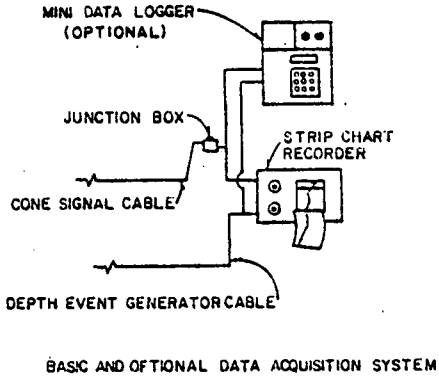


FIGURE E-1 - Schematic of Cone Assembly (from reference E-1)

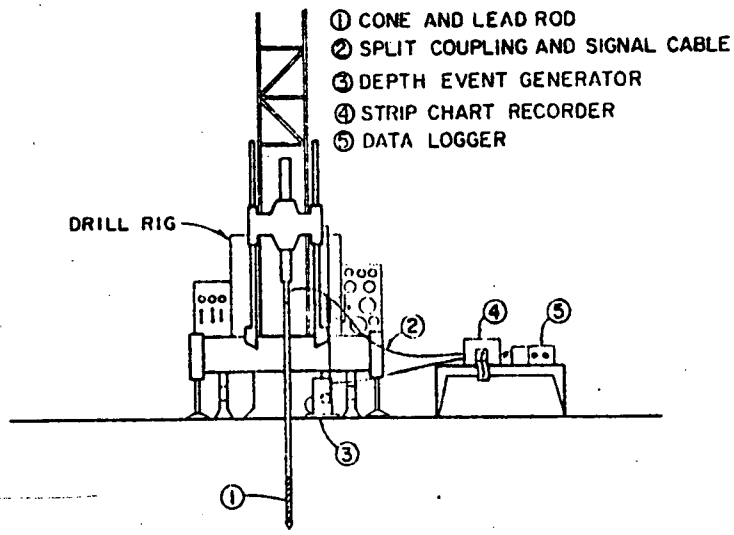


FIGURE E-2 - SCHEMATIC OF PORTABLE CONE SYSTEM DEPLOYED BY DRILL RIG (from reference E-1)

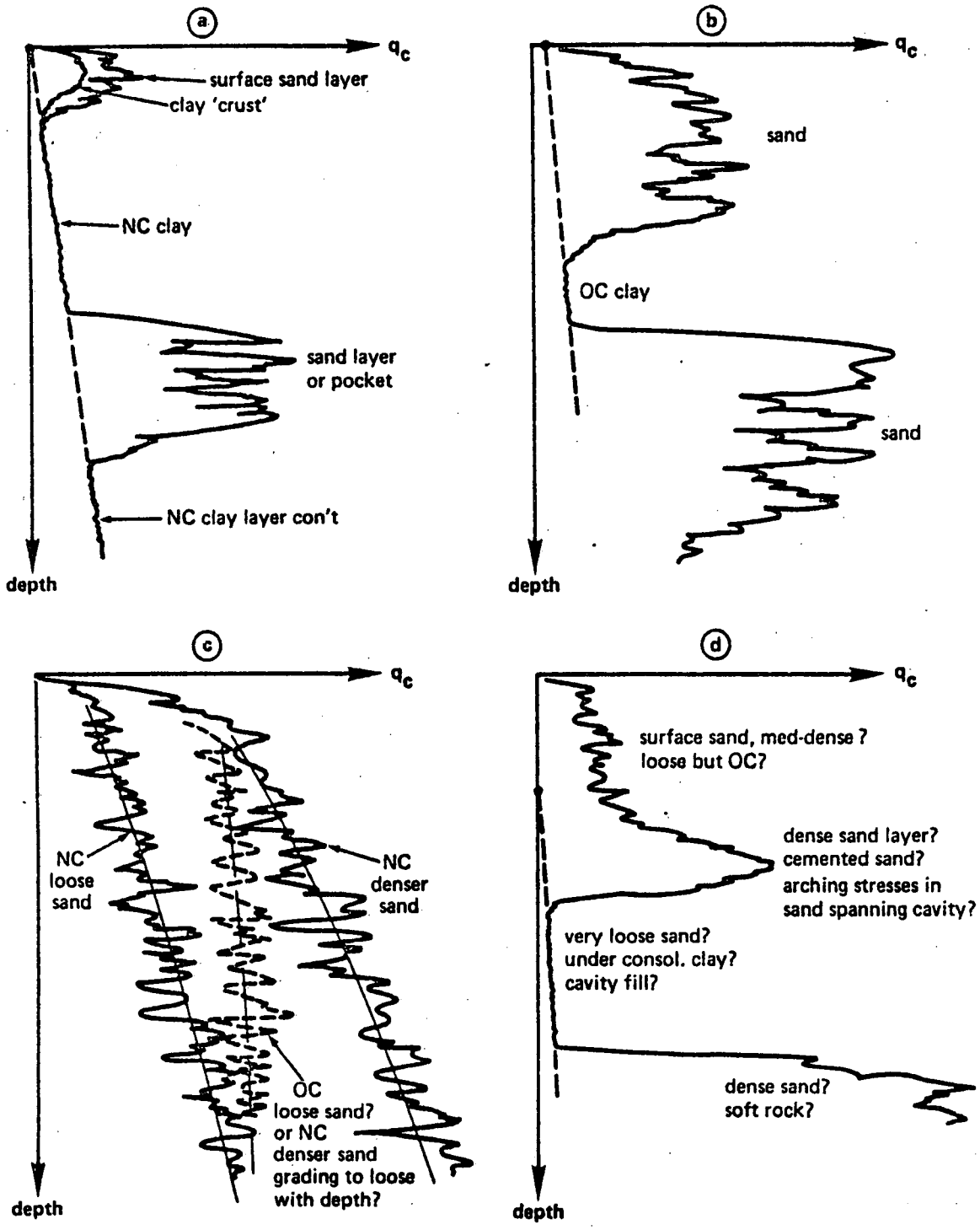


FIGURE E-4 - SIMPLIFIED EXAMPLES OF q_c-LOG PROFILES SHOWING LIKELY AND POSSIBLE INTERPRETATIONS FOR SOIL TYPES AND CONDITIONS (from reference E-2)