Liquefaction Evaluation of the ERCW Pipeline Route Watts Bar Nuclear Plant Task 9

Prepared for

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TABLE OF CONTENTS

	Page
LETTER OF TRANSMITTAL	i
TABLE OF CONTENTS	ii
LIST OF TABLES	iii
LIST OF FIGURES	iv
1.0 INTRODUCTION	1
1.1 Organization of Report	1
2.0 FIELD EXPLORATION AND SUBSURFACE SOIL CONDITIONS	3
2.1 Field Exploration	3
2.2 Subsurface Soil Conditions	4
3.0 METHODOLOGY OF EVALUATION	6
3.1 General Procedures for Evaluation of Liquefaction Potential	6
3.2 Method Based on Empirical Correlations	7
3.2.1 Historical Development of Empirical Correlations	7
3.2.2 Procedure Used for Evaluation of Liquefaction Potential	11
3.3 Method Based on Cyclic Strength Determined By Laboratory Cyclic Testing	14
4.0 RESULTS OF EVALUATION	17
4.1 Introduction	17
4.2 Evaluations at Selected Boring Locations	17
4.2.1 Borings SS-134 and SS-134A	18
4.2.2 Borings SS-138 and SS-138A	20
4.2.3 Borings SS-161 and SS-161A	22
4.3 Results of Evaluation Along the ERCW Pipeline Route	24
5.0 CONCLUSIONS	31
REFERENCES	R-1
APPENDIX A SITE-SPECIFIC STRESS REDUCTION COEFFICIENT	A-1

LIST OF TABLES

Table	
1	SUMMARY OF LIQUEFACTION EVALUATION OF SILTY SANDS (SM) ALONG ERCW PIPELINE ROUTE
2	SUMMARY OF LIQUEFACTION EVALUATION OF SILTS (ML) ALONG ERCW PIPELINE ROUTE
3	SUMMARY OF LIQUEFACTION EVALUATION OF CLAYEY SOILS ALONG ERCW PIPELINE ROUTE
A-1	ACCELEROGRAMS USED IN GROUND RESPONSE ANALYSES

LIST OF FIGURES

Figure

1 ERCW PIPELINE SECTION A-A

2 MAIN PLANT AREA SECTION

3 DIESEL GENERATOR BUILDING SECTION

4 SITE-SPECIFIC STRESS REDUCTION COEFFICIENT, r_d, FOR WATTS BAR NUCLEAR PLANT SITE

5 RECOMMENDED CURVES FOR DETERMINATION OF C_N BASED ON AVERAGES FOR W.E.S. TESTS (AFTER SEED AND IDRISS, 1981)

6 CHART FOR EVALUATION OF LIQUEFACTION POTENTIAL FOR M=6 EARTHQUAKE (FROM SEED AND IDRISS, 1981)

7 RESULTS OF CYCLIC TRIAXIAL TESTS ON SILTY SANDS (SM)

8 RESULTS OF CYCLIC SIMPLE SHEAR TESTS ON CLAYEY SILTS AND SILTY CLAYS (ML-CL) - CYCLIC STRESS RATIO AND NUMBER OF CYCLES TO CAUSE SHEAR STRAIN = $\pm$3.75 PERCENT

9 COMPARISON OF INDUCED STRESS RATIO WITH CYCLIC STRESS RATIO CAUSING LIQUEFACTION - BORING SS-134

10 COMPARISON OF INDUCED STRESS RATIO WITH CYCLIC STRESS RATIO CAUSING LIQUEFACTION - BORING SS-134A

11 GRAIN SIZE DISTRIBUTION CURVES FOR SILTY SAND SAMPLES FROM BORING SS-134

12 GRAIN SIZE DISTRIBUTION CURVES FOR SILTY SAND SAMPLES FROM BORING SS-134A

13 COMPARISON OF INDUCED STRESS RATIO WITH CYCLIC STRESS RATIO CAUSING LIQUEFACTION - BORING SS-138

14 COMPARISON OF INDUCED STRESS RATIO WITH CYCLIC STRESS RATIO CAUSING LIQUEFACTION - BORING SS-138A

15 GRAIN SIZE DISTRIBUTION CURVE FOR SILTY SAND SAMPLE FROM BORING SS-138

16 GRAIN SIZE DISTRIBUTION CURVES FOR SILTY SAND SAMPLES FROM BORING SS-138A

17 COMPARISON OF INDUCED STRESS RATIO WITH CYCLIC STRESS RATIO CAUSING LIQUEFACTION - BORING SS-161

18 COMPARISON OF INDUCED STRESS RATIO WITH CYCLIC STRESS RATIO CAUSING LIQUEFACTION - BORING SS-161A

LIST OF FIGURES (continued)

Figure

19 GRAIN SIZE DISTRIBUTION CURVE FOR SILTY SAND SAMPLE FROM BORING SS-161

20 GRAIN SIZE DISTRIBUTION CURVES FOR SILTY SAND SAMPLES FROM BORING SS-161A

21 SUMMARY OF RESULTS OF LIQUEFACTION EVALUATION

A.1 DETERMINATION OF MAXIMUM SHEAR STRESS

A.2 SHEAR WAVE VELOCITY PROFILE USED IN GROUND RESPONSE ANALYSIS

A.3 VARIATIONS OF STRESS REDUCTION COEFFICIENT, r_d, WITH DEPTH COMPUTED FROM GROUND RESPONSE ANALYSIS

1.0 INTRODUCTION

Subsequent to the submittal of Report No. CEB 82-6 R1 (Ref.15) to the NRC and a meeting between the NRC staff and the representatives of TVA and Woodward-Clyde Consultants (WCC) on May 14, 1982, additional studies were made to reevaluate the liquefaction potential of the foundation soils along the Essential Raw Cooling Water (ERCW) pipeline route at the Watts Bar Nuclear Plant site. The studies consisted primarily of two parts - 1. Development of a Site-Specific Top-of-Ground Motion, and 2. Development of a Site-Specific Reduction Coefficient and Reevaluation of Liquefaction Potential. The results of the Part 1 study were presented in a separate report (Ref. 18). This report presents the results of the Part 2 study.

The evaluations in this report were made for an earthquake with a peak acceleration of 0.22 g at the ground surface computed in the Part 1 study. The peak acceleration of 0.22g is the 84th percentile peak acceleration computed from the statistical analysis of peak accelerations from a suite of accelerograms obtained at recording stations having soil conditions comparable to those along the ERCW pipeline route. The available accelerograms used in the analysis were obtained during earthquakes of magnitude M_L 5.8 $\pm$ 0.5 within a distance range of 0 to 25 km. Thus, for the purpose of this study, the SSE was assumed to result in a horizontal peak acceleration of 0.22 g at the ground surface.

1.1 Organizaton 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 evauations; and Section 5 summarizes the conclusions of the study; Appendix A describes the ground response analyses conducted to develop the site-specific stress reduction coefficient for computing induced cyclic stresses with depth.

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 21. The borings were spaced at approximately 100 feet along the ERCW pipeline route. 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 were presented in the FSAR Section 2.5.

Ten additional borings and SPT sampling were made in 1981 in the vicinity of ten selected old boring locations. These additional borings include No. SS-49A, -50A, -134A, -135A, -65B, -138A, -143A, -158A, -161A, and -163A. At the locations of Borings SS-138A and -143A, redrilling was done 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 in these borings. The results of this additional field exploration and laboratory testing were presented in Ref. 15. 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 soils in the test pits was made by the NRC staff on December 15, 1981. A description of the test pits was presented in Ref. 15.

As described in Ref. 15, undisturbed block samples of silty sands (SM) and clayey silts to silty clays (ML and CL) were taken from the two test pits. One block sample of silty sands was obtained from pit 1 and two samples were from pit 2. Four block samples of clayey silts to silty clays were taken from pit 2. The block sample materials were selected on the basis of being the loosest and softest soils within each test pit; the sampling locations were selected by TVA in concurrence with the representatives of the NRC and WCC during the test pit inspection. Locations and soil types of the block samples were presented in Ref. 15. Cyclic tests and index properties tests were conducted on these block samples. The test results were presented in Ref. 15.

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 Figures 1, 2 and 3. Examination of soil types and blow counts obtained at the borings along the pipeline route 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 1). In addition, the thickness of each soil type of the alluvial soils (e.g. clays, silts, silty sands and gravels) varies significantly along the pipeline route. Underlying the alluvial soils are the laminated residuum and/or the weathered shale and the shale bedrock. The alluvial sand present along the ERCW pipeline route is generally classified as silty sand (SM). The silty sand contains a high percentage of fines; the percent passing a No. 200 U.S. Sieve is generally in the range of 30 to 50 percent. Most of the silty sand tested also showed some plasticity. Soil classification and SPT blow counts of each sample encountered in each boring are presented in Figures 1, 2 and 3.

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 were 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 saturated 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, Seed and Idriss, 1981). The earlier 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 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 resist-

ance 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.

(a) Data from 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 (Seed, 1976) 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 ((d) above) 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) in the silty sands are increased by 7.5.

A recent study by the Corps of Engineers for a dam-site in Oklahoma (Seed, 1982) 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). Data obtained by Hammond (1982) for a site in Northern California show that the Seed and Idriss correlations for silty sands are conservative.

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. A series of cyclic simple shear tests were conducted on block samples from the site. The test results were presented in Ref. 15 and summarized in Section 3.3. The evaluation of cyclic mobility of this material was made based on the cyclic test results.

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.

In addition to the above criteria, clayey soils with blow counts of 5 or greater are considered non-liquefiable. Clays with blow counts of 5 are classified as medium stiff and can be considered non-liquefiable.

In this study, all Atterberg limit tests on the split-spoon samples were performed by TVA's Singleton Materials Engineering Laboratory in accordance with ASTM Test Designation D 423 and D 424. In accordance with the testing procedure, all soil samples were air-dried first.

As discussed in Ref. 15 there are two methods for determining the liquid limit of soils. One method is called the air-dry method and the other is called the wet method. All correlations using the Atterberg limits by A. Casagrande were based on the wet method. The American Society for Testing and Materials (ASTM) is currently revising the testing procedure for the Atterberg limits determination. According to Ladd (1982), the wet method is recommended as a preferred method in the revision and should result in correct values. The air-dry method generally results in lower values of the liquid limit than those determined by the wet method. This was confirmed by tests performed by TVA's Singleton Materials Engineering laboratory as described in Ref. 15. Thus, it is likely that

the values of the liquid limit determined previously for the clayey soils encountered in the borings are lower than the correct values. In the evaluation described in Section 4, clayey soils identified as having water contents greater than 0.9 LL may actually have water content less than 0.9 LL if the correct values of the liquid limit were used.

In this study, the cyclic mobility of the clayey soils was also evaluated based on the cyclic test results.

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 (Seed and Idriss, 1970):

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

where a_{max} = maximum acceleration at the ground surface, equal to 0.22 g for this study

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 reduction factor decreasing with depth from a value of unity at the ground surface to account for response of a deformable soil column as compared to response of a rigid body column.

The variation of r_d with depth was computed from a series of ground response analyses described in Appendix A. Based on the analysis results in Appendix A, the variation of r_d with depth shown in Figure 4 was developed and used in conjunction with Equation (1) to compute the induced cyclic shear stress.

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 5. 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 6 with the value of N_1 calculated in Equation 2. For this study, the SSE corresponds to an earthquake with a magnitude approximately equal to 5.8. Thus, the corresponding curve for $M \approx 6$ developed by Seed and Idriss (1981) 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 6 to estimate the cyclic strength of the silty sands. 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 - <u>Comparison of Induced Shear Stress with Cyclic Strength</u>

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 stress ratio causing liquefaction (Step 2) to induced stress ratio (Step 1).

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 loca-

tion 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 some differential settlements in the foundation soils after 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 could lead to landsliding or lateral spreading of the foundation soils. However, potential for landsliding or lateral spreading depends 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 Ref. 15.

3.3 Method Based on Cyclic Strength Determined by Laboratory Cyclic Testing

As summarized previously, 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. 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 Ref. 15, eleven cyclic triaxial tests were conducted on block samples of silty sands from two test pits. The results of these tests were presented in Ref. 15. Cyclic strength characteristics applicable to an in-situ sand deposit

can be obtained from the cyclic triaxial test results by the following equation to account for differences in the state of stress in laboratory samples as compared to elements of soil in the field.

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

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 laboratory.

σ_o' = field effective vertical stress.

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). The cyclic strength characteristics of the silty sands based on the cyclic test data are summarized in Figure 7. The cyclic strength characteristics have been defined in terms of the cyclic shear stresses required to cause an axial strain of $\pm$2.5 percent. A conservative curve was derived based on the test data and is shown in Figure 7. Based on the curve in Figure 7 and equation (3), the cyclic shear stress ratio required to cause $\pm$2.5 percent strain in the field for 6 cycles of loading, representing an $M \approx 6$ earthquake (Seed, et al., 1975), is calculated to be 0.21. The cyclic strength of the silty sands is used in the liquefaction evaluation presented in Section 4.

For clayey silts to silty clays four cyclic simple shear tests

were conducted on block samples to evaluate the cyclic mobility of these soils. Cyclic strength characteristics applicable to an in situ soil deposit can be obtained directly from the cyclic simple shear test results without applying correction factors (Seed et al., 1975). The results of the cyclic tests were presented in Ref. 15. As discribed in Ref. 15, the natural water contents of the samples tested range from 33.5 to 35.2 percent. The liquid limit of these samples ranges from 37 to 43 and the plasticity index ranges from 6 to 18. Three of the samples are classified as ML materials and one is classified as a CL material. It is noted that the water contents of the samples tested correspond to the upper-bound values of the water content of the clayey soils encountered along the pipeline route and summarized in Tables 2 and 3. This indicates that the samples tested are representative of the weakest clayey soils encountered along the pipeline route.

The cyclic strength characteristics of the clayey soils based on the data in Ref. 15 are presented in Figure 8. Figure 8 shows the variations of cyclic shear stress ratio with number of cycles to cause a shear strain of ± 3.75 percent (corresponding to an axial strain of ± 2.5 percent). The test data indicate that the cyclic strength of the CL material is higher than that of the ML material. Based on the data in Figure 8, the cyclic shear stress ratio for 6 cycles of loading is 0.25 for the clayey silts (ML) and 0.29 for the silty clays (CL). The cyclic strength of the clayey silts is approximately 19 percent higher than that of the silty sands. The cyclic strengths of the clayey silts and silty clays are used in the evaluation of cyclic mobility of the clayey soils encountered in the borings along the pipeline route presented in Section 4.

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.

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 established in Ref. 15. The basis for these contours was discussed in Ref. 15.

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 and SS-134A, Borings SS-138 and SS-138A, and Borings SS-161 and SS-161A. As described in Section 2.1, 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.

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 Figures 9 and 10. 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 Figures 9 and 10 consists of a 6.5-foot thick fill of silty clay (CL) underlain by a 15.5-foot 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 Figures 9 and 10. 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-foot 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 11 and 12 for Borings

SS-134 and -134A, respectively. As illustrated in Figures 11 and 12 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. The cyclic strength characteristics of the silty sands were determined based on the laboratory cyclic tests of undisturbed block samples described in Ref. 15 and summarized in Section 3.3 and the empirical correlations (i.e., the N-value approach) for silty sands developed by Seed and Idriss (1981) and described in Section 3.2.

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 9 and 10 for Borings SS-134 and -134A, respectively. The comparisons shown in Figures 9 and 10 indicate that the cyclic strength of the silty sand layer in Borings SS-134 and -134A based on the laboratory cyclic test results is higher than the induced shear stress due to the postulated SSE. The cyclic strengths based on the N-value approach for N = 8 in Boring SS-134 and N = 9 in Boring SS-134A are higher than the induced shear stress. However, the cyclic strengths for N = 3 in Boring SS-134 and N = 4 in Boring SS-134A are slightly lower than the induced shear stress. 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 0.9 to 1.3 based on the N-value approach. The factor of safety of 0.9 was obtained for N = 3 in Boring SS-134. The 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 cyclic triaxial test results described in

Ref. 15 and summarized in Section 3.3, the calculated factors of safety for the layers are greater than 1.2.

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 Figures 13 and 14. 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 Figures 13 and 14 consists of a 7-foot 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 Figures 13 and 14 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 Figures 13 and 14. 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 about 10 feet thick.

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. The evaluation of the liquefaction potential of the silty sand (SM) and sandy silt (ML) encountered in these borings is described below.

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 15. 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 16. The grain size distributions shown in Figure 16 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 Sections 3.2 and 3.3, 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 13 and 14 for Borings SS-138 and -138A, respectively. The comparisons in Figures 13 and 14 indicate that the cyclic strength of the silty sand layer based on the laboratory cyclic test results is higher than the induced shear stress due to the postulated SSE. The cyclic strengths based on the N-value approach are also higher than the induced shear stress except for the SM sample with N = 4 at elevation 707 in Boring SS-138A. The calculated factor of safety based on the N-value approach ranges from about 1.0 to 1.7. The factor of safety of 1.0 was obtained for N = 4 in Boring SS-138A. Based on the cyclic strength determined from the cyclic triaxial test results presented in Ref. 15 and Section 3.3, the calculated

factors of safety are greater than 1.3. Also shown in Figure 13 is the cyclic strength of the silt (ML) at El. 709.2 determined from the cyclic simple shear test results presented in Section 3.3. The calculated factor of safety for the silt is 1.6.

As described in Ref. 15. Boring SS-138A was also redrilled twice to investigate the SM layer with N = 4 at elevation 707. These additional borings are identified as SS-138B and -138C. The soil profile, blow counts, and index properties for these borings were presented in Ref. 15. In general, Borings SS-138B and -138C are offset 10 feet on either side of -138A. 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 very limited in extent. The calculated factor of safety based on the N-value approach for the SM material in Boring SS-138B is 1.4.

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 Figures 17 and 18. 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 Figures 17 and 18 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. 719.5 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 feet, respectively, and are depicted in Figures 17 and 18. The design water table is at El. 722 feet.

As discussed in Section 3.2, the silty clay (CL), clayey to sandy silt (ML-CL) and clayey sand (SC) are considered to be potentially liquefiable 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. These soils are evaluated for liquefaction potential when their 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 evaluated by using the results of the cyclic simple shear tests described in Ref. 15 and summarized in Section 3.3. The evaluation of the liquefaction potential of the 4-foot thick layer of silty sand (SM), the 2-foot thick layer of sandy silt (ML), and the 2-foot thick layer of clayey silt (CL-ML) encountered in these two borings is presented below.

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 19 and 20, 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 19 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 19. 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 liquefaction are shown in Figures 17 and 18 for Borings SS-161 and -161A, respectively. The comparisons in Figures 17 and 18 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 based on the cyclic strength determined from the N-value approach range from 1.4 to 2.0. Based on the cyclic strength determined from the cyclic triaxial test results, the calculated factors of safety are greater than 1.3. Also shown in Figures 17 and 18 are the cyclic strengths of the silts (ML) and clayey silts (CL-ML) determined from the cyclic simple shear test results summarized in Section 3.3. The calculated factors of safety for the silts and clayey silts are about 1.5 and 1.7, respectively.

4.3 Results of Evaluation Along the ERCW Pipeline Route

Using the cyclic test results, the SPT blow counts, the grain size distributions, and index properties (Atterberg limits, water contents, etc.), the liquefaction potential of the alluvial soils encountered along the ERCW pipeline route due to the postulated SSE has been evaluated. The results of this evaluation are summarized in Tables 1, 2, 3 and Figures 1, 2, 3 and 21. Table 1 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 liquefaction to the induced cyclic stress ratios due to earthquake shaking) is relatively low. The lower factors of safety are those calculated based on the cyclic strength determined from the empirical correlations with SPT blow counts. As summarized in Table 1, in those locations where lower factors of safety were calculated using SPT blow counts, the factors of safety based on the cyclic tests are higher, ranging from 1.2 to 1.6.

The locations of the SPT samples identified in Table 1 are depicted on the soil profiles along the ERCW pipeline route shown in Figures 1, 2 and 3 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 silty sands summarized in Table 1 and Figures 1, 2 and 3 show that among all the SPT data obtained from a total of 70 boring locations spaced at approximately 100 feet along the ERCW pipeline route and a total of 464 SPT data points below the design water table, only 14 N-values result in a calculated factor of safety based on the N-value approach equal to or slightly less than one (i.e. N=2 at El. 693.8 in Boring SS-50, N=3 at El. 710.5 in Boring SS-134, N=4 at El. 709.5 in Boring SS-134A, N=5 at El. 706.0 in Boring SS-65, N=3 at El. 709.2 in Boring SS-65B, N=5 at El. 707.2 in Boring SS-65B, N=4 at El. 707.2 in Boring SS-138A, N=2 at El. 711.5 in Boring SS-158, N=3 at El. 717.0 in Boring SS-163, N=4 at El. 715.0 in Borings SS-163, N=2 at El. 713.4 in Boring SS-84, N=2 at El. 712.1 in Boring SS-128, N=2 at El. 714.4 in Boring SS-125, N=2 at El. 715.6 in Boring SS-25). The evaluation of the liquefaction potential at each of these SPT sample locations is discussed in the following paragraphs.

At two of the boring locations identified above (Borings SS-50 and -158), redrilling (Borings SS-50A and -158A) was made using the rotary drilling technique in 1981. The results indicate that the N-value increased from 2 to 5 at the corresponding depth in Boring SS-50 and -50A. In addition, the soil classification of the N=5 sample from Boring SS-50A based on the grain size distributions and the Attenberg limits indicates that one half of the split-spoon sample was classified as ML (D_{50} = 0.07 mm, 55 percent finer than 0.074 mm) and the other half was classified as SM (D_{50} = 0.07mm, 39 percent finer than 0.074 mm). 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 previous N-value was 2. Thus, the results of the redrilling in 1981 indicated that there is not a liquefaction potential or the thickness and the extent of the silty sand are localized.

At Borings SS-134 and -134A, the soil profile consists of a stiff to very stiff clay of about 15 feet overlying a 4-ft thick silty sand. Two SPT samples were taken at each boring. The N-values were 3 and 8 at Boring SS-134. The N-values were slightly higher (N=4 and 9) at the companion boring (Boring SS-134A). The calculated factors of safety based on the N-value approach are slightly less than one for N=3 and 4 and are equal to 1.3 for N=8 and 9. Thus, the thickness of the silty sand with factor of safety less than one based on blow counts at this location is only about 2 feet.

At Borings SS-65 and -65B, the soil profile consists of a very stiff clay and a dense silt overlying a 10-ft to 11-ft thick silty sand. Five SPT samples were taken at each boring. The N-values were 12, 10, 7, 5, and 8 at Boring SS-65 and 9, 6, 3, 5, and 7 at Boring SS-65B. The N-values less than 5 resulted in calculated factors of safety equal to or slightly less than one. The thickness of the silty sand with these factors of

safety is about 4 feet at Boring SS-65B and 2 feet at Boring SS-65.

At Boring SS-138A, the soil profile consists of a very stiff to very dense clay and silt about 12 feet thick overlying a 10-foot thick silty sand. Five SPT samples were taken. The N-values were 8, 8, 12, 4, and 9. The calculated factors of safety are all greater than 1.3 except for the N-value of 4. The factor of safety for N=4 is slightly less than one. As discussed earlier, Boring SS-138A has two companion Borings SS-138B and -138C in addition to original Boring SS-138. The companion borings indicated that the lower blow count SM material (N=4) in SS-138A is very limited in its lateral extent.

At Borings SS-163 and -163A, the soil profile consists of about 6 feet of gravel/clay fill and about 12 feet of silty clay to clayey sand overlying about 4 feet of silty sand. At Boring SS-163, N-values of the silty sand were 3 and 4 which resulted in calculated factors of safety being equal to or slightly less than one. Companion Boring SS-163A indicates that the silty sands encountered in Boring SS-163 with N=3 and 4 correspond to a silty sand (D_{50} = 0.08mm) with N=4, and a clayey sand (CL-SM) with N=5 in Boring SS-163A. Thus, the lower blow count SM material in SS-163 is very limited in its lateral extent.

It should be noted that in all borings identified to have N-values with calculated factors of safety equal to or less than one, only one low N-value is present at each boring location except in Borings SS-65B and -163. In Borings SS-65B and -163, two successive drives resulted in low N-values (3 and 5 in Boring SS-65B, and 3 and 4 in Boring SS-163). This indicates that the thickness of the silty sand with low N-values is less than two feet except at Borings SS-65B and -163. Companion borings at Boring SS-65B and -163 indicate that the thickness of the

silty sand with low N-value is very limited in its lateral extent.

Borings SS-25, -82, -125, -128 and -130 in the main plant area each contain only a single 2-foot layer of SM material with lower factors of safety except at Boring SS-130 where the thickness of the SM layer is 4 feet. 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 resulted in increased factors of safety. Nevertheless, these borings are located in a level area where there is no potential for slope failure. Thus, the only possible consequence of liquefaction in this area would be induced settlement as the excess pore water pressure is dissipated. Such settlements could range from approximately 1% to not more than 5% of the layer thickness. Considering the thickness of the questionable SM material to be 2 feet (the sampling interval for the continuous split-spoon 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 silty sands (SM) having lower blow counts are summarized in Table 1. These same materials were evaluated using the cyclic strength determined

from the cyclic tests on undisturbed block samples. The factors of safety calculated using the cyclic test data are shown in the last column of Table 1. These factors of safety range from 1.2 to 1.6. The cyclic test results are directly applicable for those borings in the vicinity of the two test pits (SS-49A, -131, -50, -134, -134A, -135, -135A, -65, -65B, -136, -138A, -139 and -140). Assuming that the test results are also applicable in the main plant area, the listed factors of safety for Borings SS-158, -162, -163, -163A, -80, -25, -84, -125 and -128 were obtained. The results in Table 1 indicate that for both regions the materials possess adequate factors of safety against liquefaction.

For the silts (ML) encountered along the pipeline route, the evaluation was made by using the cyclic strength determined based on the results of the cyclic simple shear tests. The cyclic tests were conducted on undisturbed block samples obtained from test pit 2. The results of the cyclic tests are summarized in Section 3.e. The induced stresses due to the SSE were compared with the cyclic strength. The calculated factors of safety for the ML layers having an N-value less than 5 are summarized in Table 2. The factors of safety calculated for the silts (ML), identified in Table 2, range from 1.4 to 1.8. Therefore, the silt (ML) material encountered along the pipeline route should have adequate factors of safety against cyclic mobility or generation of significant excess pore pressures.

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 3. The cyclic behavior of these soils was evaluated based on the cyclic strength determined from the cyclic simple shear test summarized in Section 3.3. The cal-

culated factors of safety for these soils are summarized in Table 3. The data in Table 3 indicate that the calculated factors of safety are greater than 1.7 for the clayey soils. In addition, as described in Section 3.2, the correct values of the liquid limit could have been higher than those shown in Table 3 and the natural water contents of most of these soils could be less than 0.9 LL. Thus, the clayey soils encountered along the pipeline route should have adequate factors of safety against cyclic mobility and generation of significant excess pore pressure.

5.0 CONCLUSIONS

The liquefaction evaluation of the alluvial soils along the ERCW pipeline was conducted using blow count data in the procedure of Seed and Idriss (1981) and using cyclic test results of silty sands, silts and clayey soils. The results of this evaluation are presented and discussed in Sections 4.2 and 4.3. Based on these results the following general conclusions can be made.

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

2. The silts (ML) along the pipeline were evaluated based on the cyclic strength determined from the cyclic test results. The evaluation indicates that the silts have adequate factors of safety against cyclic mobility and generation of significant excess pore pressures.

3. The clays (CL and SC) were evaluated based on the results of the cyclic tests. The evaluation shows that the clays have adequate factors of safety against cyclic mobility and generation of significant excess pore pressures.

4. The silty sands (SM) were evaluated using the cyclic strength based on both the empirical correlation with SPT blow counts and the cyclic test results. The evaluation indicates that the soils have adequate factors of safety based on the results of the cyclic tests. Lower factors of safety were calculated at some locations using the SPT blow counts and the empirical correlation.

5. On the east and north legs of the pipeline, the evaluation based on blow counts shows all soils to possess ample factors of safety against potential liquefaction.

6. On the south leg of the pipeline, borings SS-134, -65, and -138 contain SM material with relatively low factors of safety (equal to or slightly less than 1.0) when using the N-value approach. The materials with a low factor of safety are scattered and localized. The evaluation based on the results of the cyclic tests indicates that these same SM materials have factors of safety ranging from 1.2 to 1.3. Thus, the results of the study indicate that liquefaction potential is not a significant problem on the south leg of the pipeline.

7. In the main plant area, Borings SS-163, -25, -84, -125, -128, and -130 contain SM material with low factors of safety when using the N-value approach. The evaluation based on the results of the cyclic tests indicates that those soils have adequate factors of safety against potential liquefaction. While these zones may have been eliminated by drilling additional borings, 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.

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TABLE 1

SUMMARY OF LIQUEFACTION EVALUATION OF SILTY SANDS (SM) ALONG ERCW PIPELINE ROUTE[7]

Boring No.	Elev. (ft)	Blow Counts	Soil Type	L.L.[4]	PI	W.C. %	D_{50} mm	Fines[1] Content %	F.S.[2] N-value	F.S.[2] Lab. Tests	Remarks
SS-49A	696.7	5	SM	29.0	4.0	32.4	0.08	47.	1.12	1.33	(3), (4)
SS-49A	690.7	6	SM	NP	NP	30.0	0.12	31.	1.05	1.21	(3), (4)
SS-131	699.9	4	SM	31.0	7.0	28.1	--	--	1.12	1.44	(4)
SS-131	697.9	5	SM	26.0	3.0	30.1	0.08	45.	1.13	1.37	(4)
SS-131	695.9	5	SM	26.0	3.0	29.7	0.08	45.	1.08	1.31	(4)
SS-131	691.9	7	SM	NP	NP	24.0	0.08	45.	1.16	1.25	(4)
SS-50	697.8	5	SM	NP	NP	28.2	0.098	43.	1.02	1.34	(3), (4)
SS-50	695.8	8	SM	NP	NP	29.1	0.093	43.	1.19	1.30	(3), (4)
SS-50	693.8	2	SM	NP	NP	31.5	0.089	47.	0.76	1.27	(3), (4)
SS-134	710.5	3	SM	NP	NP	29.3	0.148	26.	0.90	1.29	(4)
SS-134A	709.5	4	SM	23.0	1.0	30.0	0.105	35.	0.94	1.24	(4)
SS-135	704.9	8	SM	NP	NP	25.3	--	--	1.18	1.20	(4)
SS-135A	708.5	5	SM	NP	NP	34.2	0.12	29.	1.02	1.25	(4)
SS-65	708.0	7	SM	30.1	5.1	23.7	0.091	43.	1.13	1.21	(4)
SS-65	706.0	5	SM	28.9	3.5	28.2	0.140	34.	0.93	1.18	(4)
SS-65B	711.2	6	SM	25.0	1.0	27.5	0.090	41.	1.19	1.33	(4)
SS-65B	709.2	3	SM	25.0	1.0	33.1	0.100	38.	0.88	1.28	(4)
SS-65B	707.2	5	SM	25.0	1.0	34.0	0.100	34.	1.00	1.25	(4)
SS-65B	705.2	7	SM	26.0	2.0	27.1	0.075	50.	1.12	1.22	(4)
SS-136	710.9	5	SM	NP	NP	26.3	0.10	40.	1.13	1.37	(4)
SS-138A	707.2	4	SM	25.0	2.0	28.1	0.090	44.	0.98	1.31	(3), (4)
SS-139	705.5	14	SM	NP	NP	22.1	0.375	13.	1.12	--	(6), (3)

TABLE 1 (continued)

Boring No.	Elev. (ft)	Blow Counts	Soil Type	L.L.(4)	PI	W.C. %	D_{50} mm	Fines(1) Content %	F.S.(2) N-value	F.S.(2) Lab. Tests	Remarks
SS-140	706.7	4	SM	NP	NP	38.7	0.11	36.	1.02	1.39	(4)
SS-158	711.5	2	SM	22.7	2.5	32.2	0.088	44.	0.81	1.29	(3)
SS-162	713.8	5	SM	NP	NP	34.3	0.115	36.	1.03	1.27	(5)
SS-163	717.0	3	SM	27.2	3.3	31.1	0.097	45.	0.95	1.40	(5)
SS-163	715.0	4	SM	29.7	4.7	33.5	0.090	43.	1.00	1.37	(5)
SS-163A	717.5	4	SM	30.0	3.0	36.3	0.080	45.	1.04	1.42	(5)
SS-80	721.2	5	SM	41.6	14.6	29.1	0.12	44.	1.12	1.58	(5)
SS-84	713.4	2	SM	24.8	2.2	30.1	0.11	42.	0.88	1.41	(5)
SS-128	712.1	2	SM	NP	NP	23.7	0.22	16.	0.60	1.25	(5)
SS-125	714.4	2	SM	NP	NP	20.0	0.13	8.	0.90	1.32	(5)
SS-25	715.6	2	SM	NP	NP	29.2	0.08	48.	0.99	1.54	(5)
SS-130	715.7	10	SM	NP	NP	17.8	0.24	23.	0.86	-	(5), (6)
SS-130	713.7	9	SM	NP	NP	15.5	0.29	15.	0.88	-	(5), (6)

Notes:
(1) Percent finer than 0.074 mm
(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 axial strain to induced cyclic stress ratio
(3) Restricted by companion borings or adjacent borings
(4) Eliminated by results of cyclic testing
(5) Eliminated by settlement analysis
(6) Cyclic strength was estimated from the empirical correlation for clean sands
(7) This table summarizes the blow counts that result in lower factors of safety

TABLE 2

SUMMARY OF LIQUEFACTION EVALUATION OF SILTS (ML) ALONG ERCW PIPELINE ROUTE(3)

Boring No.	Elev. (ft)	Blow Counts	Soil Type	L.L.(4)	PI	W.C. %	D_{50} mm	(1) Fines Content %	(2) F.S.
SS-140	708.7	3	ML	--	--	17.4	0.073	50.	1.69
SS-101	712.5	3	ML	24.7	2.0	31.9	0.072	53.	1.42
SS-161	712.4	5	ML	25.7	2.3	30.9	0.076	51.	1.42
SS-84	711.4	3	ML	24.5	1.3	31.4	0.07	52.	1.63
SS-27	713.1	3	ML	23.1	2.9	24.5	0.07	51.	1.76

Notes: The cyclic strength of silts was determined based on the results of the cyclic simple shear tests.

(1) Percent finer than 0.074mm

(2) Factor of safety is defined as a ratio of cyclic stress ratio causing ±3.75 percent shear strain to induced cyclic stress ratio.

(3) This table summarizes those split-spoon samples having blow counts less than 5.

(4) Liquid limits determined by the air-dry method.

TABLE 3

SUMMARY OF LIQUEFACTION EVALUATION OF CLAYEY SOILS ALONG ERCW PIPELINE ROUTE(3)

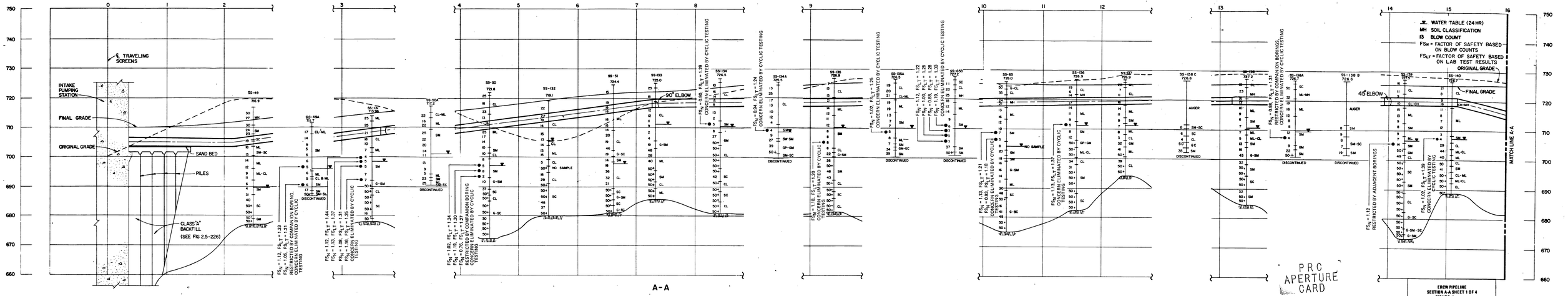
Boring No.	Elev. (ft)	Blow Counts	Soil Type	L.L.(4)	PI	W.C. %	D_{50} mm	(1) Fines Content	(2) F.S.
SS-137	706.9	3	CL	34.7	10.7	33.9	0.048	67.	1.74
SS-138C	708.6	4	CL	31.0	12.0	32.8	0.065	61.	1.78
SS-143	691.1	2	CL	31.2	15.5	--	--	--	1.81
SS-143A	701.0	3	SM-SC	21.0	5.0	21.2	0.093	45.	1.96
SS-143A	699.0	4	CL	25.0	8.0	24.9	0.065	55.	1.92
SS-143C	699.6	2	CL	23.0	8.0	23.7	0.04	62.	1.91
SS-143C	696.6	3	CL-ML	32.0	10.0	46.5	--	72.	1.85
SS-161	714.4	3	CL-ML	36.8	13.2	35.8	--	--	1.70
SS-165	716.7	3	SM-SC	30.7	8.1	33.3	--	--	2.00
SS-165	714.7	2	SM-SC	30.7	8.1	34.4	--	--	1.96
SS-167	711.7	2	CL	31.0	15.2	34.1	--	--	1.96
SS-168	713.6	1	CL	25.5	9.0	28.4	--	--	2.00
SS-127	714.2	2	CL	26.3	8.3	32.2	--	--	1.82
SS-127	712.2	0	SM-SC	23.3	4.4	36.1	--	--	1.74
SS-125	712.4	1	CL	25.6	8.3	33.7	--	--	1.72
SS-27	715.1	2	CL	29.3	12.4	31.8	--	--	2.09
SS-28	715.4	4	CL	31.3	12.0	31.6	--	--	2.09
SS-158A	712.6	4	CL	31.0	13.0	30.6	0.07	54.	1.80

Notes:
(1) Percent finer than 0.074mm

(2) Based on the cyclic strength of clayey soils estimated from the results of the cyclic simple shear tests.

(3) This table summarizes those clayey soils having water contents greater than 0.9 LL and blow counts less than 5.

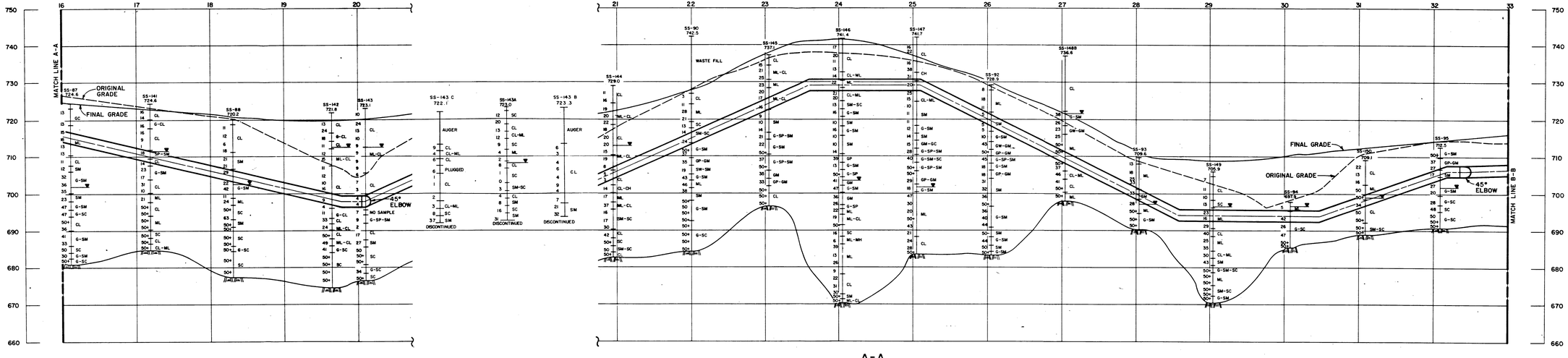
(4) Liquid limits determined by the air-dry method.



PRC APERTURE CARD

ERCW PIPELINE
SECTION A-A SHEET 1 OF 4
FIGURE 1

10-0902702018



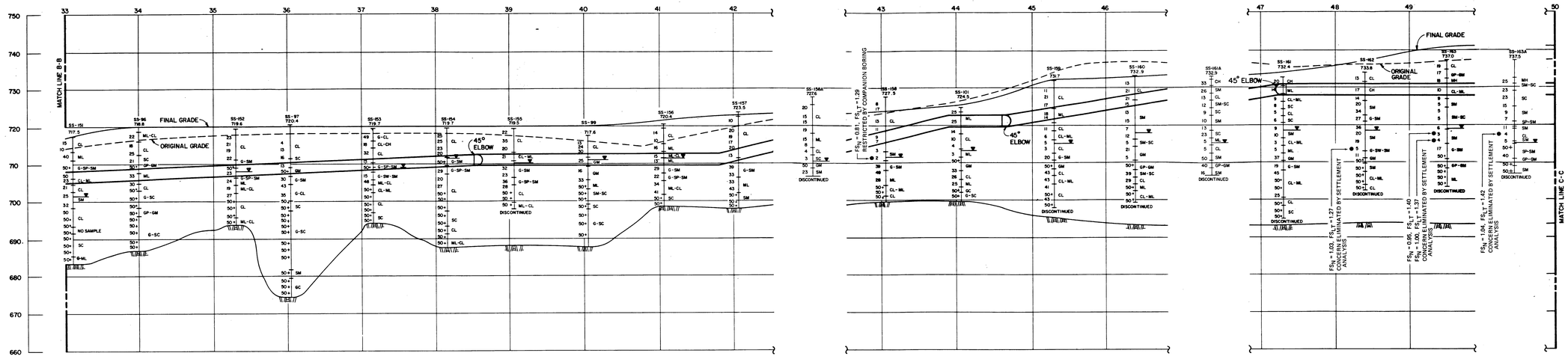
A-A

▼ WATER TABLE (24 HR)
MH SOIL CLASSIFICATION
13 BLOW COUNT
FS_N = FACTOR OF SAFETY BASED ON BLOW COUNT
FS_{LT} = FACTOR OF SAFETY BASED ON LAB TEST RESULTS

PRC APERTURE CARD

ERCW PIPELINE
SECTION A-A SHEET 2 OF 4
FIGURE 1

82128270290-02

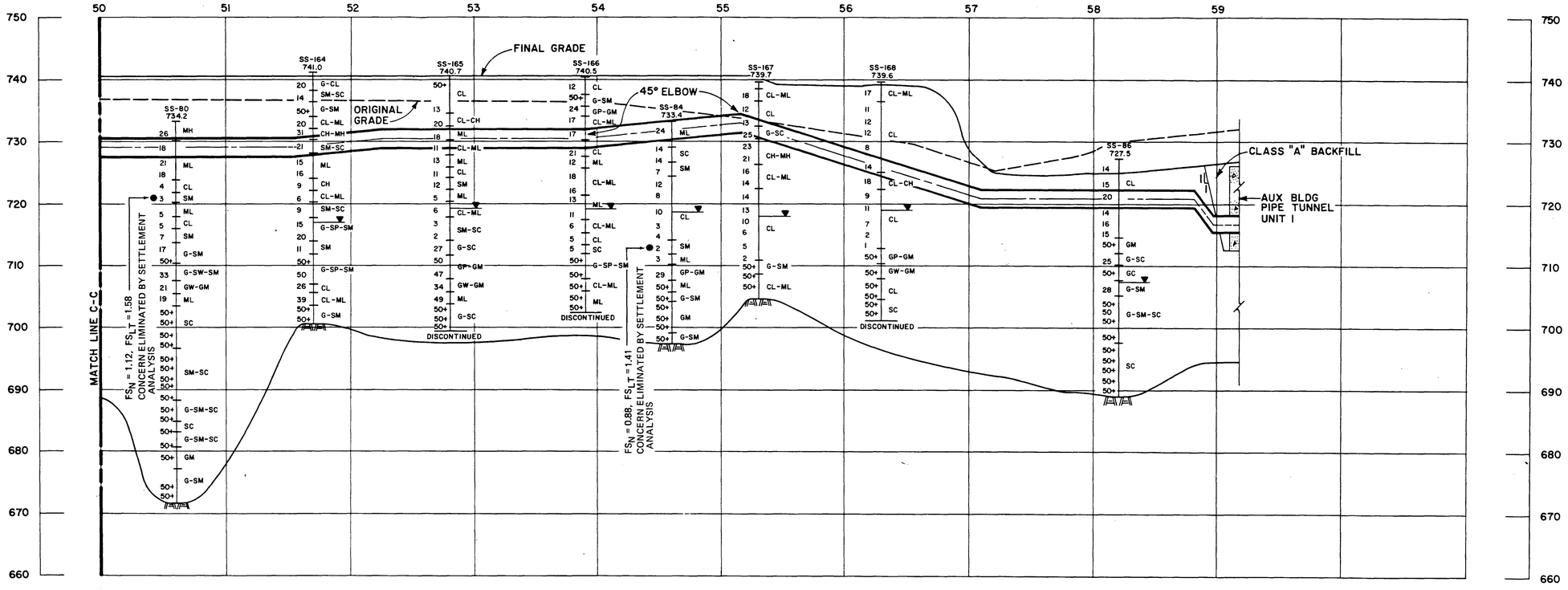


▼ WATER TABLE (24 HR)
MH SOIL CLASSIFICATION
15 BLOW COUNT
FS_M = FACTOR OF SAFETY BASED ON BLOW COUNT
FS_{LT} = FACTOR OF SAFETY BASED ON LAB TEST RESULTS

P.R.C. APERTURE CARD

8212270290-03

ERCW PIPELINE
SECTION A-A SHEET 3 OF 4
FIGURE 1



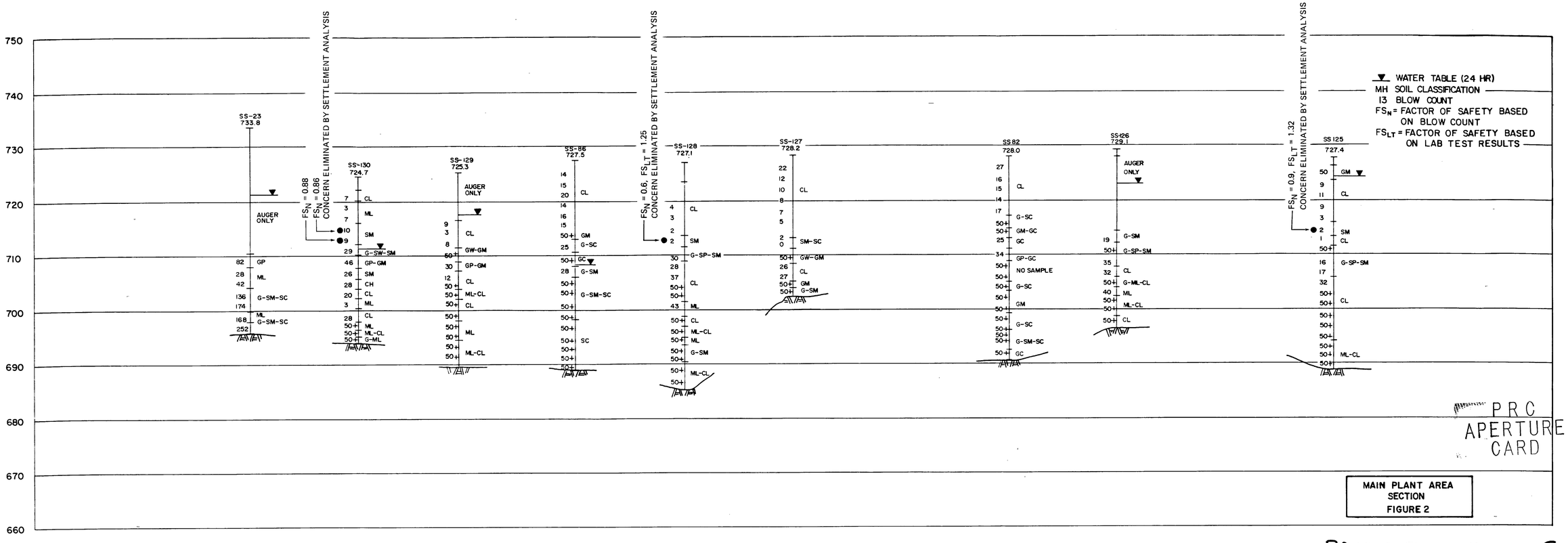
A-A

▼ WATER TABLE (24 HR)
MH SOIL CLASSIFICATION
13 BLOW COUNT
FS_N = FACTOR OF SAFETY BASED ON BLOW COUNT
FS_{LT} = FACTOR OF SAFETY BASED ON LAB TEST RESULTS

PRC APERTURE CARD

ERCW PIPELINE
SECTION A-A SHEET 4 OF 4
FIGURE I

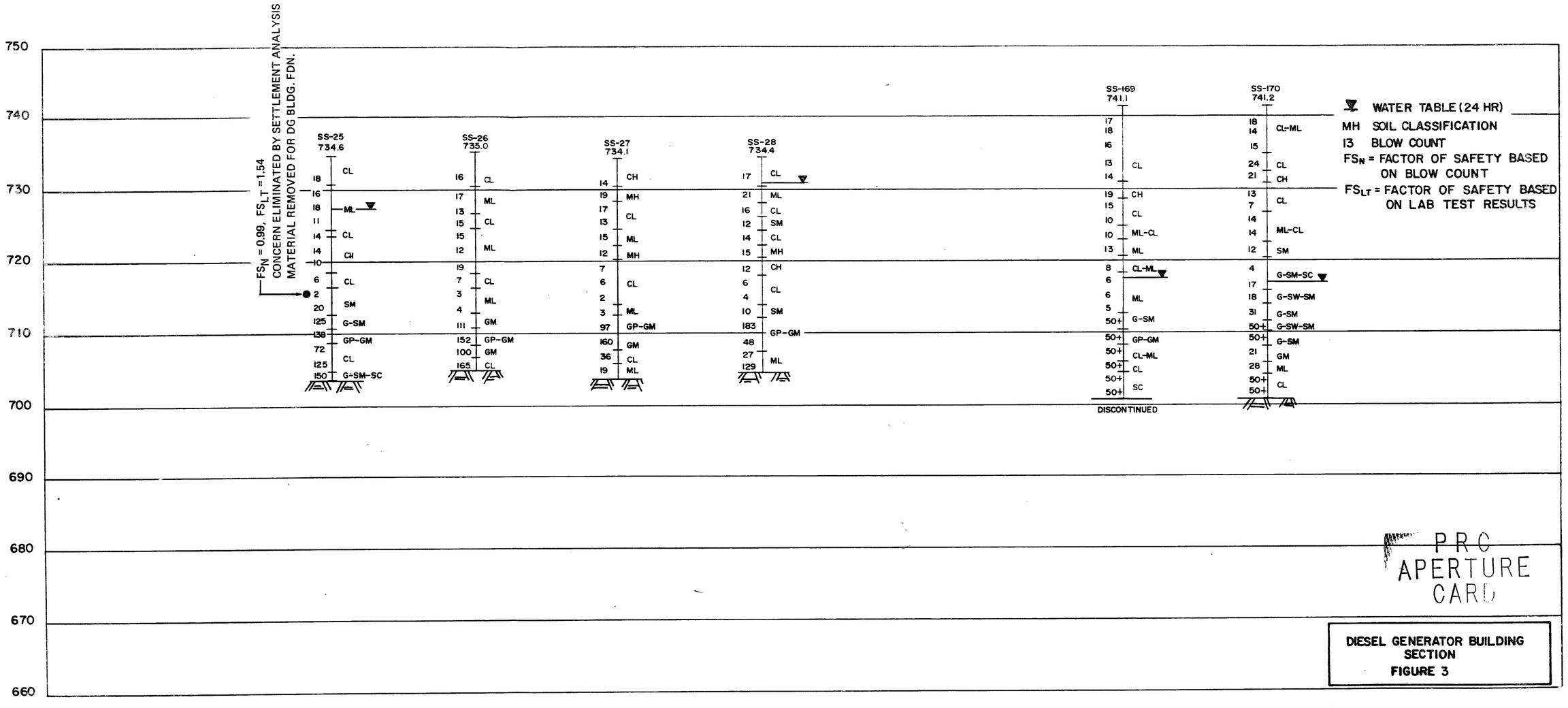
8212270290-04



PRC APERTURE CARD

MAIN PLANT AREA SECTION
FIGURE 2

8212270290-05



PRC
APERTURE
CARD

DIESEL GENERATOR BUILDING
SECTION
FIGURE 3

8212270290-06

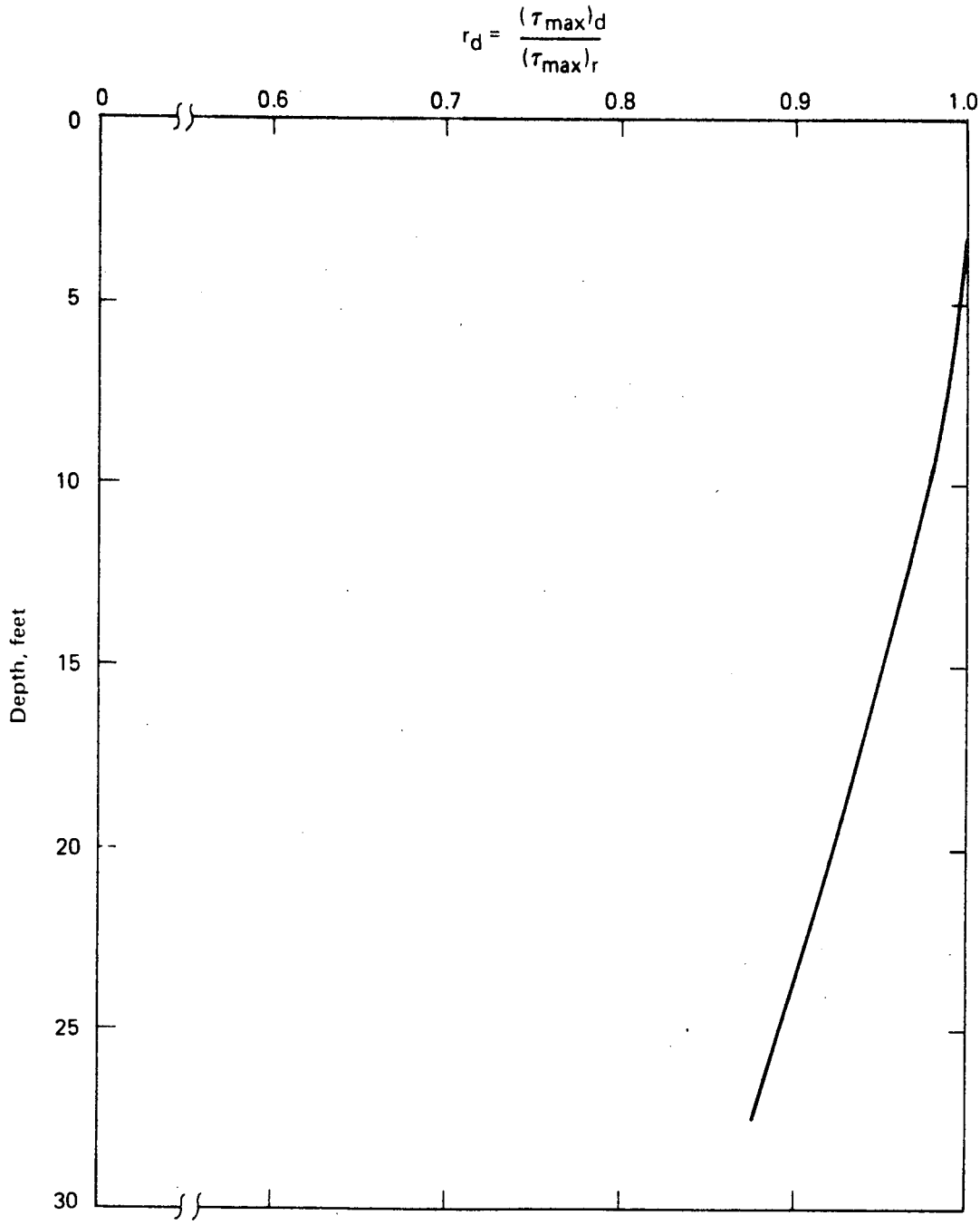


Figure 4 – Site-Specific Stress Reduction Coefficient, r_d, for Watts Bar Nuclear Plant Site

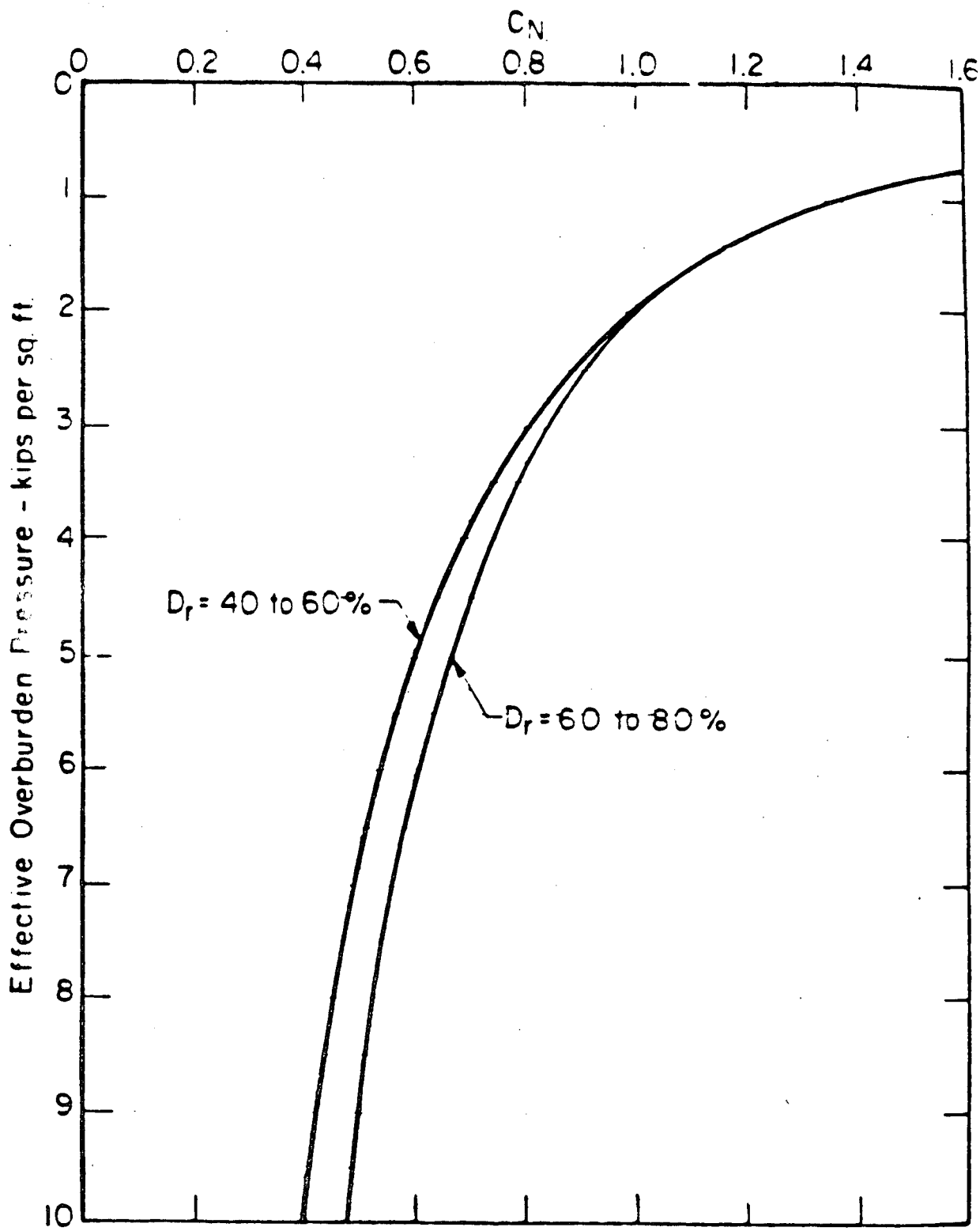


Figure 5 — Recommended Curves for Determination of C_N Based on Averages for W.E.S. Tests (After Seed and Idriss, 1981)

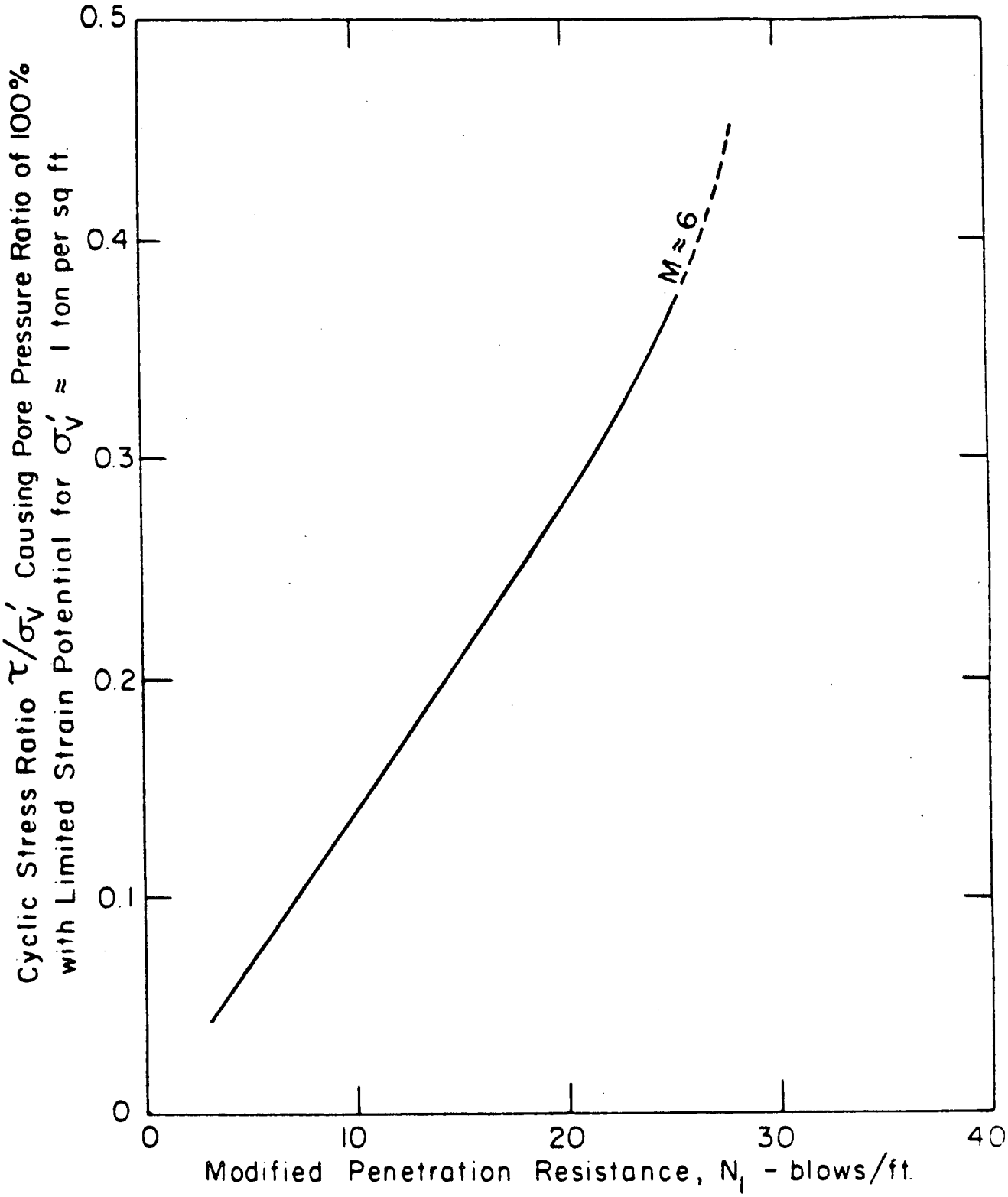


Figure 6 — Chart for Evaluation of Liquefaction Potential for M = 6 Earthquake (From Seed and Idriss, 1981)

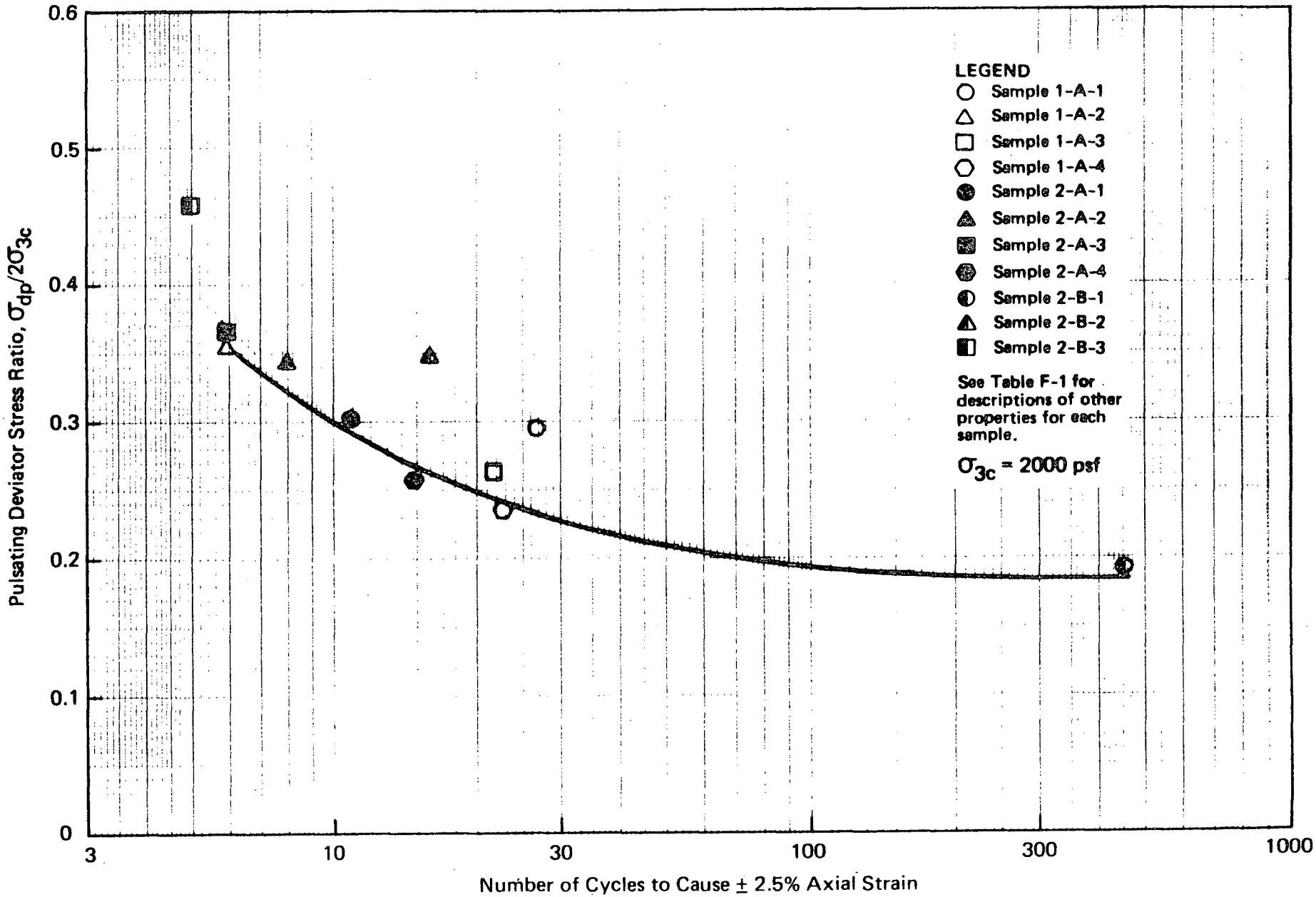


Figure 7 — Results of Cyclic Triaxial Tests on Silty Sands (SM)

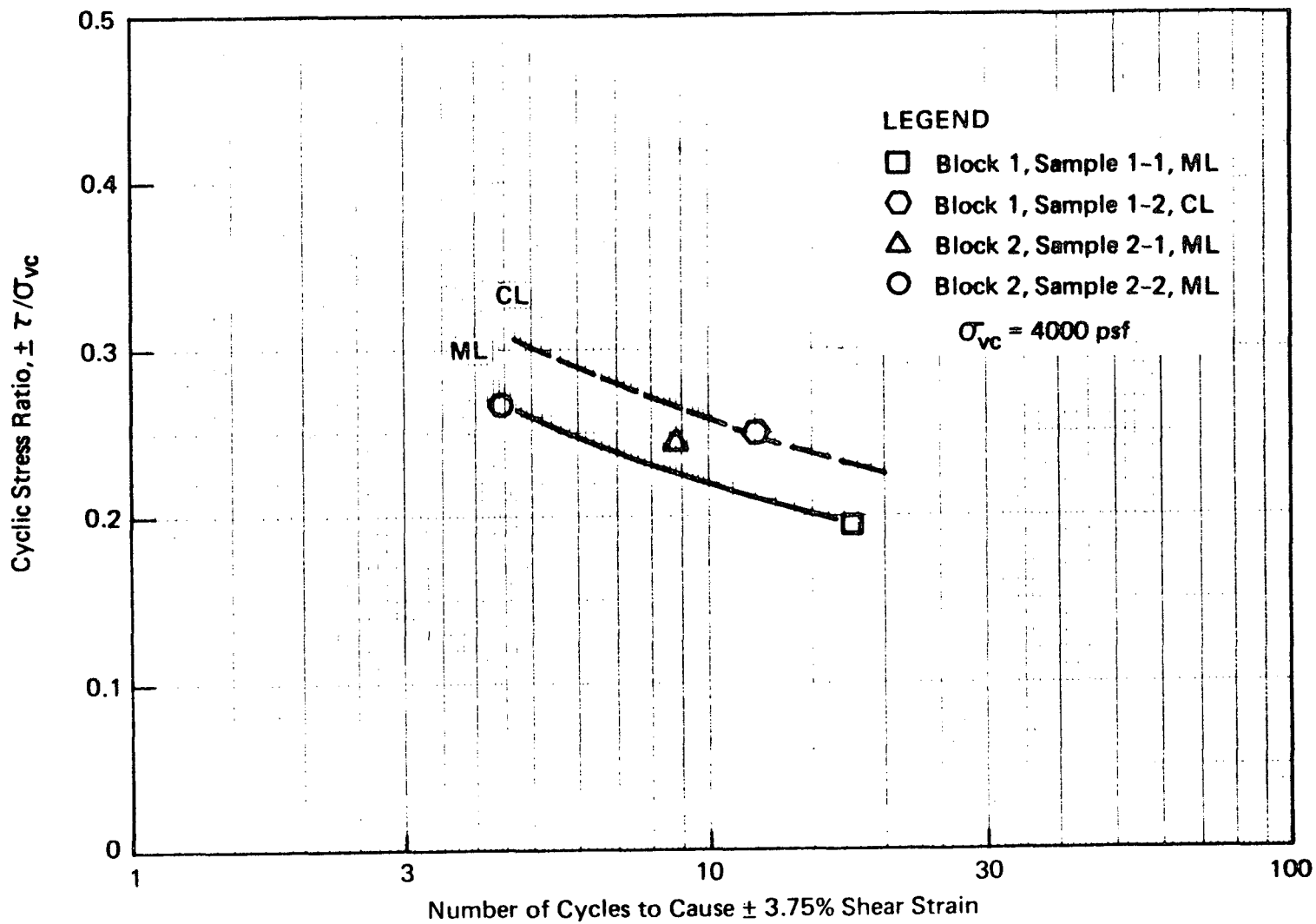
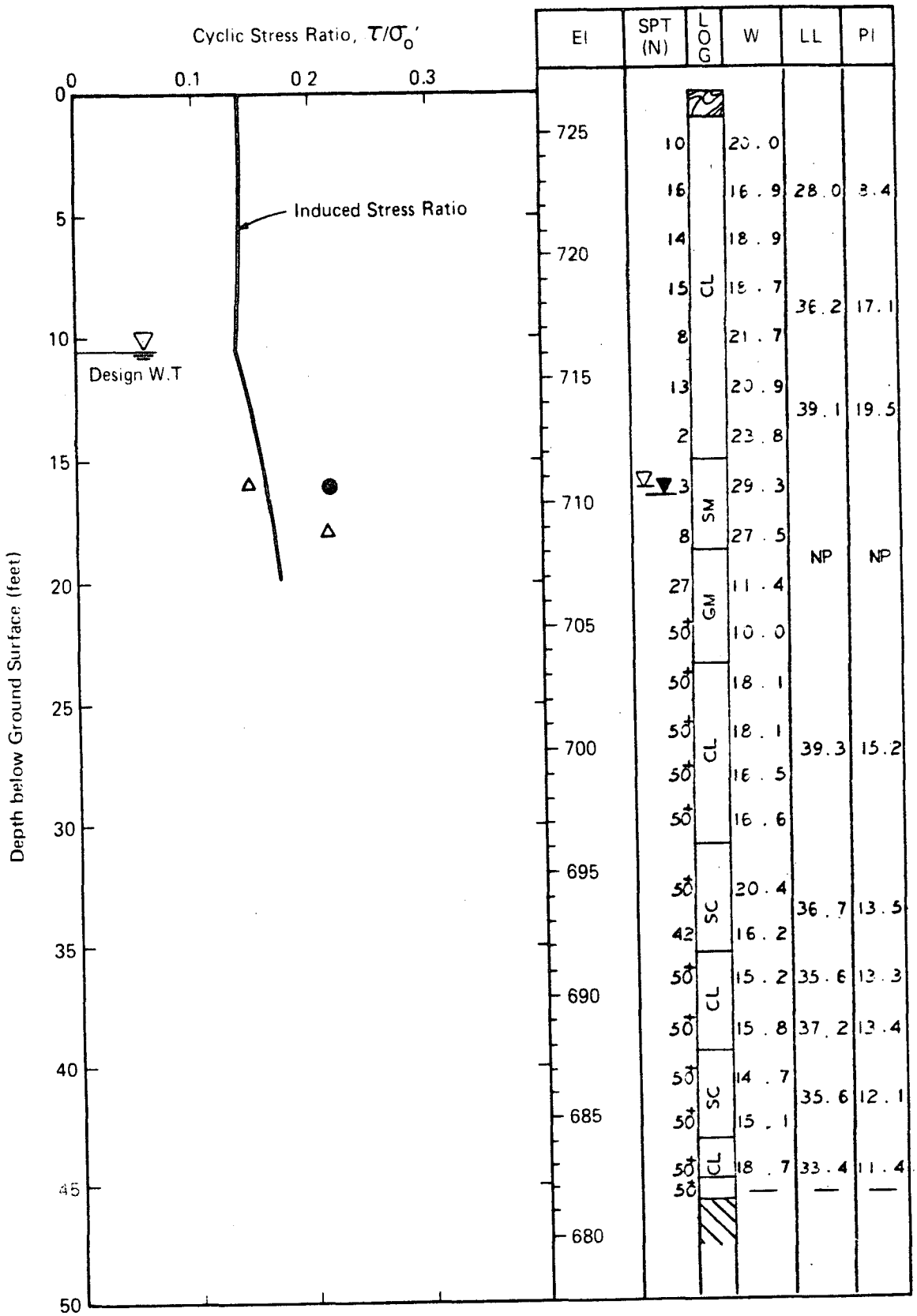


Figure 8 — Results of Cyclic Simple Shear Tests on Clayey Silts and Silty Clays (ML–CL) — Cyclic Stress Ratio and Number of Cycles to Cause Shear Strain = ±3.75 Percent

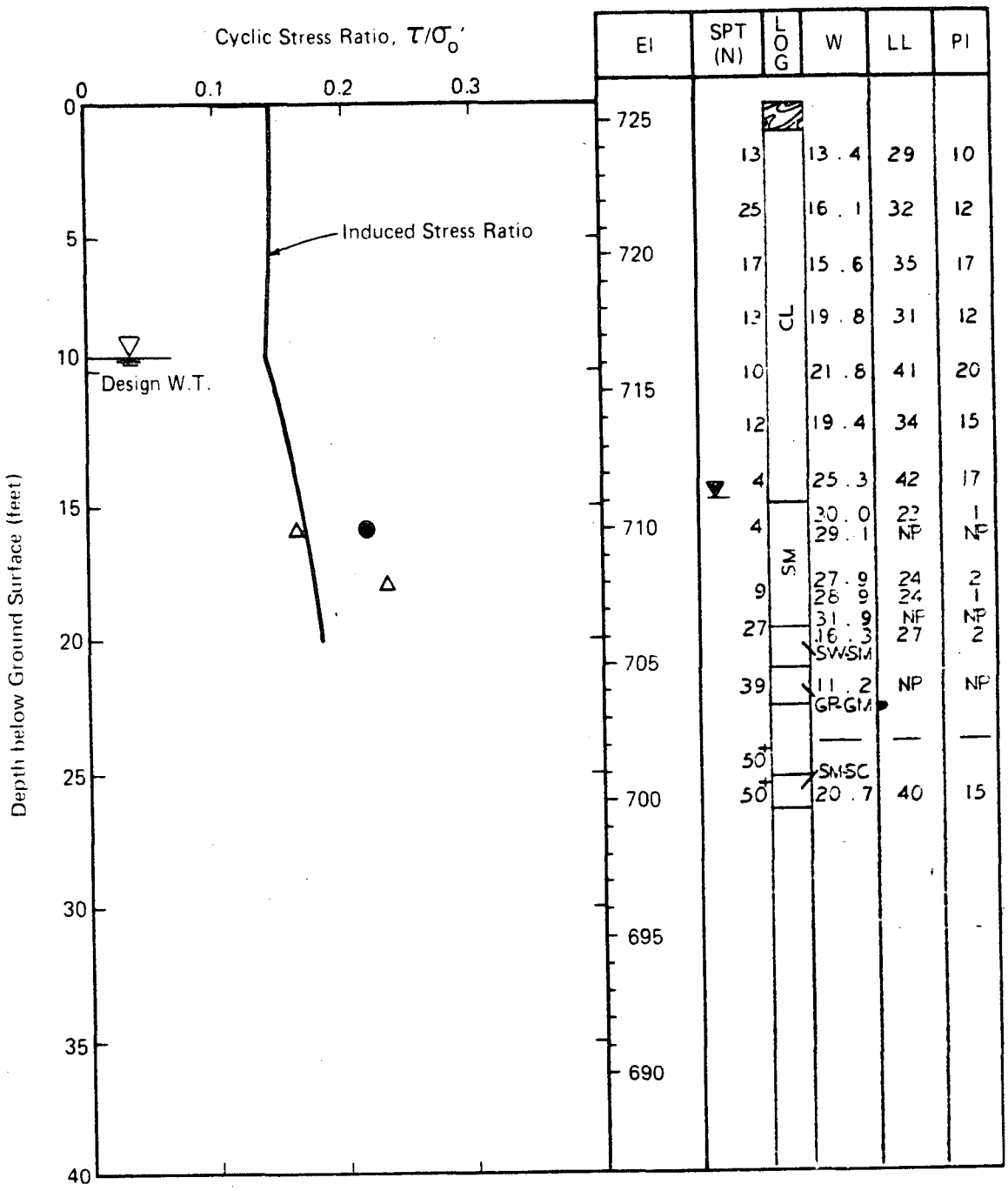


LEGEND

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

● Cyclic strength of silty sands (SM) based on cyclic test results

Figure 9 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Liquefaction – Boring SS–134



LEGEND

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

● Cyclic strength of silty sands (SM) based on cyclic test results

Figure 10 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Liquefaction – Boring SS–134A

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

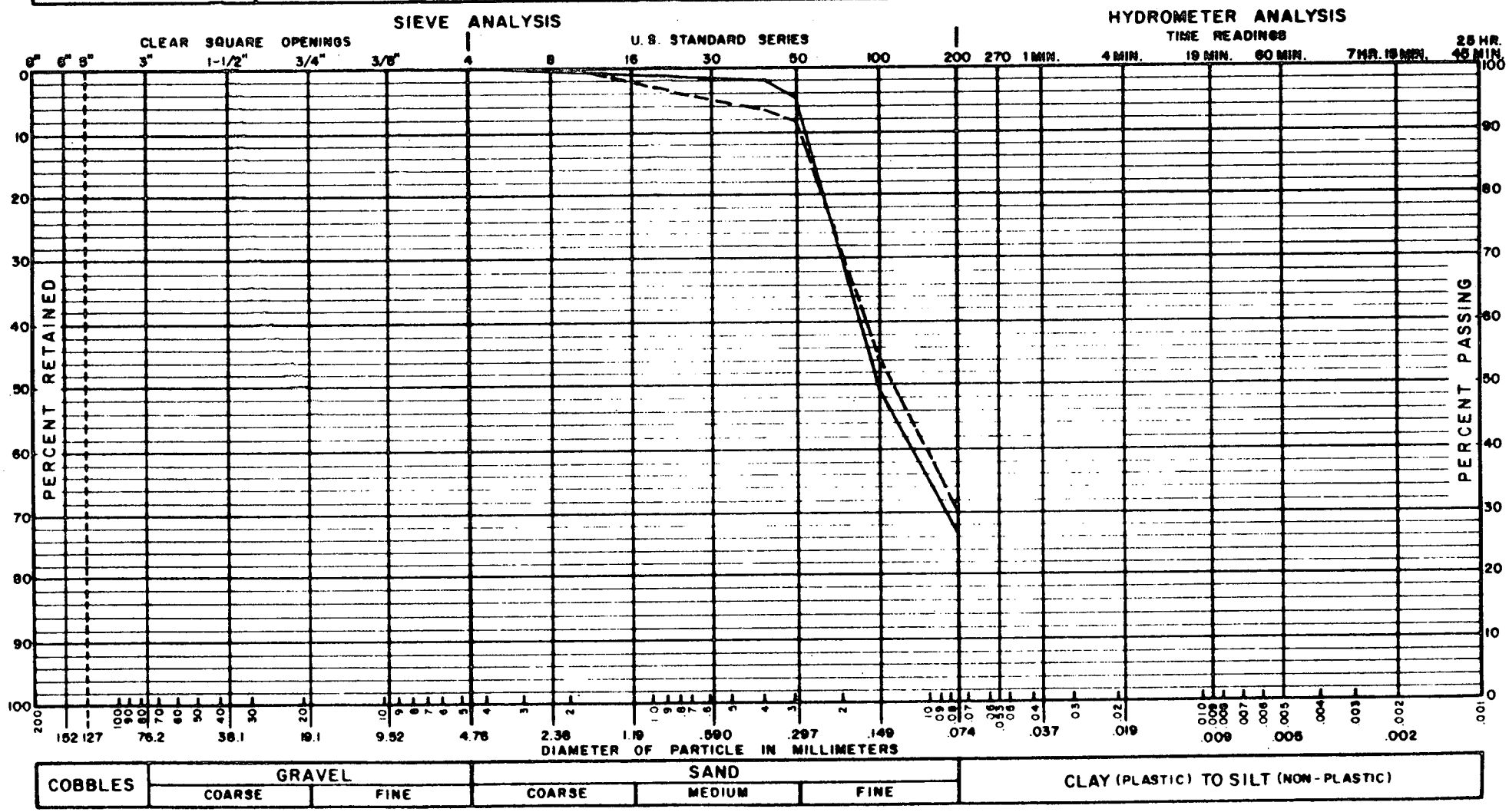


Figure 11 – Grain Size Distribution Curves for Silty Sand Samples from Boring SS–134

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

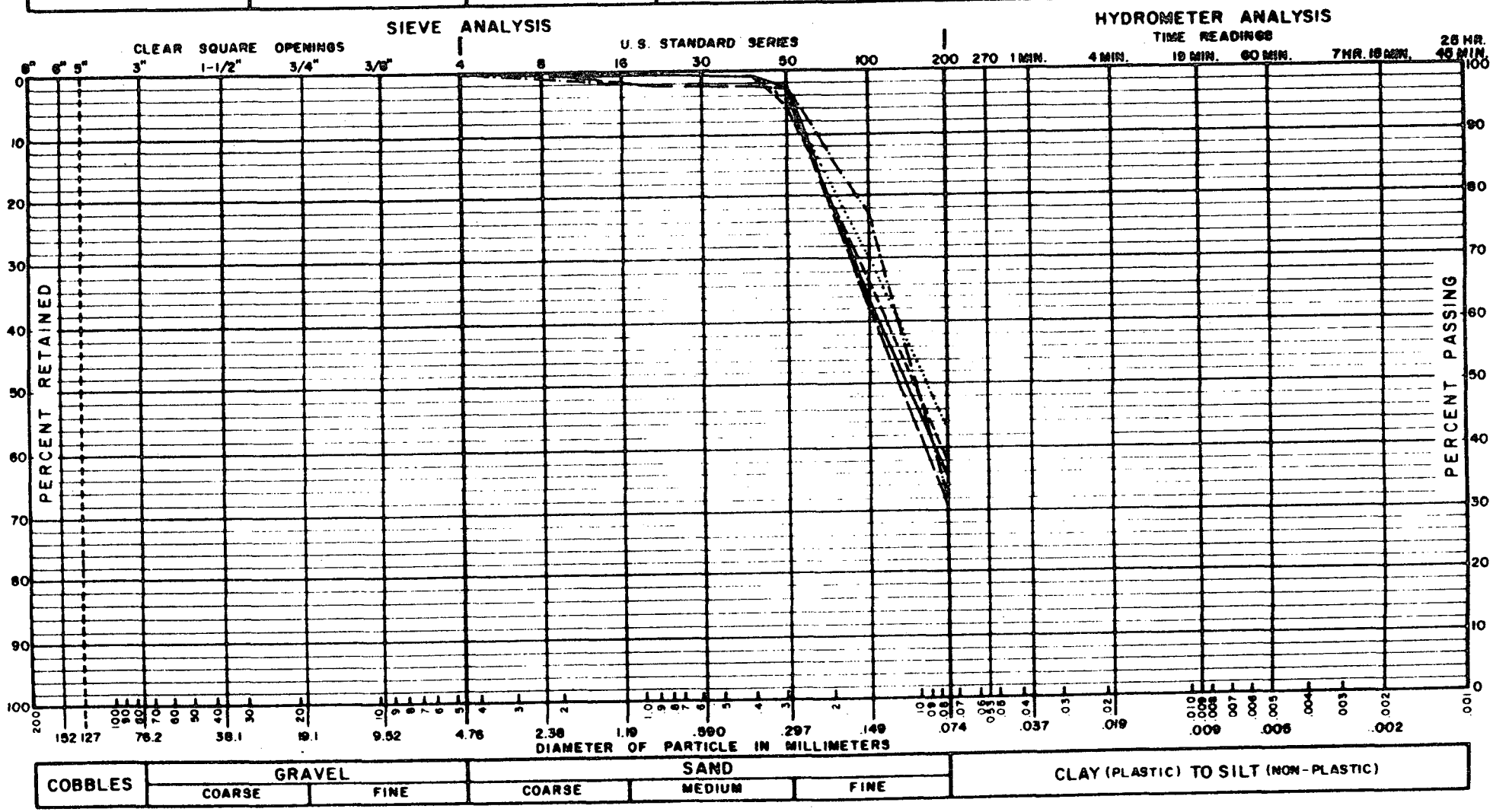
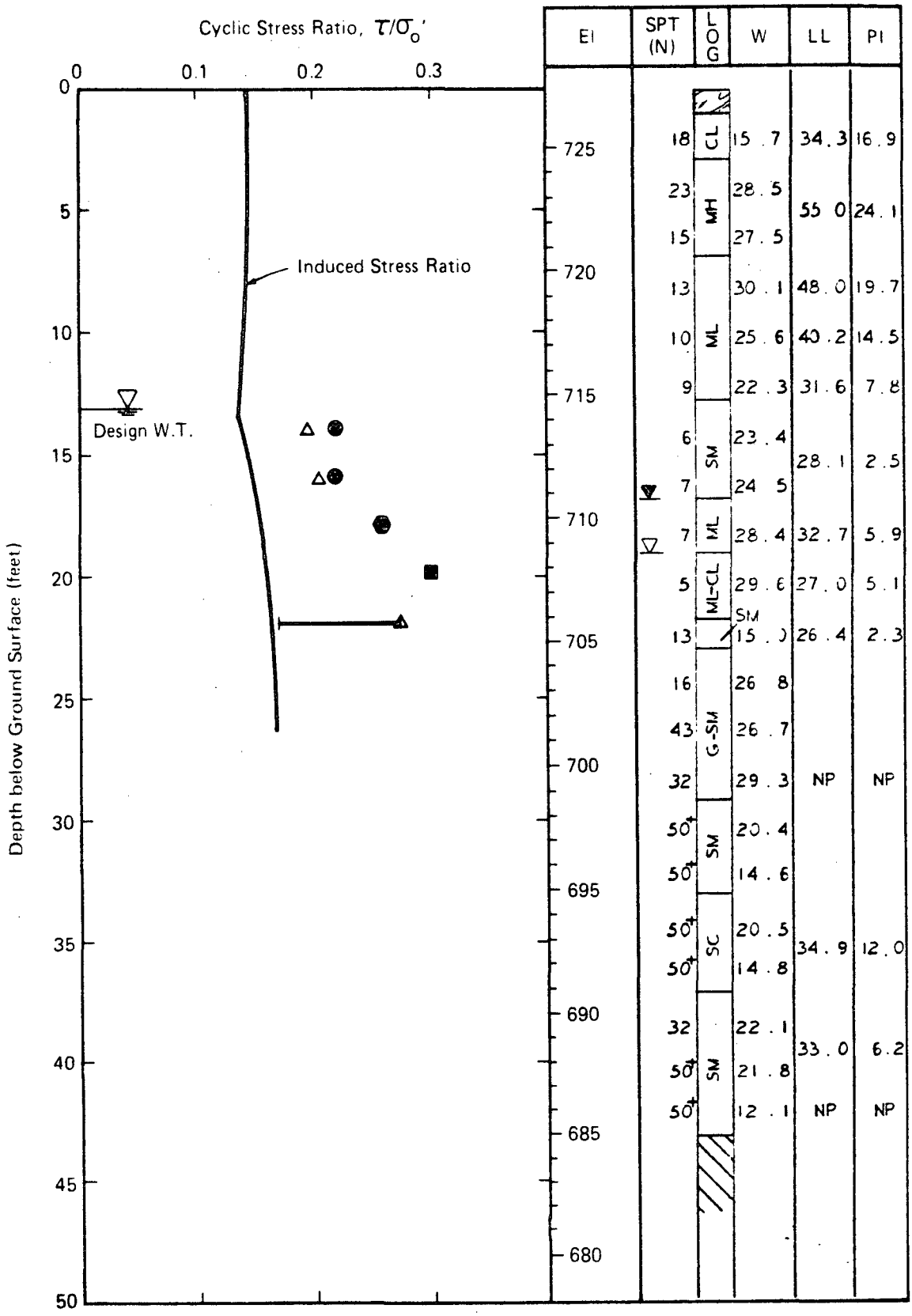


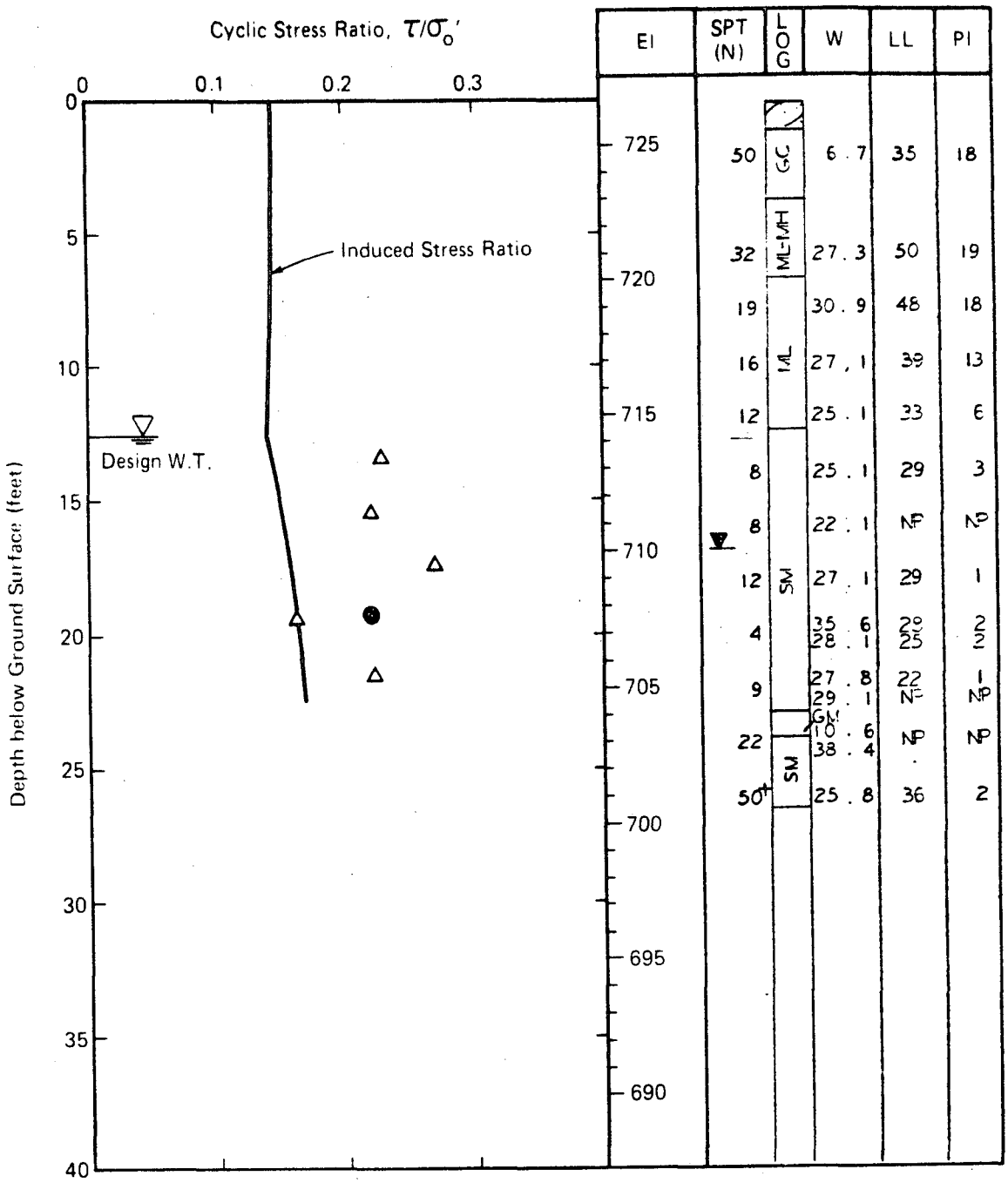
Figure 12 – Grain Size Distribution Curves for Silty Sand Samples from Boring SS–134A



LEGEND

- △ Cyclic strength (cyclic stress ratio causing pore pressure ratio of 100%) based on N–values
- ⊢—⊣ Possible range of cyclic strength based on N–values
- ● Cyclic strength of silty sands (SM) based on cyclic test results
- ● Cyclic strength of clayey silts (ML) based on cyclic test results
- ■ Cyclic strength of silty clays (CL) based on cyclic test results

Figure 13 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Liquefaction – Boring SS–138



LEGEND

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

● Cyclic strength of silty sands (SM) based on cyclic test results

Figure 14 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Liquefaction – Boring SS–138A

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

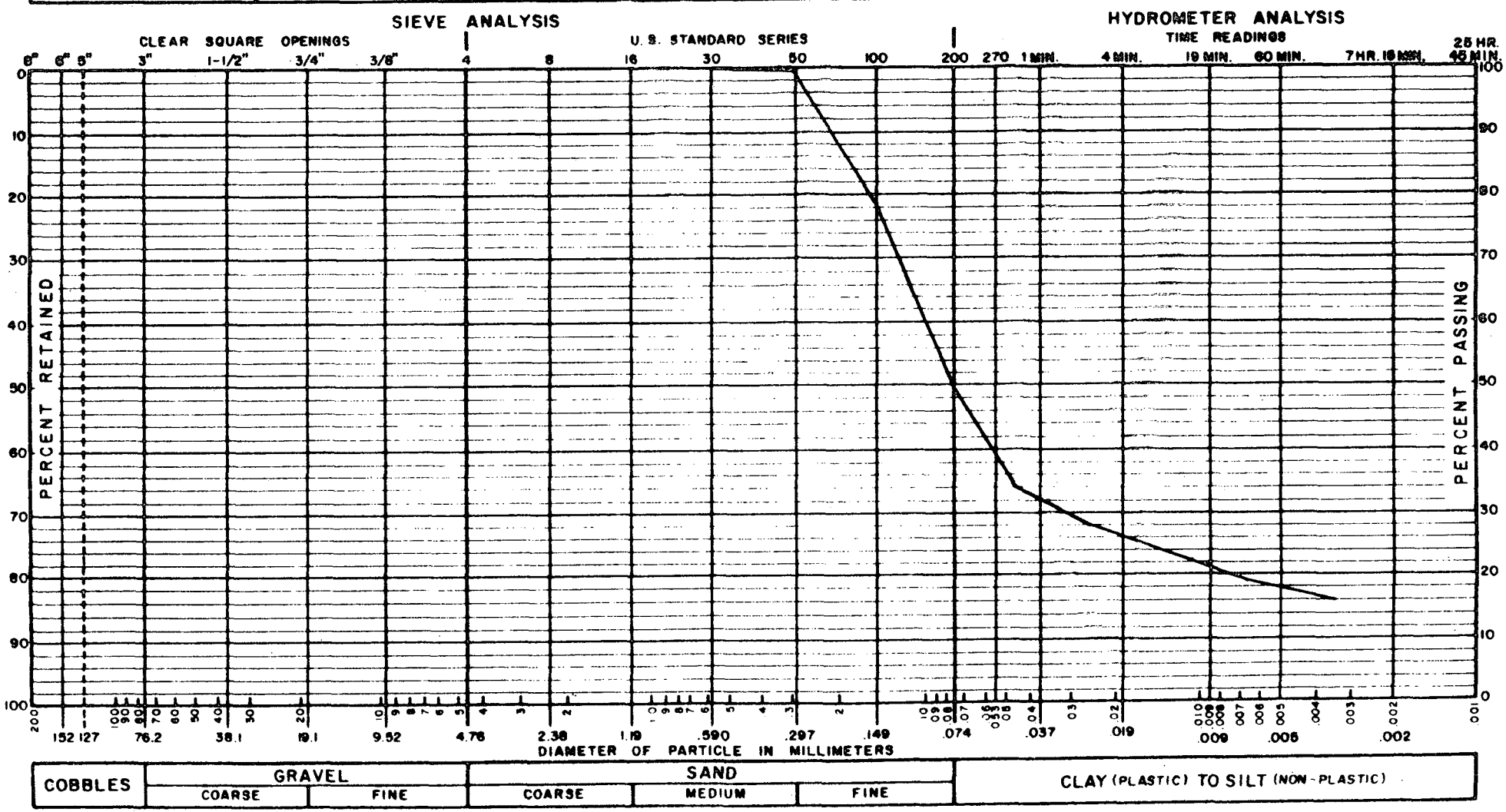


Figure 15 – Grain Size Distribution Curve for Silty Sand Sample from Boring SS–138

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

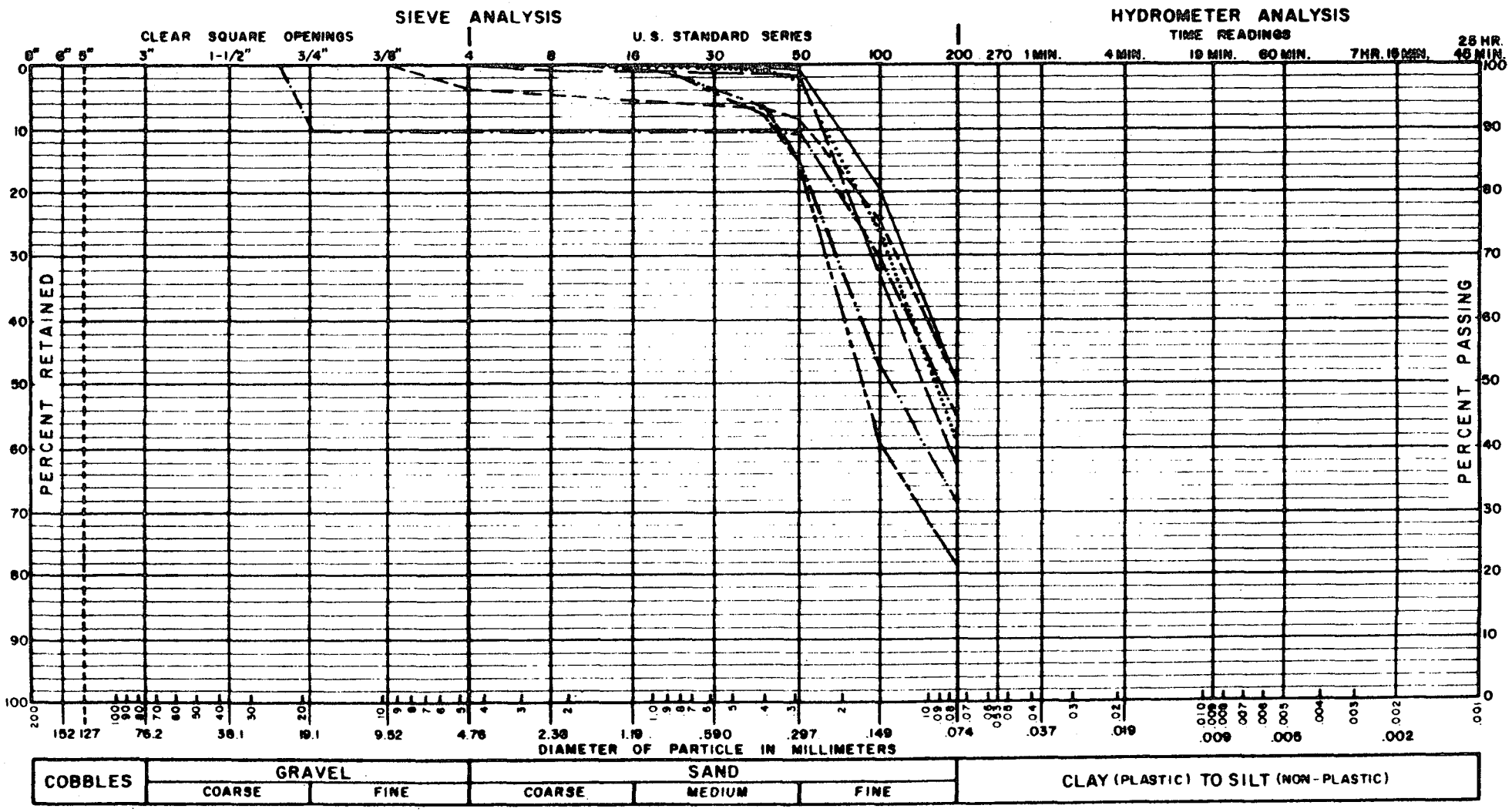
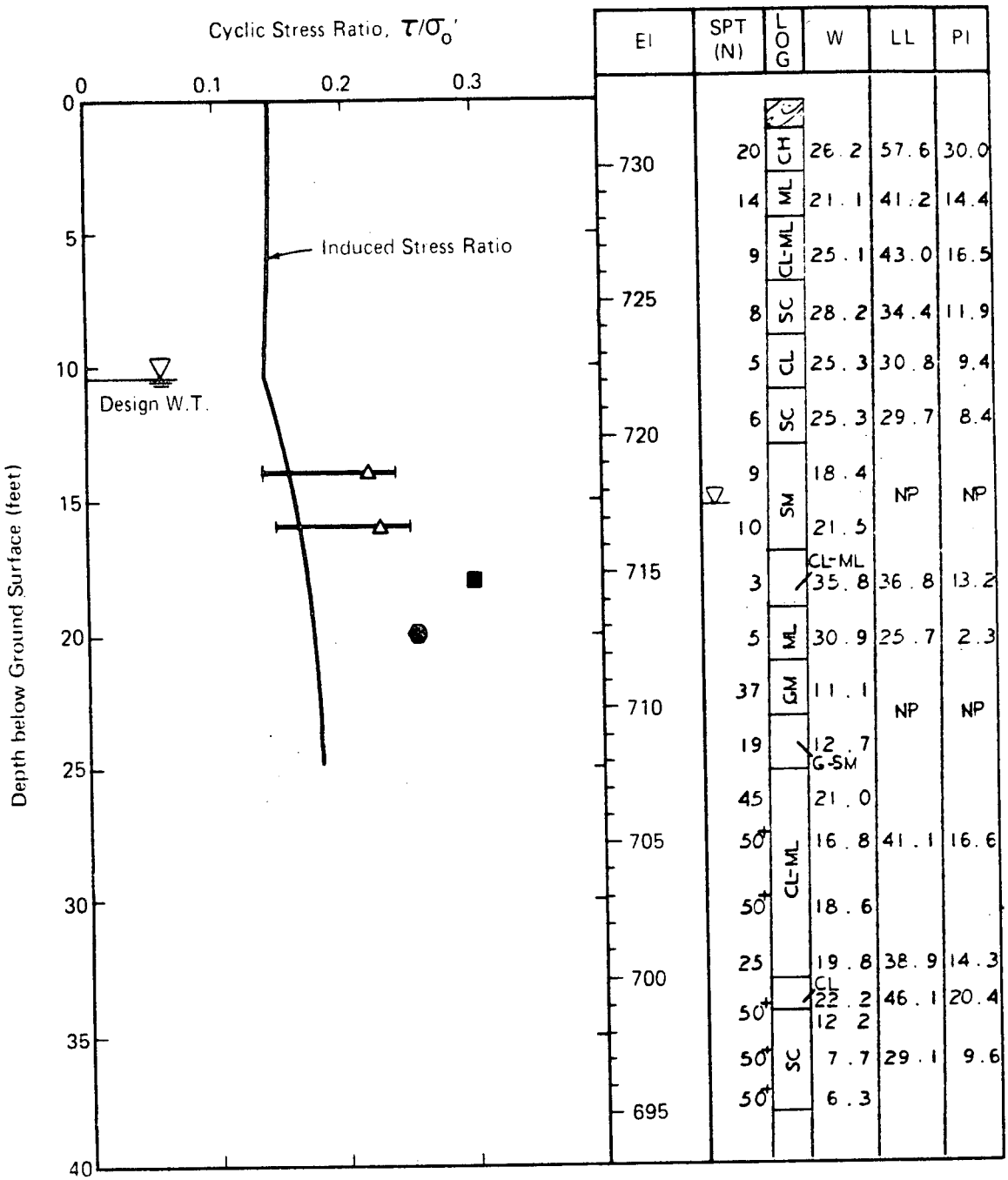


Figure 16 — Grain Size Distribution Curves for Silty Sand Samples from Boring SS–138A



LEGEND

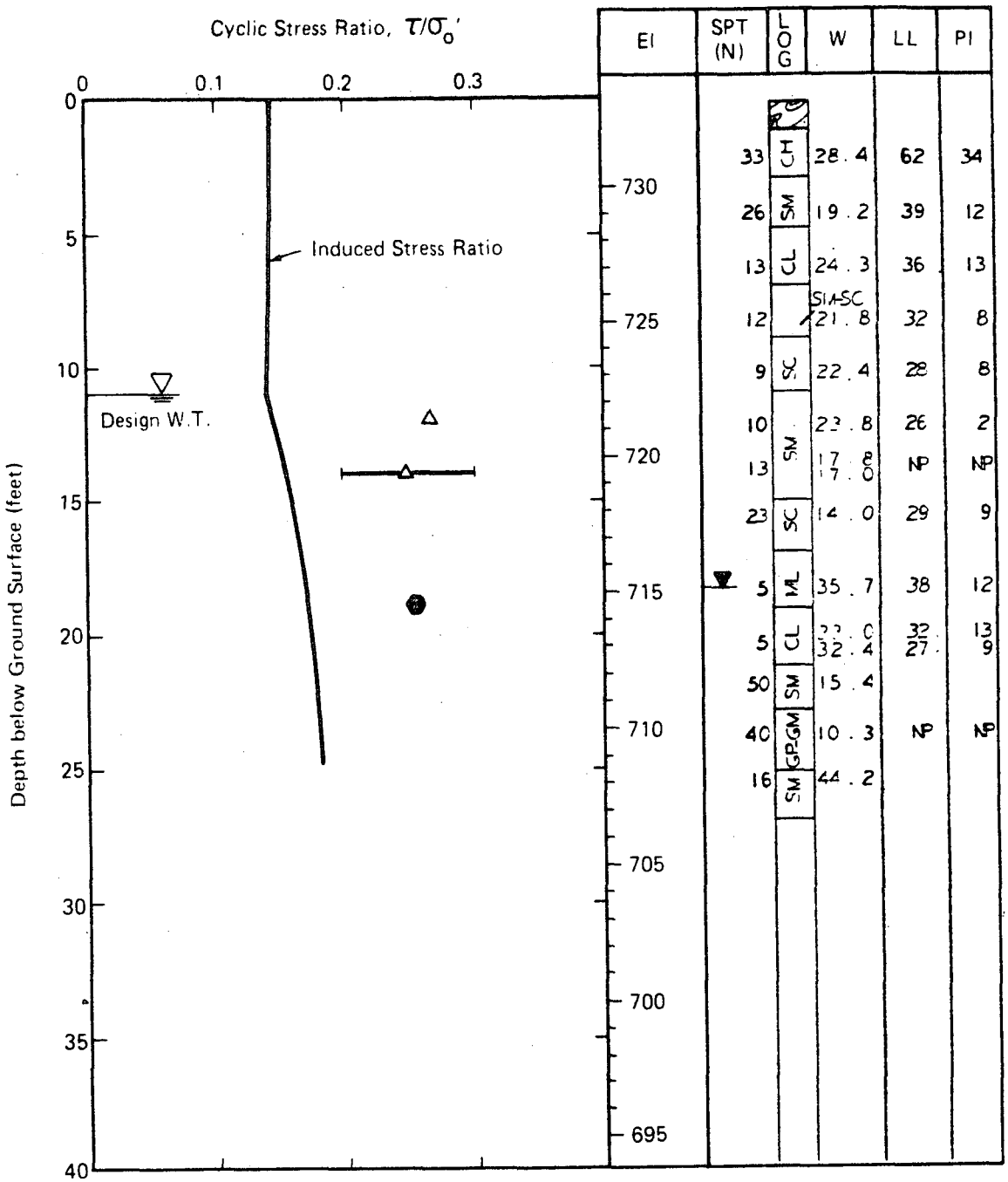
△ Cyclic strength (cyclic stress ratio causing pore pressure ratio of 100%) based on N–values

Possible range of cyclic strength based on N–values

Cyclic strength of clayey silts (ML) based on cyclic test results

Cyclic strength of silty clays (CL) based on cyclic test results

Figure 17 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Liquefaction – Boring SS–161



LEGEND

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

Possible range of cyclic strength based on N–values

Cyclic strength of clayey silts (ML) based on cyclic test results

Figure 18 – Comparison of Induced Stress Ratio with Cyclic Stress Ratio Causing Liquefaction – Boring SS–161A

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)		— — —			

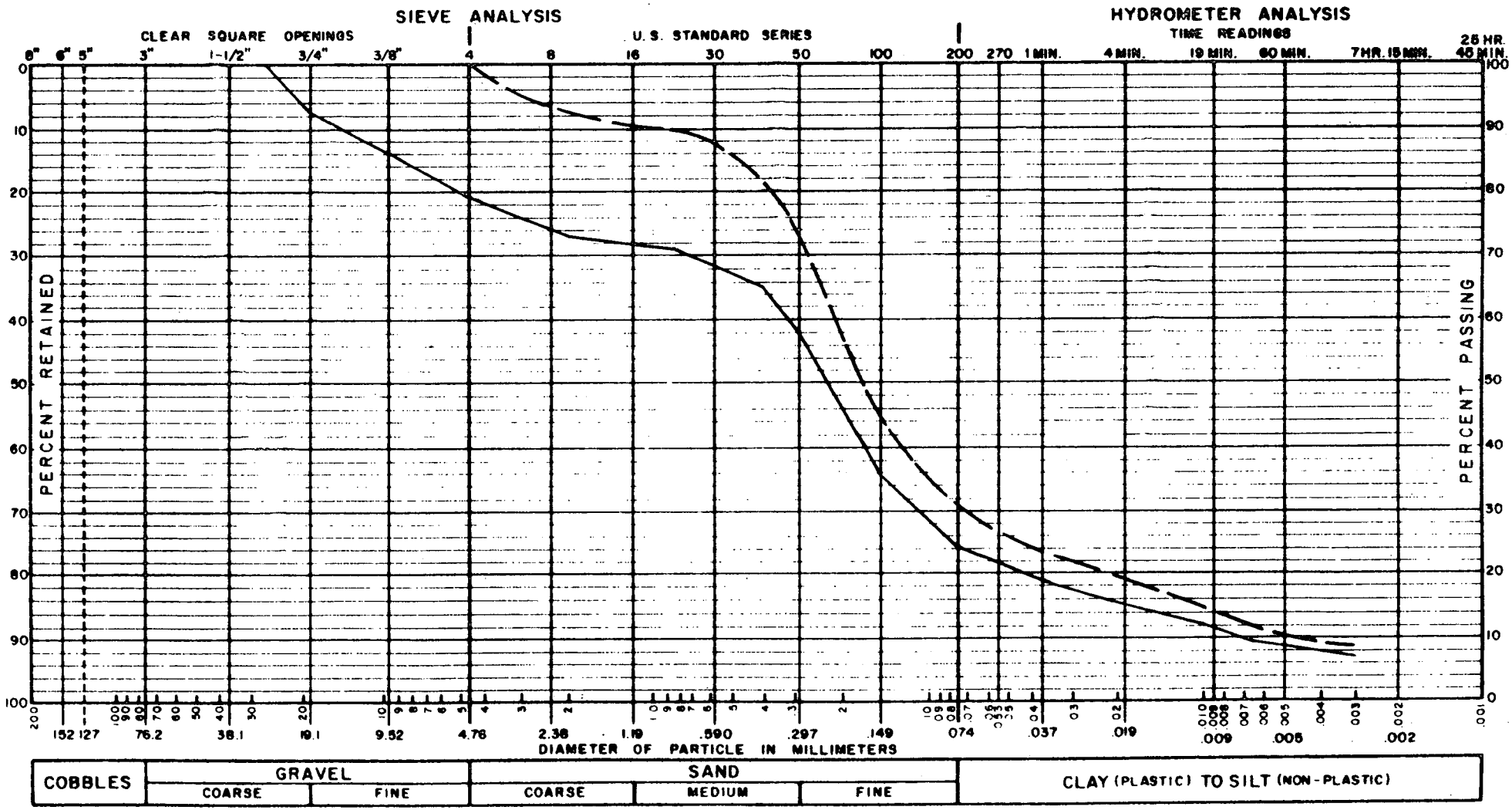


Figure 19 – Grain Size Distribution Curve for Silty Sand Sample from Boring SS–161

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

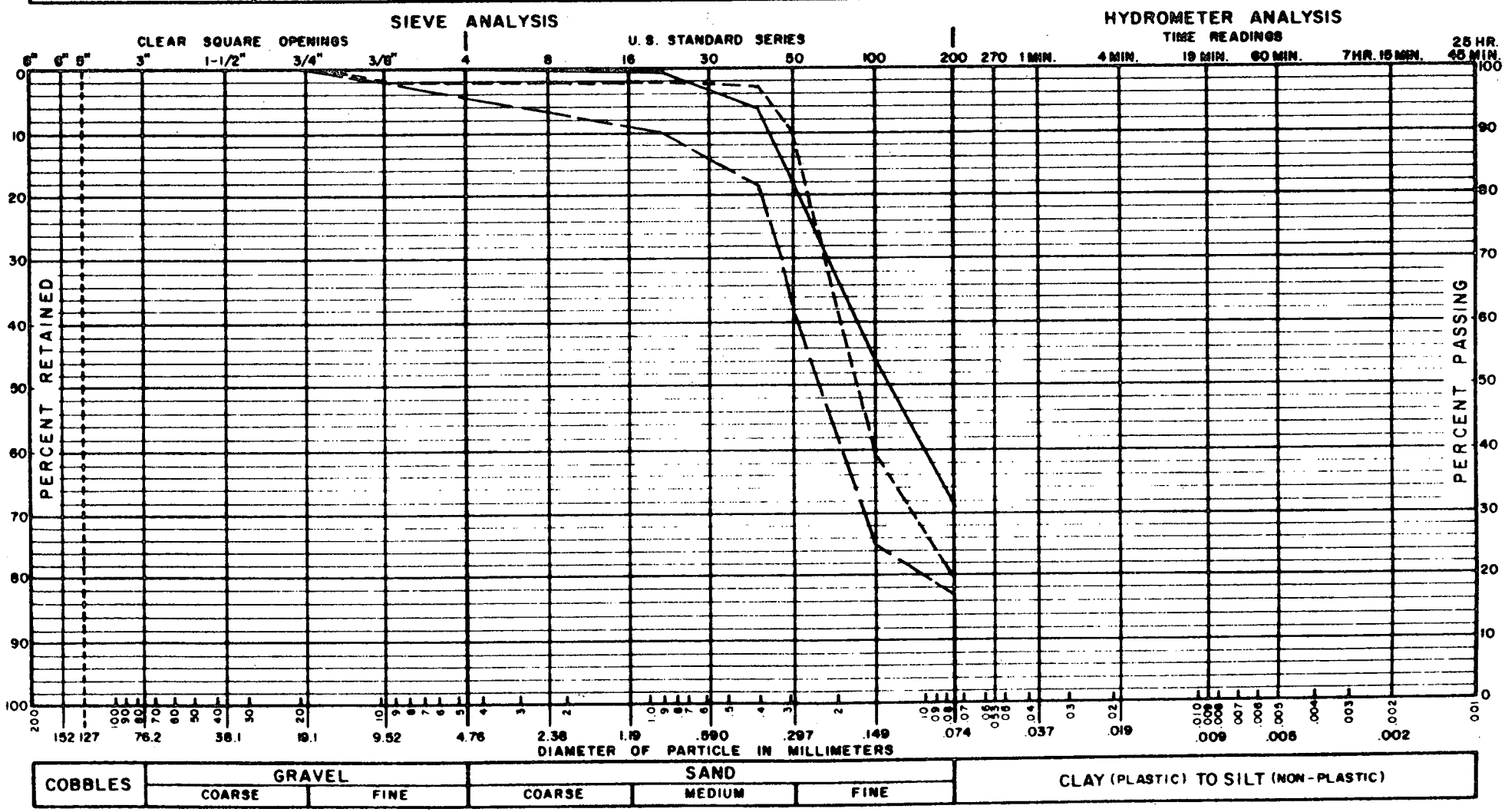
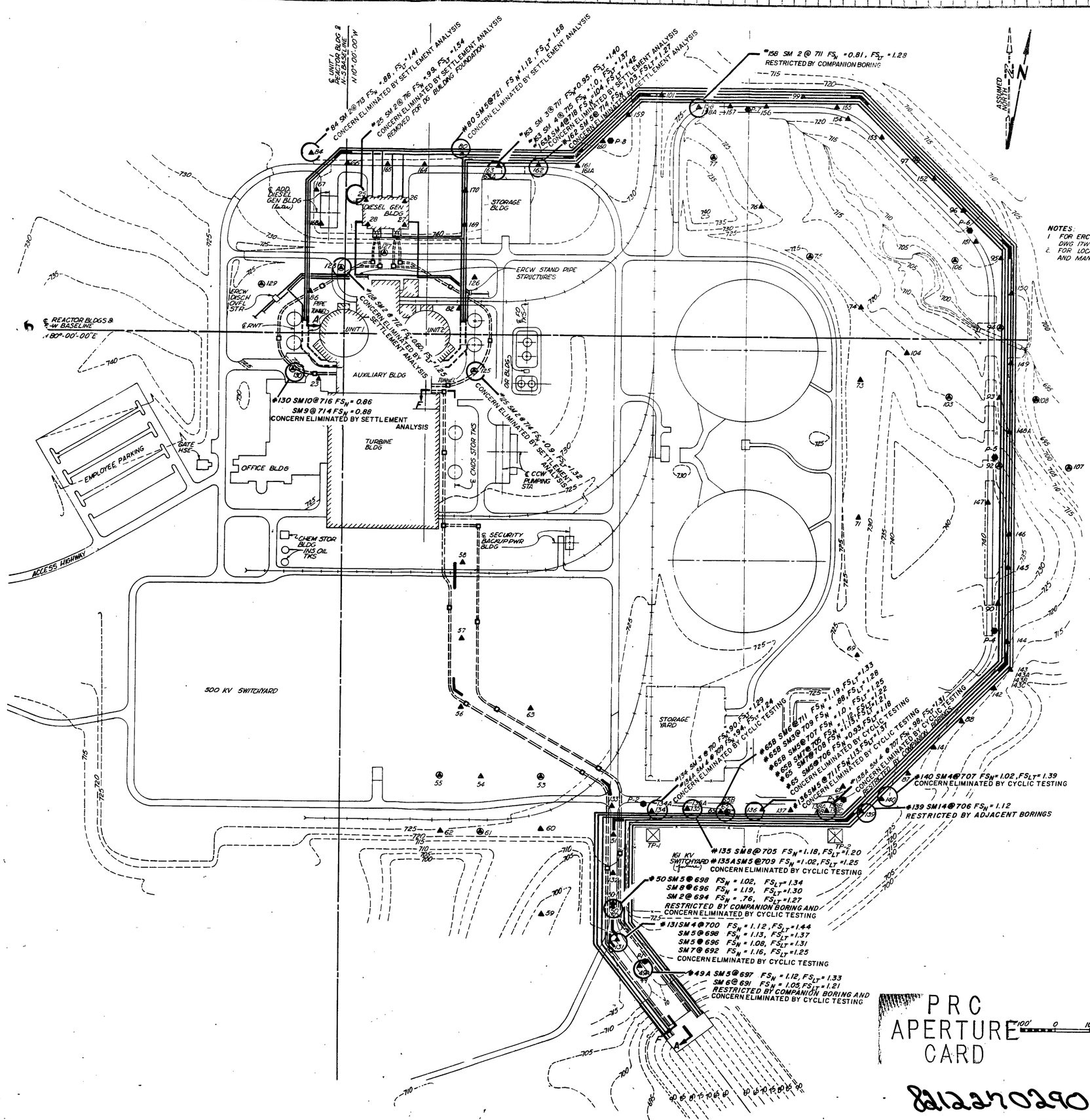


Figure 20 – Grain Size Distribution Curves for Silty Sand Samples from Boring SS–161A



NOTES:
1. FOR ERCW MISSILE PROTECTION CONCRETE, SEE MECH DWG 17W302.
2. FOR LOCATION OF CLASS 1E ELECTRICAL CONDUIT BANKS AND MANHOLES SEE 15W810-1.

LEGEND:
- SPLIT SPOON AND UNDISTURBED BORING
- SPLIT SPOON BORING
- SAFETY RELATED PIPE
- ELECTRICAL MANHOLE
- NON-SAFETY RELATED PIPE
- PIEZOMETER
- TESTPIT

#138 A SM 4 @ 707 FSN = .98, FSLT = 1.31 - BORING, SOIL CLASSIFICATION, BLOW COUNT, ELEVATION, FACTOR OF SAFETY

FSN = FACTOR OF SAFETY BASED ON BLOW COUNT

FSLT = FACTOR OF SAFETY BASED ON LABORATORY TEST RESULTS

PRC APERTURE CARD

8212270290-07

SCALE: 1" = 100' EXCEPT AS NOTED

YARD

SUMMARY OF RESULTS OF LIQUEFACTION EVALUATION

WATTS BAR NUCLEAR PLANT
TENNESSEE VALLEY AUTHORITY
DIVISION OF ENGINEERING DESIGN

SUBMITTED | RECOMMENDED | APPROVED

KNOXVILLE

FIGURE 21

RECORD DRAWING AS CONSTRUCTED

INSPECTED AND APPROVED FOR ISSUE

APPENDIX A

SITE-SPECIFIC STRESS REDUCTION COEFFICIENT

A.1 INTRODUCTION

As described in the main body of the report, the procedure used for evaluation of liquefaction potential requires estimates of the shear stresses developed in a soil deposit during the postulated earthquake. The induced shear stresses are used to compare with the cyclic strength of the deposit to assess its liquefaction potential. Seed and Idriss (1970a) described a simplified procedure for estimating the shear stresses developed in a soil deposit during an earthquake. The procedure was developed based on ground response due to the vertical propagation of shear waves in the deposit. Assuming that the soil column above a soil element at depth behaved as a rigid body and the maximum ground surface acceleration were a_{max}, the maximum shear stress on the soil element would be

$$(\tau_{max})_r = \frac{\gamma h}{g} \cdot a_{max} \qquad \text{(A-1)}$$

where γ is the unit weight of the soil, see Figure A.1. Because the soil column behaves as a deformable body, the actual shear stress at depth h, $(\tau_{max})_d$, as determined by a ground response analysis will be less than $(\tau_{max})_r$, and may be expressed by

$$(\tau_{max})_d = r_d \cdot (\tau_{max})_r \qquad \text{(A-2)}$$

where r_d is a stress reduction coefficient with a value less than 1. Seed and Idriss (1970a) developed a range of r_d values for a wide variety of earthquake motions and soil conditions. Thus, the maximum shear stress developed during an earthquake can be estimated from the relationship as expressed by

$$(\tau_{max})_d = \frac{\gamma h}{g} \cdot a_{max} \cdot r_d \qquad \text{(A-3)}$$

The actual shear stress at any point in a soil deposit during an earthquake consists of a time history of stresses having varying peak amplitudes. To compare with the cyclic strength characteristics of the soil deposit, it is necessary to determine the equivalent uniform average shear stress. By appropriate weighting of the individual stress cycles, based on cyclic strength data, values of equivalent uniform cyclic shear stresses for a specific number of cycles can be obtained. Ratios of an equivalent uniform cyclic stress to the maximum peak stress depend on the duration of ground shaking or the magnitude of the earthquake. Based on analyses for a number of different cases, Seed and others (1975) developed a relationship between number of significant cycles and earthquake magnitude for which the average equivalent uniform shear stress, τ_{av} is 65% of the maximum shear stress, τ_{max}. Combining the factor of 0.65 with Equation (A-3), the average cyclic shear stress may be determined by:

$$\tau_{av} = 0.65 \cdot \frac{\gamma h}{g} \cdot a_{max} \cdot r_d \qquad \text{(A-4)}$$

Equation (A-4) can be used to estimate the average cyclic shear stress induced in a soil deposit due to earthquake excitation as described in the main body of the report.

As described above, the variation of r_d with depth is a function of earthquake motions and soil conditions. To take into account the soil conditions and earthquake motion appropriate at the Watts Bar Nuclear Plant site, a series of ground response analyses were made to estimate the stress reduction factor, r_d. The computed variation of r_d with depth was used in the evaluation of liquefaction potential described in Section 3. The following sections describe the ground reponse analyses made and the variation of r_d computed in this study.

A.2 SOIL PROFILE AND DYNAMIC SOIL PROPERTIES

The soil conditions along the ERCW pipeline route 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 thickness of the alluvial soils ranges from 20 to 50 feet. Geophysical measurements were made at a number of locations along the ERCW pipeline route by TVA using the seismic down-hole survey technique. The shear wave velocity of the soil deposit based on the geophysical measurements ranges from 700 to 1800 ft/sec with an average value of about 1200 ft/sec as shown in Figure A.2 (FSAR 2.5.4.2). These shear wave velocities represent average values for the total thickness of the soil deposit. For the alluvial soil deposit at the site, it is reasonable to assume that the shear wave velocity increases gradually with depth due to an increase in overburden pressure. The shear wave velocity or the shear modulus is assumed to vary with a mean effective stress as follows

$$G_{max} = \rho v_s^2 = 1000 \cdot K_{2max} (\sigma_m')^{\frac{1}{2}} \qquad \text{(A-5)}$$

where G_{max} = shear modulus at small shear strain, lbs/sq ft

v_s = shear wave velocity at small shear strain, ft/sec

ρ = mass density, slugs/cu ft

σ_m' = mean effective stress, lbs/sq ft

The mean effective stress, σ_m', may be calculated by

$$\sigma_m' = \frac{(1+2\ K_o)}{3}\ \sigma_v' \qquad \text{(A-6)}$$

where K_o = coefficient of lateral earth pressure at rest
σ_v' = effective overburden pressure, lbs/sq ft

The value of K_o was assumed to be 0.5 for a normally consolidated soil in this study. Using Equations (A-5) and (A-6), the variation of shear wave velocity with depth can be calculated for a given value of K_{2max}. A value of K_{2max} equal to 109 was calculated assuming a value of shear wave velocity of 1200 ft/sec at a depth of 30 feet. Based on the above assumptions, a reasonable variation of shear wave velocity with depth for the soil conditions at the Watts Bar Nuclear Plant site was calculated as shown in Figure A.2. The variation of shear wave velocity with depth shown in Figure A.2 was used in the ground response analysis conducted in this study. The variations of shear modulus and damping ratio with shear strain of the soil were assumed to be the average variations for sands developed by Seed and Idriss (1970b).

A.3 ACCELEROGRAMS USED IN GROUND RESPONSE ANALYSES

Thirty horizontal accelerograms obtained at recording stations having soil conditions comparable to those along the ERCW pipeline route were identified in Ref. 18. These accelerograms were recorded during earthquakes of magnitude M_L 5.8 $\pm$ 0.5 within a distance range of 0 to 25 km. Statistical analyses of peak accelerations for these accelerograms resulted in mean and 84th percentile peak accelerations of 0.10 g and 0.22 g, respectively.

For the ground response analyses, only accelerograms were used which had a peak acceleration close to 0.22 g. Seven accelerograms with a peak acceleration within ± 20 percent of 0.22 g were identified and used ; these accelerograms are identified in Table A-1.

A.4 RESULTS OF GROUND RESPONSE ANALYSES

Ground response analyses were conducted using the computer program SHAKE. The analyses were made using the horizontal accelerograms identified in Table A-1 as input motions. Each of the accelerograms was scaled to a peak acceleration of 0.22 g and prescribed at the ground surface. A frequency cutoff of 25 Hz was used.

The results of the ground response analyses include peak values of shear stresses and strains at the mid-point of each layer in the soil profile. Using the peak values of shear stresses, values of r_d were computed using Equation (A-3). The computed variations of r_d with depth for each of the accelerograms used are summarized in Figure A-3. The results shown in Figure A-3 indicate that the scatter of the r_d values is very small except for the values obtained from the analysis using accelerogram No. F133NORT. The values of r_d computed from the analysis using accelerogram No. F133NORT are much smaller than those computed using the other six accelerograms. An average variation of r_d with depth was computed from the results obtained using the six accelerograms, excluding the results of accelerogram No. F133NORT. The computed average variation is shown in Figure A-3. The average variation of r_d was used in the evaluation of liquefaction potential described in Section 3.2.2.

TABLE A-1
ACCELEROGRAMS USED IN GROUND RESPONSE ANALYSES

Earthquake	Magnitude M_L	Recording Station	Record	Component	Volume 2 Peak Acceleration (g's)
Lytle Creek, CA 9/12/1970	5.4	6074 Park Drive Wrightwood, CA	W334	S25W	0.198
Friuli Sequence 5/11/1976	5.3	Forgaria-Corn. Italy	F059	NORT	0.186
Friuli Sequence 9/11/1976	5.5	Tarcento Italy	F133	NORT	0.205
Friuli Sequence 9/11/1976	5.9	Forgaria-Corn. Italy	F138	EAST	0.228
Friuli Sequence 9/15/1976	6.1	Forgaria-Corn. Italy	F152	NORT	0.256
			F152	EAST	0.212
Livermore Sequence Shock A 1/24/1980	5.9	Del Valle Dam	LA23	S66W	0.254

Note: In the ground response analysis, each of the accelerograms was scaled to a peak accleration of 0.22 g.

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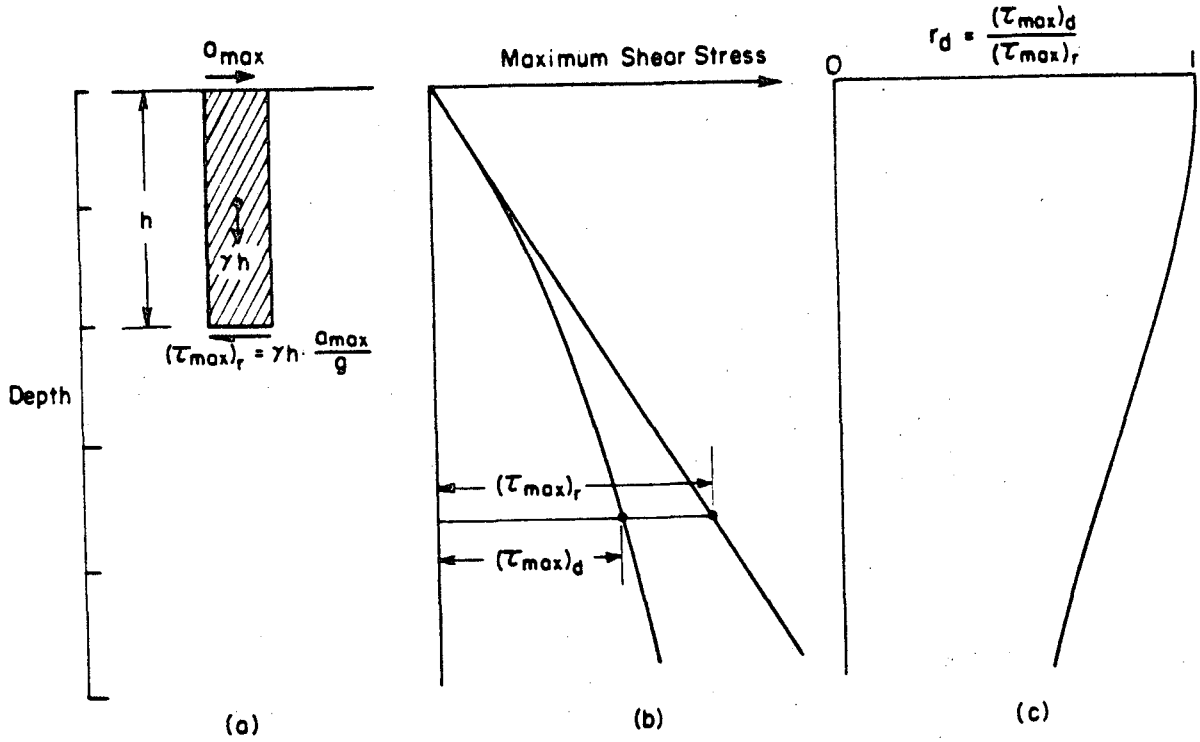

Figure A.1 — Determination of Maximum Shear Stress

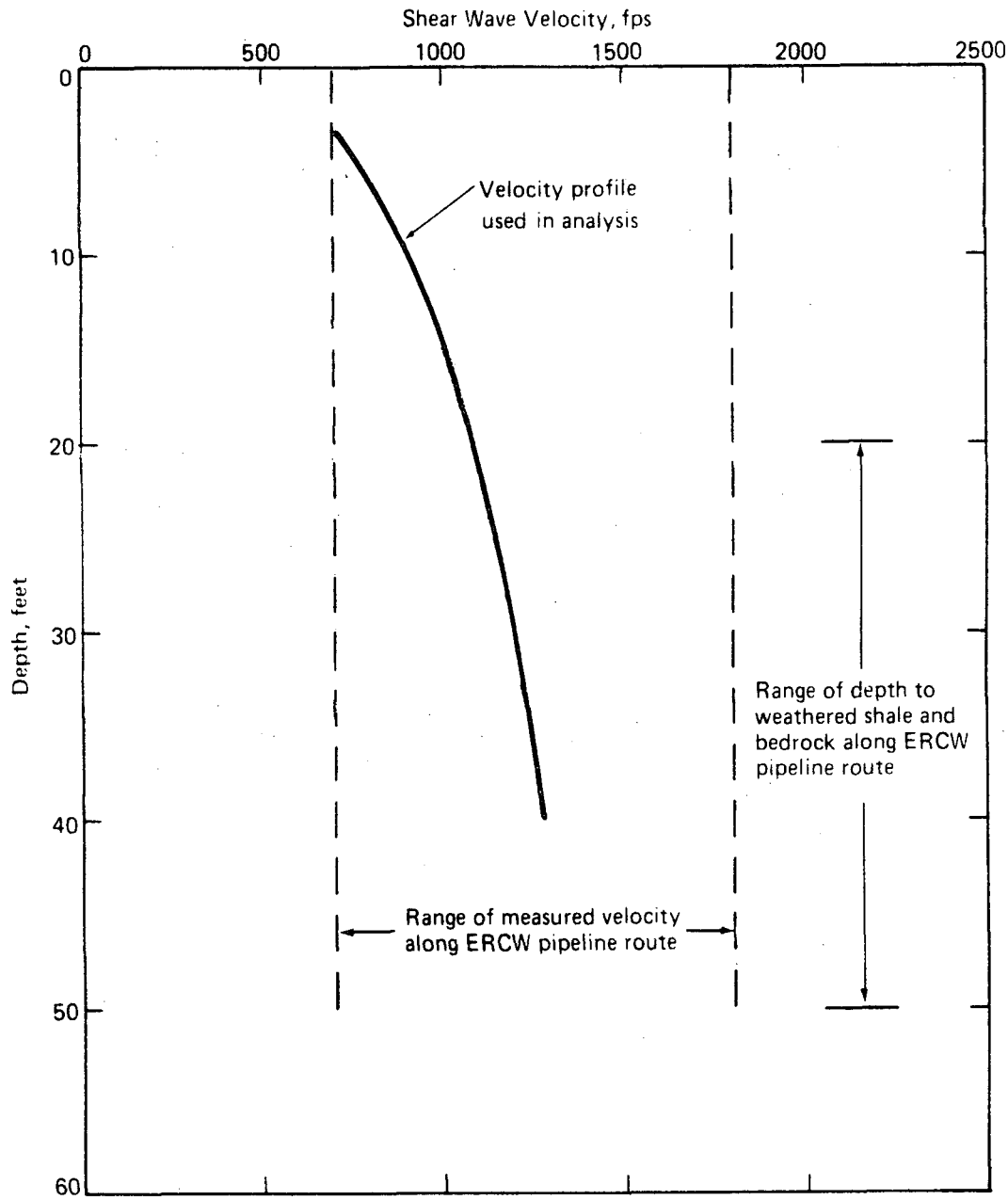

Figure A.2 Shear Wave Velocity Profile Used in Ground Response Analyses

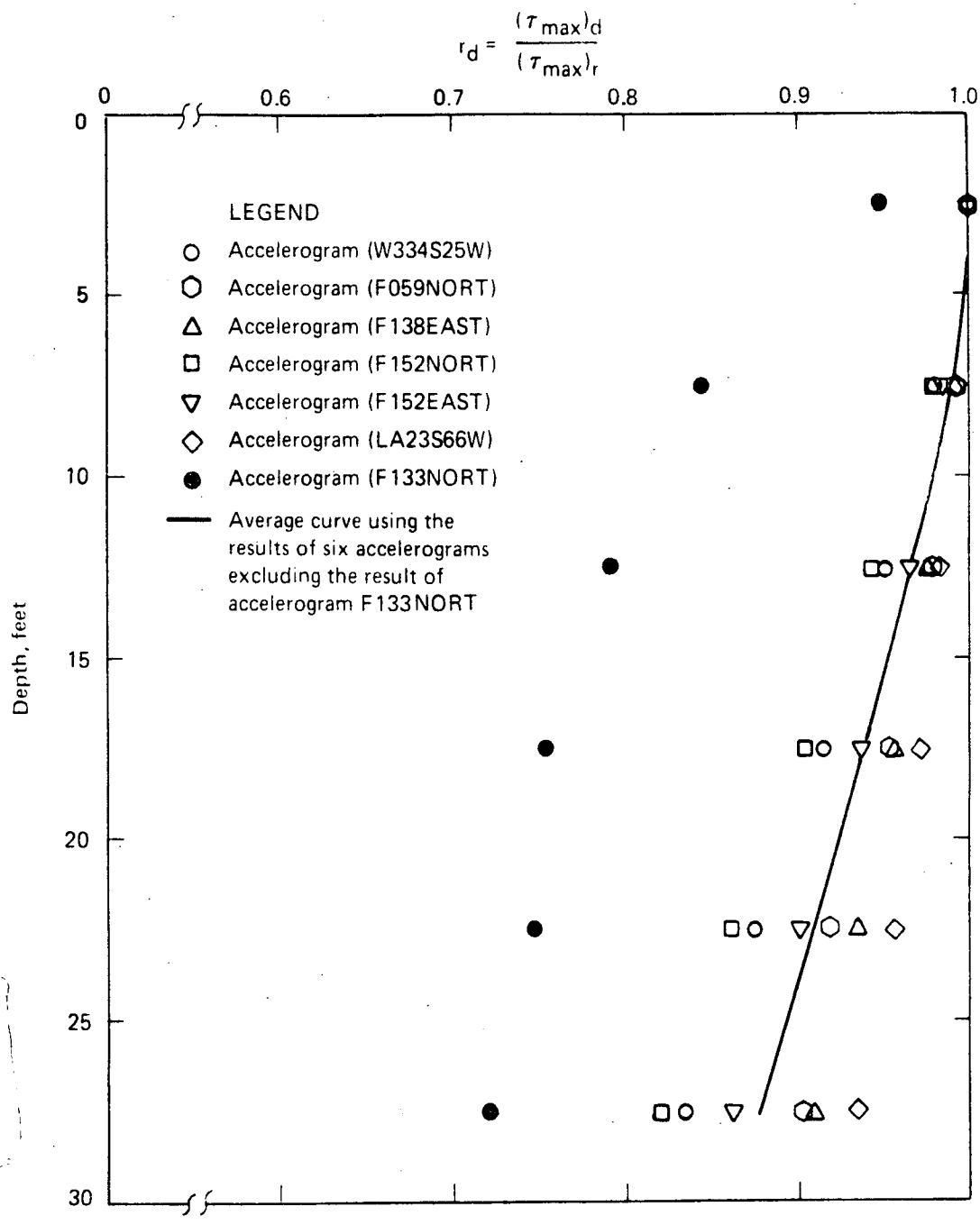

Figure A.3 – Variations of Stress Reduction Coefficient, r_d, with Depth Computed from Ground Response Analysis