RESPONSE TO NRC

LIQUEFACTION QUESTIONS

PALO VERDE NUCLEAR GENERATING STATION

Prepared for:

NUS Corporation
Sherman Oaks, California

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NRC LIQUEFACTION QUESTIONS

323.67

The factor of safety (1.2) for liquefaction potential is too low. Reevaluate the liquefaction potential for the site. If you cannot demonstrate that the factor of safety is 1.5 or higher based on undisturbed sample test data, provide a plan to lower the groundwater level under and around the Category I structures and components, or an alternate plan, to preclude liquefaction risks. A factor of safety of 1.3 is required for remolded sample data evaluations.

323.68

Clearly describe the dynamic input motion. The input motion acceleration time history and the induced shear stress time histories, for the depths presented in Tables 2T-10 and 11, representing the SSE condition, should be presented and discussed as well as cross referenced.

323.70

Provide a step-by-step description of how your evaluation of the liquefaction potential was made, and show where and how you use conservatism in your analysis. Clearly indicate groundwater conditions.

323.71

Demonstrate that analysis based on in situ data only is conservative. Compare the presented analysis with the laboratory test data analysis. Show how this data is used in the analysis.

323.72

Expand your discussion and analysis of "sequence F" liquefaction potential. Provide soil sections and list the borings and test results used to conclude that this interval presents no stability problems.

Response

The response to the above questions is provided in the following text.

INTRODUCTION

Analyses were performed to determine the liquefaction potential of the saturated granular soils underlying the Category I structures at Units 1, 2 and 3 of the Palo Verde site. The analyses consisted of calculating factors of safety against liquefaction by comparing the calculated induced shear stresses for various input acceleration-time histories to the cyclic strengths of the granular soils as determined from laboratory tests on undisturbed samples.

The steps involved in the analyses included:

A. Development of Soil Models

1. Establishing the location of soil strata characteristically considered susceptible to liquefaction,
2. Predicting the highest anticipated water levels for the site, and
3. Determining the appropriate static and cyclic properties of the saturated granular soils underlying Category I structures.

B. Selection of Acceleration-Time Histories as Input Motions

C. Performance of One-dimensional Shear Beam Analyses for Determination of Induced Stresses for each Input Motion.

1. Calculation of average induced stresses and corresponding equivalent loading cycles.

D. Evaluation of Cyclic Strengths and Selection of Repre-

sentative Strength Curves, and

E. Determination of Factor of Safety Against Liquefaction.

The following sections present the details and results of the liquefaction evaluation for the Palo Verde site.

SOIL MODELS -- UNITS 2 AND 3

The subsurface soil conditions underlying the three units are described in Appendix Section 2T.5 of the PSAR. Geologic profiles (PSAR Figures 2T-2, 2T-2a, and 2T-2b) were prepared from the boring logs for each unit to show the stratigraphic sequences in relation to the Category I structures. The down-hole geophysical logs were used in addition to undisturbed sampling to accurately determine the thicknesses of individual sequences. The upper granular soil sequences which occur below the maximum anticipated water levels at the three units are designated as B, C and D (30 to 50 feet below present grade). Sequence F (70 to 80 feet below grade) is primarily a silt with isolated discontinuous lenses and pockets of silty sand. Other granular sequences at the site occur below a depth of 150 feet and contain appreciable amounts of fines.

The perched water mound which exists at the site due to irrigation (Figure 2.4-29c, PSAR Section 2.4.13) causes water levels to occur at different depths beneath the three units. These water levels occur at elevations 888, 913, and 920 beneath Units 1, 2 and 3, respectively; corresponding to depths below existing grade (as well as approximate finished grade) of 65, 42, and 31 feet, respectively. The water levels were established from monitor wells located at the unit locations and throughout the site property (PSAR Section 2.4.13). Since the site property boundaries encompass a major portion of the perched mound and no further irrigation is proposed, the present

water levels are the highest which are expected in the unit areas during the life of the facility. It should be noted that the theoretical analyses (PSAR Section 2.4.13.1.2.1) indicate that the evaporation pond and storage reservoir may produce maximum water level rises of 5.6, 1.2, 1.8 feet at Units 1, 2, and 3, respectively. However, these rises would have a negligible effect on the calculated factors of safety.

On the basis of subsurface information described above, soil models were developed for the liquefaction evaluation. The models were prepared only for Units 2 and 3 since it is at these locations where water is high enough (even with the possible rise at Unit 1 noted above) to saturate the granular soils occurring at depths of approximately 30 to 50 feet. For the evaluation of the deeper soil sequence F (70 to 80 feet), Units 2 and 3 with the higher water levels would clearly have a lower factor of safety than Unit 1. Thus, the Unit 1 soils were not modelled.

The soil models developed for Units 2 and 3 are shown on Tables 2T-8 and 2T-8a. The models were not extended beyond 100 feet because the response analyses were performed by inputing acceleration records at the ground surface and deconvoluting downward. In this case, the depths of the soil models must only include the deepest soil layer to be evaluated.

The basis for the division of the soil model into layers included changes in soil type, location of water levels, and the general thickness requirements for the analytical solution used. These latter requirements indicate soil layers should be divided into thicknesses approximately equal to $\Delta t \times V_s$, where Δt is defined

as the time interval between digitized acceleration values in the input records and V_s is the shear velocity of the soil layer being divided.

The total unit weights shown on the soil models represent average values obtained from Pitcher tube samples for each of the two units. The maximum shear moduli (G_{max}) values shown were obtained from two sources: 1) laboratory values determined by resonant column testing on undisturbed samples (PSAR Section 2.5.4.2), and 2) in-situ values determined from the crosshole surveys performed at each unit (PSAR Section 2.5.4.4 and Appendix Section 2U). The damping values listed for the soil layers were derived from the resonant column laboratory test results on undisturbed samples.

Curves representing the variation of shear moduli and damping with shear strain were determined for each unit from laboratory resonant column and cyclic triaxial testing. The normalized curves used in the liquefaction analyses for the undisturbed soils are shown in Figures 2T-9 through 2T-9c.

The final selection of the maximum shear moduli (laboratory or in-situ) to be used in the liquefaction analyses was determined from a one-dimensional shear beam analysis (as described in the subsection <u>Response of Soil to Dynamic Loading</u>). Separately, the laboratory and in-situ G_{max} values were used with the normalized moduli versus strain curves (Figures 2T-9 and 2T-9a) and with the soil models to determine which produced the more conservative (higher) induced stresses. As can be seen in Table 2T-9, the in-situ values produced the higher stresses and were therefore used in subsequent analyses.

EARTHQUAKE RECORDS

A total of five acceleration-time history records were selected as input motions for the liquefaction evaluation. Four of the records are actual recordings of the 1952 Kern County earthquake as scaled to represent the postulated Maximum Earthquake (magnitude 8 at 72 miles) for the Palo Verde site (PSAR Section 2.5.2.9). These records include Santa Barbara, the S48^{o}E component; Pasadena, S90^{o}W; Hollywood Storage (basement recording), S0^{o}W; and Hollywood P. E. lot, S0^{o}W. Each record is the more severe horizontal component recorded at the respective locations and all the records are a conservative representation of the Maximum Earthquake (documentation is provided in PSAR Section 2.5.2.10).

The fifth record selected for the liquefaction evaluation was an artificial record with a spectra which closely conforms to the shape of the standard NRC spectra as specified in Regulatory Guide 1.60. The record used was developed by the Bechtel Corporation with documentation provided in their topical report BC-TOP-4A to the NRC. For the evaluation, the record was scaled to 0.2g as a conservative representation of SSE conditions for the Palo Verde site. The conservatism can be seen in PSAR Figures 2.5-67 through 2.5-69 which show the relative spectral levels between the real records and the standard NRC shape. The relative difference in the spectral levels represents the difference in energy produced by the records which, for over most of the period range of interest,

is two to three times higher for the NRC spectral level than for the real earthquake levels.

The pertinent characteristics of the records as used for the liquefaction evaluation are summarized in Table 2T-9a. The Bechtel record, although having the least total duration of all the records, is considered to have sufficient duration of strong shaking intensity to generate stresses as high or higher than another record of same intensity but of longer duration. Bolt (1973) has shown that recorded earthquakes have produced only on the order of six seconds of strong shaking as defined by acceleration levels of 0.05g or greater for a magnitude 8 earthquake 62 miles from the recording sites. As may be noticed from Table 2T-9a, all the selected records meet these duration requirements.

RESPONSE OF SOIL TO DYNAMIC LOADING

The dynamic response of the Units 2 and 3 soils under the influence of the five earthquake records described in the preceeding section was evaluated by one dimensional shear beam analyses. The analyses were performed by the use of the computer program SHAKE (Schnabel, Lysmer, and Seed, 1972). A one dimensional analysis is considered reasonably accurate for essentially horizontally layered soils such as exist at the Palo Verde site. Because the Bechtel record was simulated for the free field condition at the ground surface, and selected real records were recorded at or near the ground surface, the design motions were applied at the ground surface of the soil models. These motions were then deconvoluted from surface to the full depth of the models.

The dynamic responses calculated for each record consisted of shear stresses, shear strains and peak acceleration levels within each layer of the soil models. The results of variation of peak acceleration versus depth is shown in Figures 2T-9d and 2T-9e for Units 2 and 3, respectively. The variation of maximum shear stresses with depth for the two units is shown in Figures 2T-9f and 2T-9g.

CYCLIC STRENGTH DETERMINATION

The dynamic strengths of the granular soils underlying the Category I structures were determined from cyclic triaxial compression tests on undisturbed samples. Samples were initially obtained by Pitcher barrel sampling equipment from the rotary-wash borings drilled for the Category I structures. In evaluating the cyclic strengths of the Pitcher samples, it was felt that sample disturbance was causing low strengths which were not compatible with the high Standard Penetration Test results (N values) obtained during drilling (PSAR Figures 2T-8, 8a and 8b).

Another field program was initiated at Units 2 and 3 to obtain undisturbed block samples from large diameter borings from both above and below the existing perched water levels. The boring at Unit 2 was designated U2-LB-1 and the two borings at Unit 3 were designated U3-LB-1 and U3-LB-2. The three borings were located within the respective containment areas.

The purpose of the new program was to compare cyclic strengths obtained from the two different sampling techniques. In general, block samples were obtained by auger drilling (6-foot diameter) to the sampling depth, cleaning out the boring and then drilling a smaller diameter (2-foot) pilot hole. Each undisturbed sample was carved by hand into a 9-1/2 inch diameter by 12-inch high polyethelene mold. Some samples were obtained directly from the side of the large diameter boring. Most samples were taken from the floor of the boring. Samples were then

carefully transported by private vehicle to the laboratory.

Both Pitcher and block samples were tested in accordance with the cyclic triaxial procedures outlined in PSAR Section 2T-4.15.3. The procedure for trimming of test-size samples (2.5-inch diameter by 6-inch) from the large block samples in some cases involved freezing of the block with liquid nitrogen prior to the final hand carving. Other samples were hand carved directly from the blocks without freezing. The procedures used for preparation of block samples for testing are outlined in the attached Enclosure 1. The results of the cyclic triaxial tests, in terms of pore pressure and strain versus number of cycles, are presented in the PSAR Figures 2T.21.93 through 2T.21.140 and in Figures 2T.21.141 through 2T.21.175 in the attached Enclosure 2. Grain size distribution curves for the block samples are included in Enclosure 3.

For the liquefaction analyses, the test results for the granular soil sequences B, C and D between depths of 30 and 50 feet were evaluated in detail. Although sequence F is primarily a silt, the test results for the granular lenses between depths of 70 and 80 feet were also evaluated. A summary of the static properties and cyclic strengths from both the Pitcher and block samples obtained from the two depth intervals is presented in Table 2T-9b. The criterion used for selection of the cyclic strengths was the number of cycles at which the pore pressure reached the test cell pressure or five percent double amplitude axial strain, whichever occurred first.

The following subsections present a discussion of the strength evaluation of the granular soils from the two depth intervals 30 to 50 feet and 70 to 80 feet. The effects of sampling technique on cyclic strength is illustrated and final strength curves are presented for use in the liquefaction analyses.

Cyclic Strength Test Results - 30 to 50 feet depth interval

A summary plot of the laboratory cyclic strengths of the Pitcher samples obtained between depths of 30 to 50 feet is presented on Figure 2T-10. The average soil characteristics (dry density, D_{50} and percent of fines) from the Pitcher samples are summarized in Table 2T-9c.

A similar summary plot for the block sample cyclic strengths is shown on Figure 2T-10a. Average soil characteristics are also summarized in Table 2T-9c. The two distinct levels of strength shown on Figure 2T-10a were attributed to a combination of density and percent fines contents. The upper strength curves drawn through Test Nos. 35, 36, 39 and 40 all exhibited higher densities (greater than 103 pounds per cubic foot) and contained relatively high percentages of fine-grained material. For the lower strength curve, all but one test sample (Test No. 44) exhibited dry densities lower than 101 pounds per cubic foot. The lower strength exhibited by Test No. 44 was attributed to the low fines content (6 percent).

The position of the average curves drawn through the data was based on typical shapes of strength curves developed from shaking table tests (De Alba, 1975).

As can be seen from Table 2T-9c, the samples obtained by the two sampling techniques compare very well in terms of D_{50} and the percent passing the No. 200 Standard ASTM sieve. No differences in material type are indicated. However, in comparing the cyclic strengths, Figure 2T-10b, significant strength differences resulted from tests performed on Pitcher samples as compared with the block samples. The difference between the lower average block sample strength and the average Pitcher sample strength is on the order of 32 percent at 30 cycles. The differences are attributed to the Pitcher barrel sampling technique which probably caused disturbance to the soil structure or caused the breakdown of a slight cementation which may exist between in-situ soil particles.

For the liquefaction analyses, the lower average cycle strength curve of the block samples was selected as being a conservative representation of the in-situ granular materials encountered between depths of 30 to 50 feet at the Palo Verde site. Despite the overconsolidated nature of the sands, a reduction of the laboratory curve by a factor (c_r) of 0.58 (De Alba, 1975) results in a conservative "field adjusted" strength curve which accounts for possible strength over-estimates caused by the cyclic triaxial apparatus. Design laboratory and field dynamic curves are presented in Figure 2T-10c.

Cyclic Strength Test Results - 70 to 80 foot depth interval

Occasionally thin layers and lenses of granular soils were encountered between depths of 70 and 80 feet, but these materials were discontinuous between borings and contained

high percentages of clay and silt (stratigraphic sequence F; refer to Appendix Figures 2T-2, 2T-2a and 2T-2b). As a result, soil sequence F is not considered susceptible to liquefaction of an extent which would cause any effect on Category I structures. Documentation of the high fines content and discontinuous nature of the granular soils, as well as an evaluation of the cyclic strength of the isolated lenses of granular soils within sequence F, is presented in the following paragraphs.

The description of soil sequence F is summarized as "sandy silt, clayey silt, and clayey sand (ML-SC) brown, locally micaceous, occasional silty sand horizons." The soils comprising this member are primarily silts in contrast to the clay layers which lie above and below. Silty clays were the second most encountered soil type within F, and the silty sand lenses having the least frequency of occurrence.

Pitcher, drive and Standard Penetration tests were used to sample soil sequence F. Because most of the sampling was not continuous, conservative determinations of thickness were interpreted from the geophysical borehole logs which produced different signature responses for F and the sequences above and below. Table 2T-10 summarizes the specific member location and the results of soil classification testing.

Table 2T-10 clearly indicates that very few granular materials were encountered and there is no significant continuity

between borings among those granular soils which were encountered. In addition, detailed analysis of the downhole geophysical logs indicates soil member F has a relatively consistent thickness across the site, but any interbedded granular materials are lenticular and are not continuous between borings.

To emphasize the lack of a liquefaction potential within soil sequence F, an evaluation was made of the cyclic strengths of the granular soil portions. The only cyclic strength results available for sequence F were for samples obtained by the Pitcher barrel sample technique. These results are shown on Figure 2T-10d and a summary of pertinent test data is listed in Table 2T-9b.

Because sampling technique was shown to have an effect on the cyclic strength of the granular soils between depths of 30 and 50 feet, an evaluation was performed to account for the effect on the cyclic test results from sequence F. This evaluation compares Pitcher sample and block sample strengths of the granular soils obtained between 30 and 50 feet which have approximately the same soils characteristics (density, D_{50}, and % - #200 sieve) of the Pitcher samples from sequence F. The range of soil properties for those Pitcher samples obtained for cyclic testing are:

Sampling Method	Depth (feet)	% fines (#200 sieve)	D_{50} (mm)	Dry Density, (pcf)
Pitcher	70-80	24-59	0.07-0.12	93-105

In order to compare block samples and Pitcher sample strengths at a depth of 30 to 50 feet, only those block samples obtained from below the perched water level and containing greater than 17 percent fine-grained soil were considered. The results obtained from block samples below the water were found to be significantly lower in strength than that of samples obtained above water. Hence, a conservative comparison could be made.

The following summarizes the soil characteristics of the Pitcher and block samples used for the comparison:

Sampling Method	Depth (feet)	% fines (#200 sieve)	D_{50} (mm)	Dry Density, (pcf)
Block	30-50	17-23	0.19-0.40	98-105
Pitcher	30-50	18-42	0.11-0.25	98-105

The cyclic strengths of the two groups of samples obtained between 30 and 50 feet are shown on Figure 2T-10e. As can be seen, there is a definite increase in the strengths exhibited by the undisturbed block samples relative to the Pitcher samples. Constructing average curves through the two groups of data shows that at 30 cycles a strength increase of 63 percent is obtained. The higher strengths obtained from the block samples are more compatible with the high N values (PSAR Figures 2T-8, 2T-8a and 2T-8b) obtained during the first drilling program.

The average Pitcher sample strengths (Figure 2T-10d) were adjusted by the differences shown in Figure 2T-10e. The resulting laboratory cyclic strength curve, considered representative of granular lenses within sequence F, is shown on Figure 2T-10f. The field curve shown incorporates a reduction to 58 percent (De Alba, 1975) of the laboratory curve to account for the possible effects of cyclic triaxial testing.

INDUCED SHEAR STRESS AND EQUIVALENT CYCLE EVALUATION

In addition to the peak acceleration and maximum shear stress data obtained from the soil column analyses described in the section Response of Soil to Dynamic Loading, shear stress-time histories were obtained for selected soil layers. These stress histories were used to determine the number of equivalent cycles of an average stress intensity that each input record induced into the site soils. The shear stress histories obtained for Layer Nos. 9 and 15 within the Unit 2 and 3 soil models are shown on Figures 2T-10g and 10h for the Bechtel input record.

The method used to determine the equivalent number of load cycles for each of the five input time histories is described by Lee and Chan (1972). Details of the steps performed for the analyses include:

1. A value of 65 percent of the maximum induced shear stress (τ_{max}) produced by each stress history was selected as a reasonable average cyclic intensity (τ_{ave}).
2. Stress values of each stress history were normalized in terms of τ_{max} ($\tau_{max} = 1$) and each $\pm$ 0.25 τ_{max} was marked off on the histories.
3. The number of stress peaks falling within each zone (e.g., $\pm$ 0.5 to 0.75) were counted and averaged to give a value P_i. The normalized value of stress at the center of each zone was noted and designated τ_i.

4. The field cyclic strength curves (Figure 2T-10c for soils between 30 to 50 feet and Figure 2T-10f between 70 to 80 feet) were extrapolated to 1 cycle ($\frac{\sigma d}{2\sigma_o'}{}_1$). Curve shapes similar to those developed by De Alba (1975) for samples tested by a shaking table were used for this purpose.
5. The extrapolated field curves were adjusted to a position parallel to themselves and intersecting at 65 percent of the stress ratio value ($\frac{\sigma d}{2\sigma_o'}{}_1$) at one cycle. Then the stress ratios were normalized by 0.65 ($\frac{\sigma d}{2\sigma_o'}{}_1$).
6. Using the normalized strength curves for the appropriate soil layers, the number of cycles, R_i, for each value of τ_i were selected.
7. The number of cycles from the normalized strength curves corresponding to $0.65\,\tau_{max}$ were established as the reference cycle, R_f, for each record.
8. The number of equivalent cycles for each divided zone was calculated from $N_{ei} = \frac{R_f \times P_i}{R_i}$
9. The total number of cycles is the sum ΣN_{ei} for all the zones.

Table 2T-11 summarizes the results of the equivalent cycles counted for the five input records for the Unit 3 soil model.

The number of cycles for similar depths within Unit 2 was not expected to differ markedly from the ratios obtained for Unit 3. This was checked by evaluating the stress histories from

INDUCED SHEAR STRESS AND EQUIVALENT CYCLE EVALUATION

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The method used to determine the equivalent number of load cycles for each of the five input time histories is described by Lee and Chan (1972). Details of the steps performed for the analyses include:

1. A value of 65 percent of the maximum induced shear stress (τ_{max}) produced by each stress history was selected as a reasonable average cyclic intensity (τ_{ave}).
2. Stress values of each stress history were normalized in terms of τ_{max} ($\tau_{max} = 1$) and each $\pm$ 0.25 τ_{max} was marked off on the histories.
3. The number of stress peaks falling within each zone (e.g., $\pm$ 0.5 to 0.75) were counted and averaged to give a value P_i. The normalized value of stress at the center of each zone was noted and designated τ_i.

4. The field cyclic strength curves (Figure 2T-10c for soils between 30 to 50 feet and Figure 2T-10f between 70 to 80 feet) were extrapolated to 1 cycle $\left(\frac{\sigma d}{2\sigma_o'}\right)_1$. Curve shapes similar to those developed by De Alba (1975) for samples tested by a shaking table were used for this purpose.
5. The extrapolated field curves were adjusted to a position parallel to themselves and intersecting at 65 percent of the stress ratio value $\left(\frac{\sigma d}{2\sigma_o'}\right)_1$ at one cycle. Then the stress ratios were normalized by 0.65 $\left(\frac{\sigma d}{2\sigma_o'}\right)_1$.
6. Using the normalized strength curves for the appropriate soil layers, the number of cycles, R_i, for each value of τ_i were selected.
7. The number of cycles from the normalized strength curves corresponding to 0.65 τ_{max} were established as the reference cycle, R_f, for each record.
8. The number of equivalent cycles for each divided zone was calculated from $N_{ei} = \frac{R_f \times P_i}{R_i}$
9. The total number of cycles is the sum ΣN_{ei} for all the zones.

Table 2T-11 summarizes the results of the equivalent cycles counted for the five input records for the Unit 3 soil model.

The number of cycles for similar depths within Unit 2 was not expected to differ markedly from the ratios obtained for Unit 3. This was checked by evaluating the stress histories from

Layer Nos. 9 and 15, Unit 2, using the Bechtel record. Values of 16 equivalent cycles were obtained for both layers and thus the values for Unit 3 could be extrapolated to Unit 2. Table 2T-11a summarizes the average stresses and equivalent cycles for the critical soil layers.

FACTORS OF SAFETY

The factors of safety against liquefaction for the saturated granular soils underlying Units 2 and 3 were calculated by comparing the field cyclic strength at the calculated equivalent number of stress cycles to the corresponding average induced stress (τ_{ave}). The results for the various soil layers and records analyzed are shown on Tables 2T-12 and 12a.

The Bechtel record scaled to the level of the 0.2g standard NRC spectra (SSE condition) produces the lowest factors of safety (1.4 for Unit 2 and 1.1 for Unit 3) at depths of approximately 40 to 50 feet. The minimum factor of safety for the 70 to 80-foot interval was 1.5 at Unit 3. The real records produced factors of safety ranging from 1.7 to 2.8 for the 30 to 50-foot interval and 2.1 to 3.3 for the 70 to 80-foot interval.

The conservatism of the evaluation of the SSE conditions with the Bechtel record is exemplified by the significantly higher factors of safety produced by real records representative of the Maximum Earthquake for the Palo Verde site. Consequently, based on the above factors of safety, a liquefaction potential is not considered to exist at the Palo Verde site.

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Table 2T-8

SOIL MODEL FOR LIQUEFACTION POTENTIAL ANALYSES - UNIT 2

Depth (ft)	Layer No.	Soil Type[1]	Total Unit Weight (pcf)	In Situ Shear Modulus x 10^6 (psf)	Laboratory Shear Modulus x 10^6 (psf)	Laboratory Damping percent of Critical	Soil Sequence[2]
5	1	2	119	3.70	2.00	4.0	A
10	2	2	119	3.70	2.00	4.7	
15	3	2	119	3.70	2.10	5.4	
20	4	2	119	3.70	2.15	5.9	B
25	5	2	119	3.70	2.20	6.5	
30	6	2	119	3.70	2.25	6.9	
36	7	2	119	4.63	2.34	7.2	C
42	8	2	119	4.63	2.51	7.5	
46	9	2	125	4.63	2.51	7.6	D
51	10	2	118	4.63	2.65	7.9	
56	11	1	118	4.98	2.70	4.4	
61	12	1	118	4.98	2.80	4.4	E
66	13	1	118	4.98	2.85	4.5	
71	14	1	118	4.98	3.00	4.7	
76	15	2	121	4.98	3.15	9.5	F
81	16	1	121	4.98	3.70	4.9	
86	17	1	123	5.55	3.35	4.9	
91	18	1	123	5.55	3.60	4.9	
96	19	1	123	5.55	3.75	5.0	G
101	20	1	123	5.55	3.80	5.1	
	21	Base	125	6.57	6.57	-	

Water Table at 42 feet

[1]Soil types: 1 - clays and silty clays; 2 - sands and silty sands
[2]Refer to PSAR Figures 2.5-45a and 2T-2a

Table 2T-8a

SOIL MODEL FOR LIQUEFACTION POTENTIAL ANALYSES - UNIT 3

Depth (ft)	Layer No.	Soil Type[1]	Total Unit Weight (pcf)	In Situ Shear Modulus $\times 10^6$ (psf)	Laboratory Shear Modulus $\times 10^6$ (psf)	Laboratory Damping Percent of Critical	Soil Sequence[2]
5	1	2	115	3.76	1.75	4.7	A
10	2	2	115	3.76	1.75	5.2	
15	3	2	121	3.76	1.90	5.5	
20	4	2	121	3.76	2.00	5.6	B
25	5	2	128	6.68	2.24	5.8	
31	6	2	131	6.68	2.41	6.0	
35	7	2	129	6.68	2.50	6.2	C
40	8	2	128	6.68	2.61	6.4	D
45	9	2	128	6.68	2.74	6.6	
50	10	1	123	6.68	2.76	6.8	
55	11	1	123	6.68	2.96	4.6	
60	12	1	123	6.68	3.03	4.7	E
65	13	1	123	5.98	3.17	4.8	
70	14	1	123	5.98	3.31	7.2	
75	15	2	123	5.98	2.89	7.4	F
80	16	1	124	5.98	2.47	4.9	
85	17	1	124	6.03	2.66	4.9	
90	18	1	124	6.03	2.99	4.9	
95	19	1	124	6.03	3.41	4.9	G
100	20	1	124	6.03	3.85	4.9	
	21	Base	125	6.57	4.28	-	

Water Table at 31 feet

[1]Soil types: 1 - clays and silty clays; 2 - sands and silty sands

[2]Refer to PSAR Figures 2.5-45b and 2T-2b

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Table 2T-9

MAXIMUM INDUCED SHEAR STRESSES FOR LABORATORY AND IN-SITU MODULI[1]

Unit No.	Depth below Existing Ground Surface (ft)	Maximum Induced Stresses (psf)	
		In Situ Moduli	Laboratory Moduli
2	42 - 46	900	774
	46 - 51	945	780
3	31 - 35	777	702
	35 - 40	879	784
	40 - 45	986	865

[1]Bechtel record input at ground surface, scaled to 0.2g.

Table 2T-9a

CHARACTERISTICS OF EARTHQUAKE RECORDS
USED FOR LIQUEFACTION ANALYSES

Earthquake Record	Scaled Maximum Acceleration Level (a/g)	Duration, Sec.		Digitized Time Interval
		Strong Motion[1]	Record as used	
Santa Barbara (S48E)	0.128	9	50	0.02
Hollywood P.E. Lot (S00W)	0.10	20	50	0.02
Hollywood Basement (S00W)	0.093	15	50	0.02
Pasadena (S90W)	0.090	17	50	0.02
Bechtel	0.200	16	24	0.01

[1]As defined by the time interval in which the records first and last exceed 0.05g.

Table 2T-9b

SUMMARY OF TEST RESULTS
Cyclic Triaxial Samples

Test No.	Boring No.	Sample No.	Depth Interval (ft.)	Dry Wt. γ_d psf	D_{50} (mm)	Passing # 200 Sieve (%)	$\frac{\sigma dp}{2\sigma_o'}$	$\frac{\sigma_o'}{(psi)}$	Cycles to Failure, N	Sample Type[1]
1	U3-B3	9	32.5-35	101.1	0.17	21	0.25	28.0	10	P
2	U1-B3	18	42.5-45	111.1	0.60	7	0.20	41.0	380	P
3	U2-B2	20	47.5-50	107.6	1.3	15	0.15	30.0	46	P
4	U2-B8	12	43.5-46	98.1	0.2	13	0.31	38.0	6.5	P
5	U2-B31	8	40-42.5	99.5	0.3	13	0.21	38.0	7	P
6	U2-B3	15	38.5-41	98.7	0.18	13	0.19	38.0	18.5	P
7	U2-B27	8	40-42.5	103.3	0.42	15	0.21	38.0	7	P
8	U2-B17	8	43.5-46	108.9	0.32	11	0.20	38.0	20	P
9	U2-B8	10	39.5-42	109.1	0.15	38	0.15	38.0	310	P
10	U2-B29	7	37.5-40	104.1	0.42	12	0.22	38.0	18	P
11	U2-B22	8	40-4 .5	110.0	0.63	15	0.20	38.0	12	P
12	U1-B8	20	39-41.5	98.4	0.11	42	0.20	35.0	260	P
13	U1-B4	9	39.5-41	105.1	0.54	14	0.13	35.0	440	P
14	U1-B7	11	43.5-46	104.5	0.45	11	0.23	35.0	18	P
15	U1-B16	2	35-37.5	105.7	0.78	8	0.22	35.0	14	P

Table 2T-9b (continued)

SUMMARY OF TEST RESULTS
Cyclic Triaxial Samples

Test No.	Boring No.	Sample No.	Depth Interval (ft.)	Dry Wt. γ_d psf	D_{50} (mm)	Passing # 200 Sieve (%)	$\frac{\sigma_{dp}}{2\sigma_o'}$	σ_o' (psi)	Cycles to Failure, N	Sample Type[1]
16	U1-B9	18	37-39.5	103.3	0.40	18	0.29	35.0	13.5	P
17	U3-B4	9	36-38.5	100.3	0.25	18	0.20	31.0	16	P
18	U2-SP20	8	40-42.5	102.6	0.22	20	0.26	35.0	27	P
19	U1-B2	9	36-38.5	101.7	1.1	13	0.16	35.0	18	P
20	U2-B2	11	38.5-41	105.4	0.20	27	0.25	33.0	6	P
21	U1-B5	8	36-38.5	99.6	0.25	18	0.29	35.0	4.5	P
22	U1-B18	6	35-37.5	103.3	0.50	13	0.18	35.0	16	P
23	U3-B18	6	37.5-40	102.1	0.40	11	0.17	32.0	100	P
24	U3-B16	5	36-38.5	104.3	0.40	15	0.16	31.0	200	P
25	U1-B9	32	71-73.5	99.0	0.12	29	0.28	48.0	10	P
26	U1-B6	22	72.5-75	100.6	0.12	32	0.31	50.0	11.5	P
27	U2-B6	23	74-76.5	104.5	0.12	37	0.23	52.0	22	P
28	U1-B16	6	75-77.5	100.5	0.12	24	0.20	50.0	400	P
29	U1-B7	23	75-77.5	100.9	.07	51	0.22	50.0	1000	P
30	U1-B2	23	72.5-75	89.5	0.13	28	0.16	50.0	300	P

Table 2T-9b (continued)

SUMMARY OF TEST RESULTS
Cyclic Triaxial Samples

Test No.	Boring No.	Sample No.	Depth Interval (ft.)	Dry Wt. γ_d psf	D_{50} (mm)	Passing # 200 Sieve (%)	$\frac{\sigma dp}{2\sigma_o'}$	$\frac{\sigma_o'}{(psi)}$	Cycles to Failure, N	Sample Type[1]
31	U1-B4	21	70-72.5	102.4	0.10	45	0.38	48.0	250	P
32	U3-B3	30	71-74	101.5	-	-	0.21	54.0	20	P
33	U3-B3	30	71-74	93.0	-	-	0.25	54.0	4.7	P
34	U2-B18	19	75-77.5	103.4	-	59	0.23	52.0	20	P
35	U3-LB-1	10A	30.7-31.8	107.1	0.32	12	0.43	26.0	20	B_t
36	U3-LB-1	10B	30.7-31.8	105.0	0.36	11	0.48	26.0	11	B_t
37	U3-LB-2	12A	34.4-35.0	98.0	0.19	17	0.40	28.0	5.8	B_f
38	U3-LB-2	13	34.9-35.8	97.9	0.29	8	0.25	30.0	24	B_f
39	U3-LB-2	15A	35.9-36.7	103.3	0.20	23	0.40	30.0	54	B_f
40	U3-LB-2	15B	35.9-36.7	104.6	0.20	23	0.43	30.0	15	B_f
41	U2-LB-1	18A	43-43.8	100.1	0.23	20	0.30	37.0	19	B_t
42	U2-LB-1	18B	43-43.8	98.2	0.27	17	0.35	37.0	7	B_t
43	U3-LB-2	19C	35.9-36.7	97.9	0.48	5	0.30	31.0	2.5	B_f
44	U2-LB-1	20	43.7-44.5	103.2	0.33	6	0.25	38.0	11	B_f
45	U2-LB-1	24	44.7-45.6	96.9	0.76	12	0.25	39.0	13	B_f

[1] P indicates Pitcher Tube Sample
B_t indicates Block Samples hand trimmed from the boring (9½" x 12" and hand trimmed into test size samples (2.5" x 6")
B_f indicates Block Samples hand trimmed from the boring and frozen before hand trimming into test size samples

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Table 2T-9c

AVERAGE SOIL CHARACTERISTICS
CYCLIC TRIAXIAL SAMPLES - 30 TO 50 FEET

Sample Type	Dry Density (pcf)		D_{50}		% Passing # 200 Sieve	
	Range	Ave	Range	Ave	Range	Ave
Block	96.9 - 107.1	101.1	0.19 - 0.76	0.33	5 - 33	14
Pitcher Tube	98.1 - 111.1	103.6	0.11 - 1.3	0.41	7 - 38	17.4

Table 2T-10

SUMMARY OF STRATIGRAPHIC SEQUENCE F, UNIT 1

Unit Boring	N Value	Depth Below Ground Surface in Feet	Unified Soil Classification	Percent Passing #200 Sieve
1	82	68.5-75.5	CH-SC-SM-ML	-
2	-	70-75	ML-SM*	28
3	+100	70-75	SM-ML*	46
4	-	70-76.5	SM*	18 46
5	-	72-78	SC-CL*	82 47
6	-	76-82.5	SC*	32
7	-	74-79	ML	28 51
8	+100	70-75	CL*	77 60
9	-	67-73.5	SM-ML	-
10	-	68-72	CL*	74
11	-	77-86	CL*	86
12	-	68.5-75	CL	-
13	-	73-78	CL*	88
14	-	68-75	SM*	-
15	37	57-62	CL*	51 73
16	-	72-78.5	SM*	36
17	-	71-76	CL*	62
18	-	69-73	ML	-

*In accordance with ASTM-D-2487-69. Other classifications in accordance with ASTM-D-2488-69.

Table 2T-10 (cont.)

SUMMARY OF STRATIGRAPHIC SEQUENCE F, UNIT 2

Unit Boring	N Value	Depth Below Ground Surface in Feet	Unified Soil Classification	Percent Passing #200 Sieve
1	-	80.5-85.5	CL*	97
2	-	75-80	CL*	89
3	-	73-78	CL*	85
4	-	73-80	CL*	80
5	-	75.5-78	SM*	42
6	-	74-81.5	CL*	65
7	-	75-81	ML*	50
8	54	76-83	SM-ML	-
9	-	73-79	SM-CL*	40 77 90
10	-	-	-	-
11	44	76.5-82	CL*	75
12	+100	76-82	ML*	84
13	-	77-82	CL*	87
14	-	77-82	ML-CL*	96
15	54	72-78	CL*	82
16	55	72-79	CL-SC*	51
17	81	75-78	CL*	73
18	-	72-77.5	ML-CL*	58

*In accordance with ASTM D-2487-69. Other classifications in accordance with ASTM D-2488-69.

Table 2T-10 (cont.)

SUMMARY OF STRATIGRAPHIC SEQUENCE F, UNIT 3

Unit Boring	N Value	Depth Below Ground Surface in Feet	Unified Soil Classification	Percent Passing #200 Sieve
1	-	70-74	CL-ML	-
2	-	72-79	ML	-
3	-	70-76	CL* ML-CL	95
4	-	70-78	ML-CL*	83
5	-	70-78	SM	24
6	-	64-70	ML	-
7	-	72.5-75	CL*	97
9	-	73-78	ML	-
10	-	70-77	ML	-
12	50	72-78	ML	-
13	-	72-76	ML	79
14	-	-	-	-
15	-	74-78.5	ML	-
16	-	66-72	CL*	79
17	-	73-77	ML	-
18	-	73-77	ML	-

*In accordance with ASTM D-2487-69. Other classifications in accordance with ASTM D-2488-69.

Table 2T-11

AVERAGE INDUCED STRESSES AND EQUIVALENT CYCLES
UNIT 3

Input Record	Layer No. 7 31'-35'; σ_o' =3920 psf		Layer No. 9 40'-45'; σ_o' =4540 psf		Layer No. 15 70'-75'; σ_o' =6370 psf	
	Average Stress τ_{ave}, psf	Equivalent Cycles[1]	Average Stress τ_{ave}, psf	Equivalent Cycles[1]	Average Stress τ_{ave}, psf	Equivalent Cycles[1]
Santa Barbara	336	11	435	11	720	9
Hollywood P.E. Lot	246	26	312	23	493	21
Hollywood Storage (Basement)	228	26	289	25	462	23
Pasadena	237	20	299	20	520	18
Bechtel	505	20	640	17	954	15

[1]Calculated values

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Table 2T-11a

AVERAGE INDUCED STRESSES AND EQUIVALENT CYCLES
UNIT 2

Input Record	Layer No. 9 42'-46'; σ_o' =5120 psf		Layer No. 10 46'-51'; σ_o' =5390 psf		Layer No. 15 71'-76'; σ_o' =6780 psf	
	Average Stress τ_{ave}, psf	Equivalent Cycles[1]	Average Stress τ_{ave}, psf	Equivalent Cycles[1]	Average Stress τ_{ave}, psf	Equivalent Cycles[1]
Santa Barbara	406	11	441	11	621	9
Hollywood P. E. Lot	300	23	326	23	436	21
Hollywood Storage (Basement)	279	25	302	25	409	24
Pasadena	293	20	337	20	488	18
Bechtel	585	16	614	16	767	16

[1]Estimated from Unit 3 analyses.

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Table 2T-12

SUMMARY OF FACTORS OF SAFETY
UNIT 2

Input Record	Layer No. 9 42'-46'; σ_o' =5120 psf			Layer No. 10 46'-51'; σ_o' =5390 psf			Layer No. 15 71'-76'; σ_o' =6780 psf		
	Equivalent Cycles	Strength, psf	Factor of Safety	Equivalent Cycles	Strength, psf	Factor of Safety	Equivalent Cycles	Strength, psf	Factor of Safety
Santa Barbara	11	845	2.1	11	883	2.0	9	1627	2.6
Hollywood P.E. Lot	23	768	2.6	23	809	2.5	21	1390	3.2
Hollywood Storage (Basement)	25	768	2.8	25	809	2.7	34	1356	3.3
Pasadena	20	794	2.7	20	836	2.5	18	1424	2.9
Bechtel	16	819	1.4	16	862	1.4	16	1458	1.9

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Table 2T-12a

SUMMARY OF FACTORS OF SAFETY
UNIT 3

Input Record	Layer No. 7 31'-35'; σ_o' =3920 psf			Layer No. 8 35'-40'; σ_o' =4220 psf		
	Equivalent Cycles	Strength, psf	Factor of Safety	Equivalent Cycles	Strength, psf	Factor of Safety
Santa Barbara	11	643	1.9	11	692	1.8
Hollywood P. E. Lot	26	596	2.4	24	633	2.3
Hollywood Storage (Basement)	26	596	2.6	26	624	2.5
Pasadena	20	607	2.6	20	654	2.4
Bechtel	20	607	1.2	19	658	1.2

Table 2T-12a (continued)

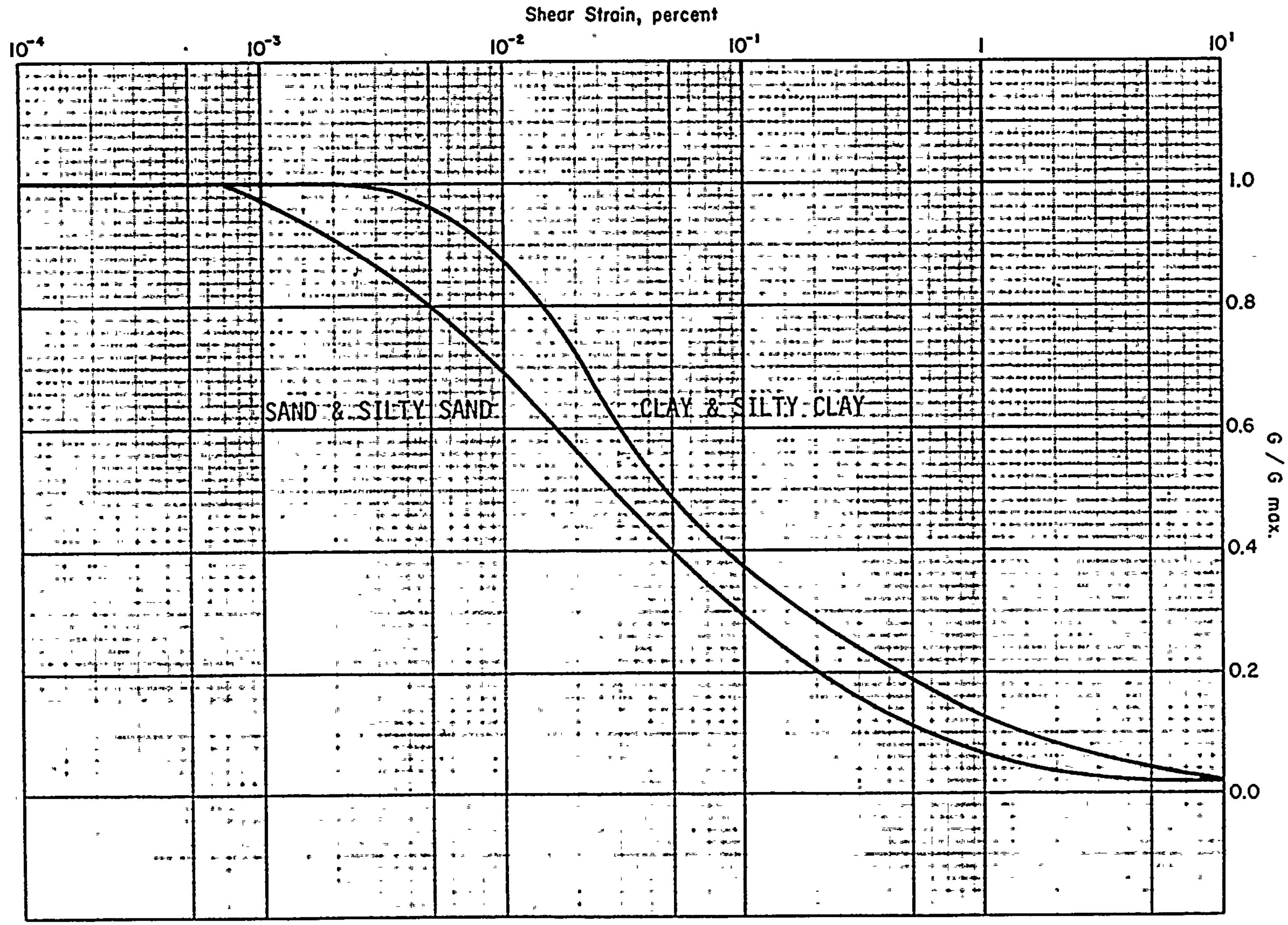
SUMMARY OF FACTORS OF SAFETY
UNIT 3

Input Record	Layer No. 9 40'-45'; σ_o' =4540 psf			Layer No. 15 70'-75'; σ_o' =6370 psf		
	Equivalent Cycles	Strength, psf	Factor of Safety	Equivalent Cycles	Strength, psf	Factor of Safety
Santa Barbara	11	749	1.7	9	1529	2.1
Hollywood P. E. Lot	23	681	2.2	21	1306	2.6
Hollywood Storage (Basement)	25	681	2.4	24	1274	2.8
Pasadena	20	704	2.4	18	1338	2.6
Bechtel	17	726	1.1	17	1370	1.5

APPROVED BY: J. Chak DATE: 10/15/75 PREPARED BY: ________ CHECKED BY: W. R. Anders

SYMBOL	BORING NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (ft.)	SOIL TYPE	CONFINING PRESSURE (psi.)	G max. X 10^6 (psf.)
	UNIT #2		–	SAND & SILTY SAND		*
	UNIT #2			CLAY & SILTY CLAY		*

*As noted on Soil Models (Tables 2T-8 and 2T-8a)



MODULUS VS. STRAIN

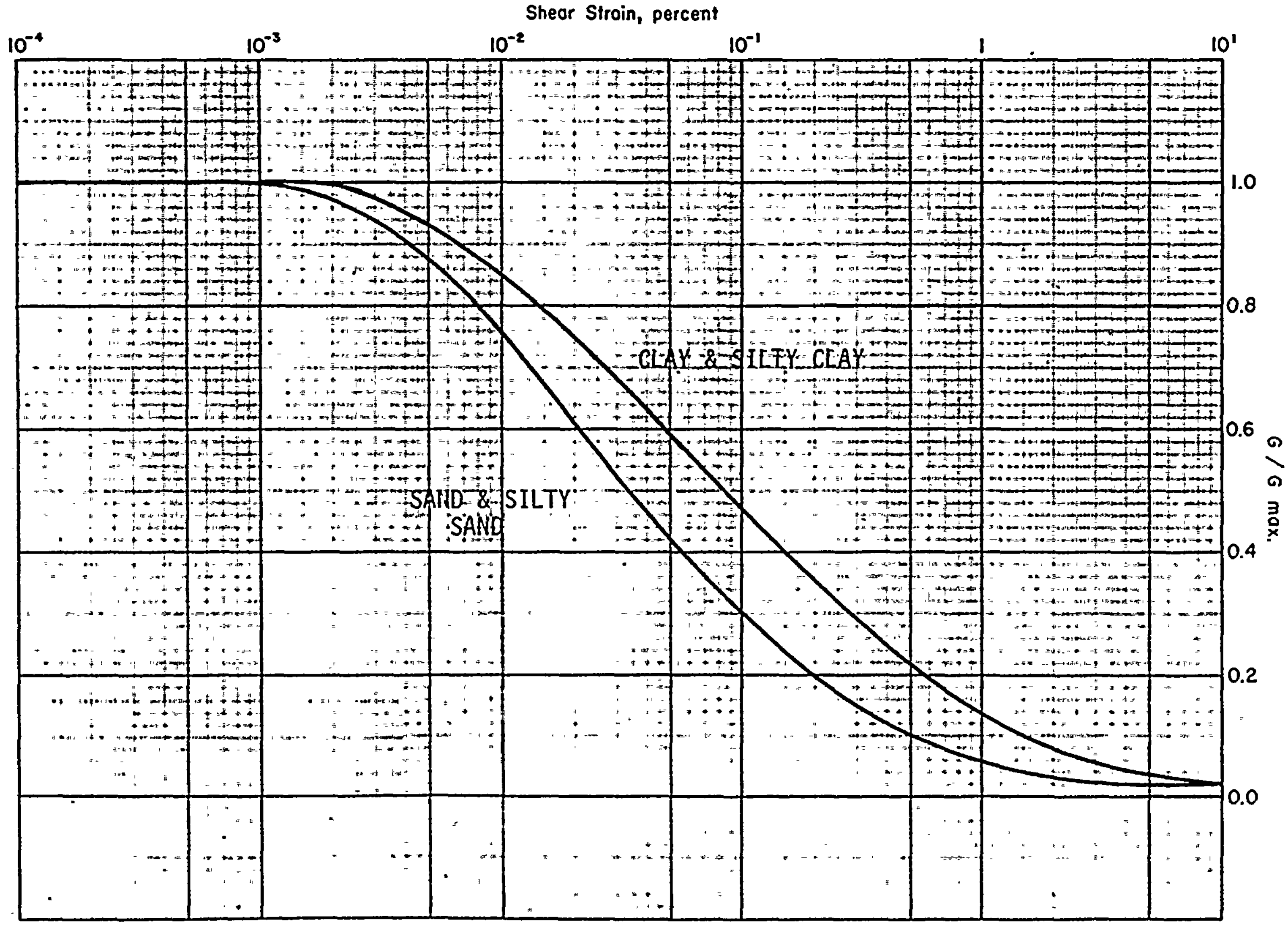
UNIT 2 - LIQUEFACTION ANALYSIS

Figure 2T-9

APPROVED BY: T. Chak DATE: 10/15/75 PREPARED BY: ________ CHECKED BY: W.K. [signature]

SYMBOL	BORING NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (ft.)	SOIL TYPE	CONFINING PRESSURE (psi)	G max. X 10^6 (psf.)
	UNIT #3			Clay & Silty	Clay	*
	UNIT #3			Sand & Silty	Sand	*

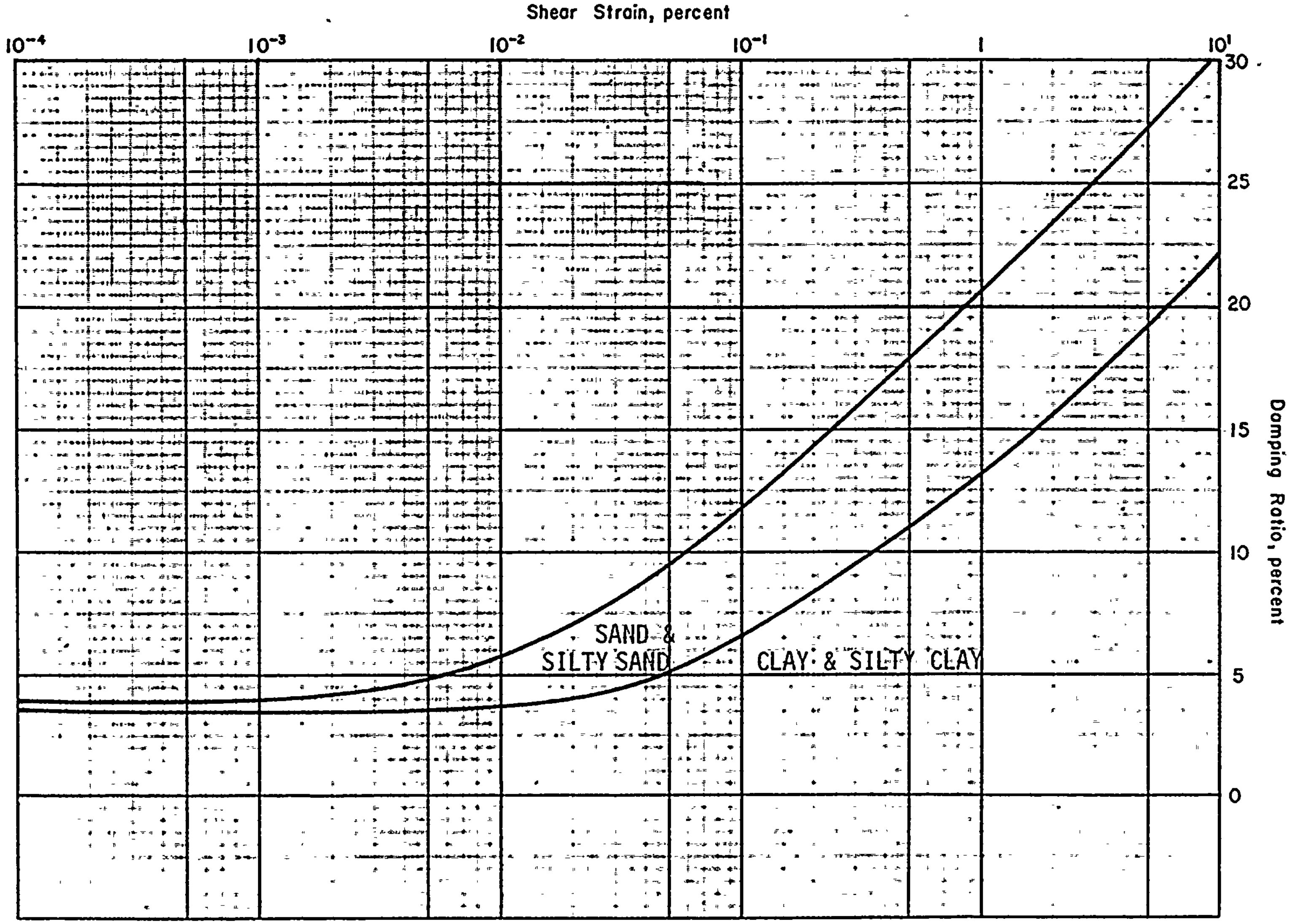
*As noted on Soil Models (Tables 2T-8 and 2T-8a)



MODULUS VS. STRAIN
UNIT 3 - LIQUEFACTION ANALYSIS
Figure 2T-9a

APPROVED BY: J. Chak DATE: 10/15/75 PREPARED BY: __________ CHECKED BY: M.R. [illegible]

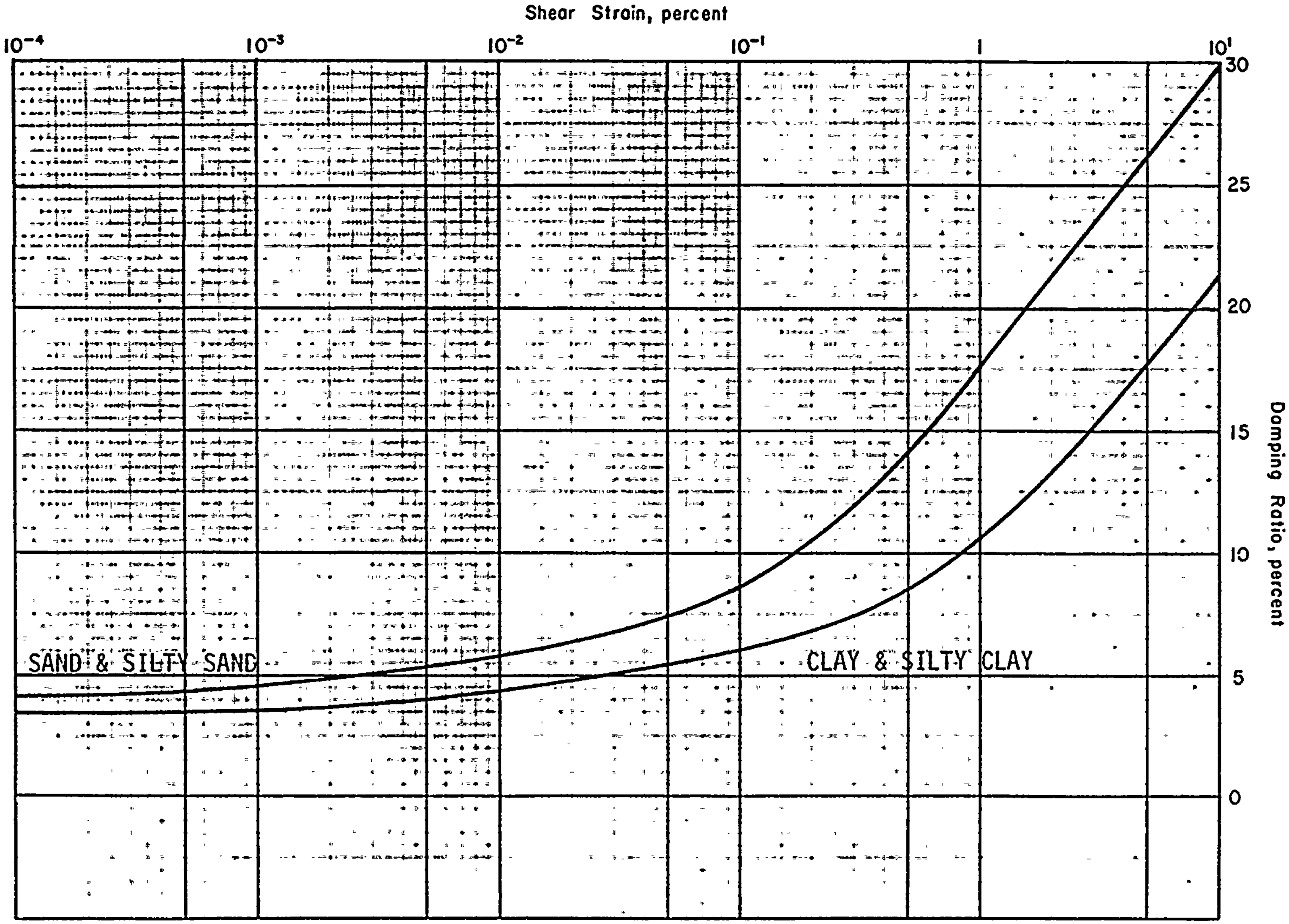
SYMBOL	BORING NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (ft.)	SOIL TYPE	CONFINING PRESSURE (psi.)
	UNIT #2	-	-	-	
	UNIT #2	-	-	-	



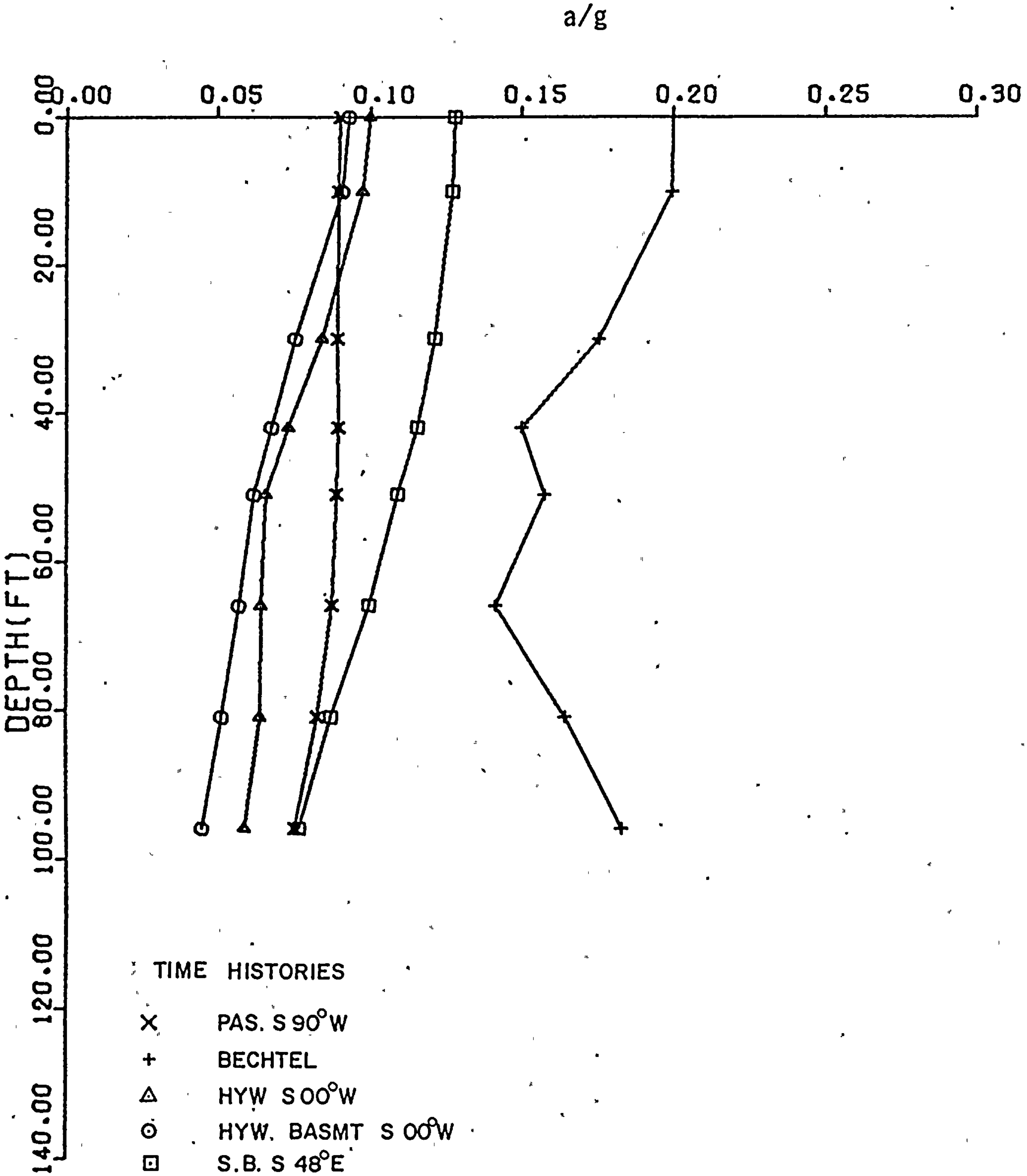
DAMPING VS. STRAIN
UNIT 2 - LIQUEFACTION ANALYSIS
Figure 2T-9b

APPROVED BY: J. Chak DATE: 10/15/75 PREPARED BY: ________ CHECKED BY: W.R. [illegible]

SYMBOL	BORING NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (ft.)	SOIL TYPE	CONFINING PRESSURE (psi.)
	UNIT #3			Clay & Silty	Clay
	UNIT #3			Sand & Silty	Sand



DAMPING VS. STRAIN
UNIT 3 - LIQUEFACTION ANALYSIS
Figure 2T-9c

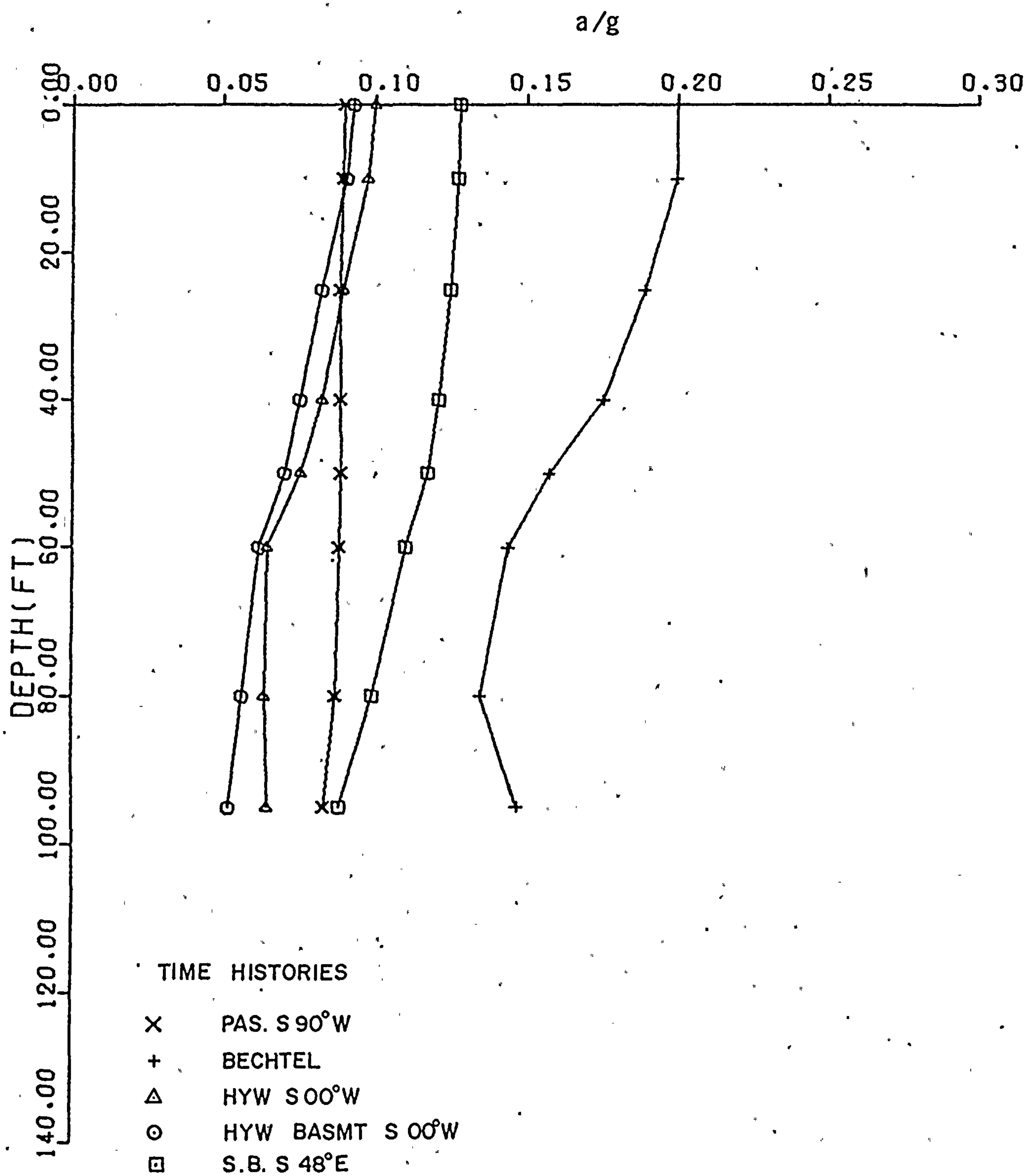


APPROVED BY: [illegible] DATE: 12-17-75 PREPARED BY: K. Chou CHECKED BY: J. Chak

ACCELERATIONS (a/g) VS. DEPTH
UNIT 2

Figure 2T-9d

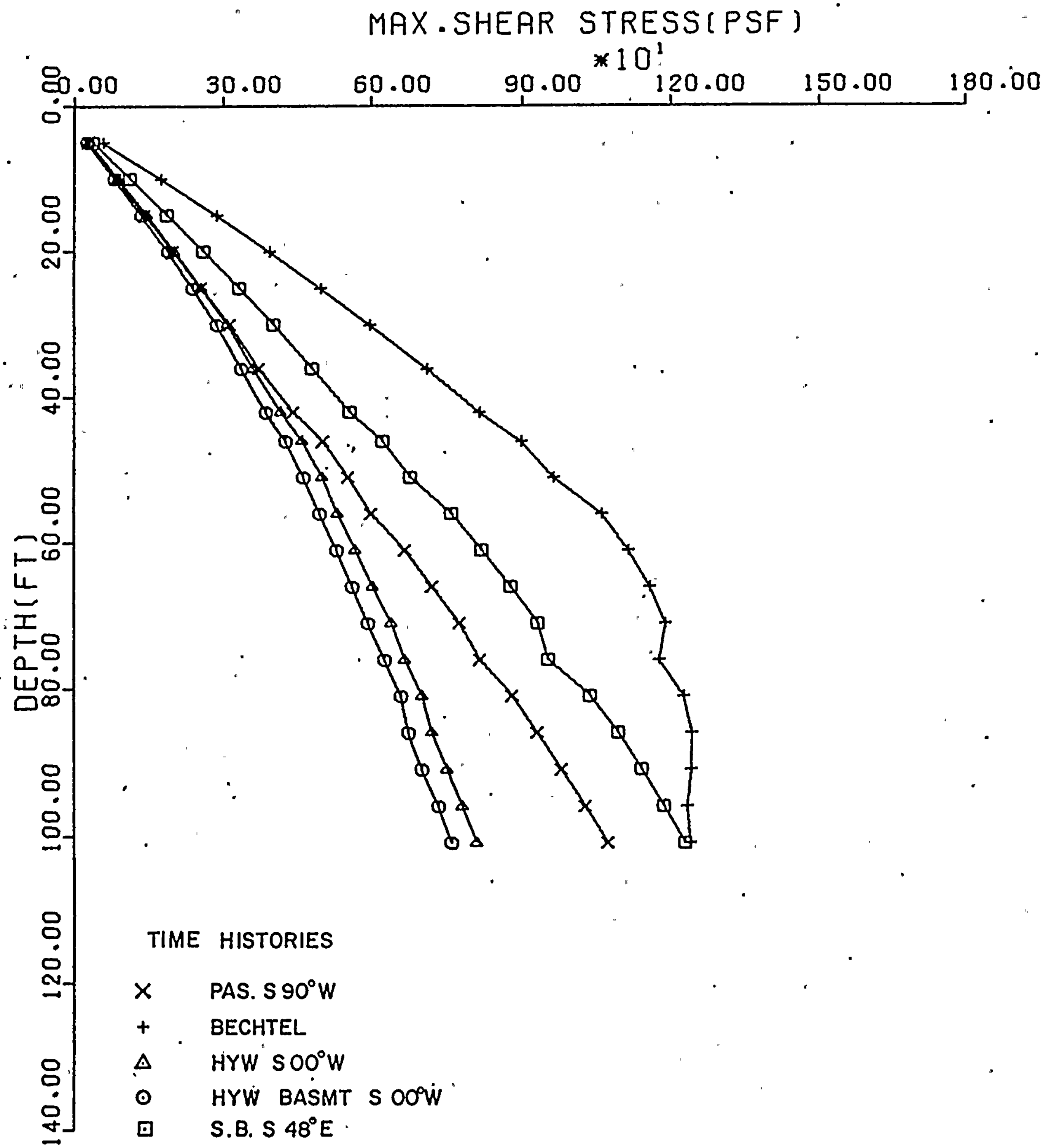
APPROVED BY: [redacted] DATE: [redacted] PREPARED BY: [redacted] CHECKED BY: J. Chuk



ACCELERATIONS (a/g) VS. DEPTH
UNIT 3

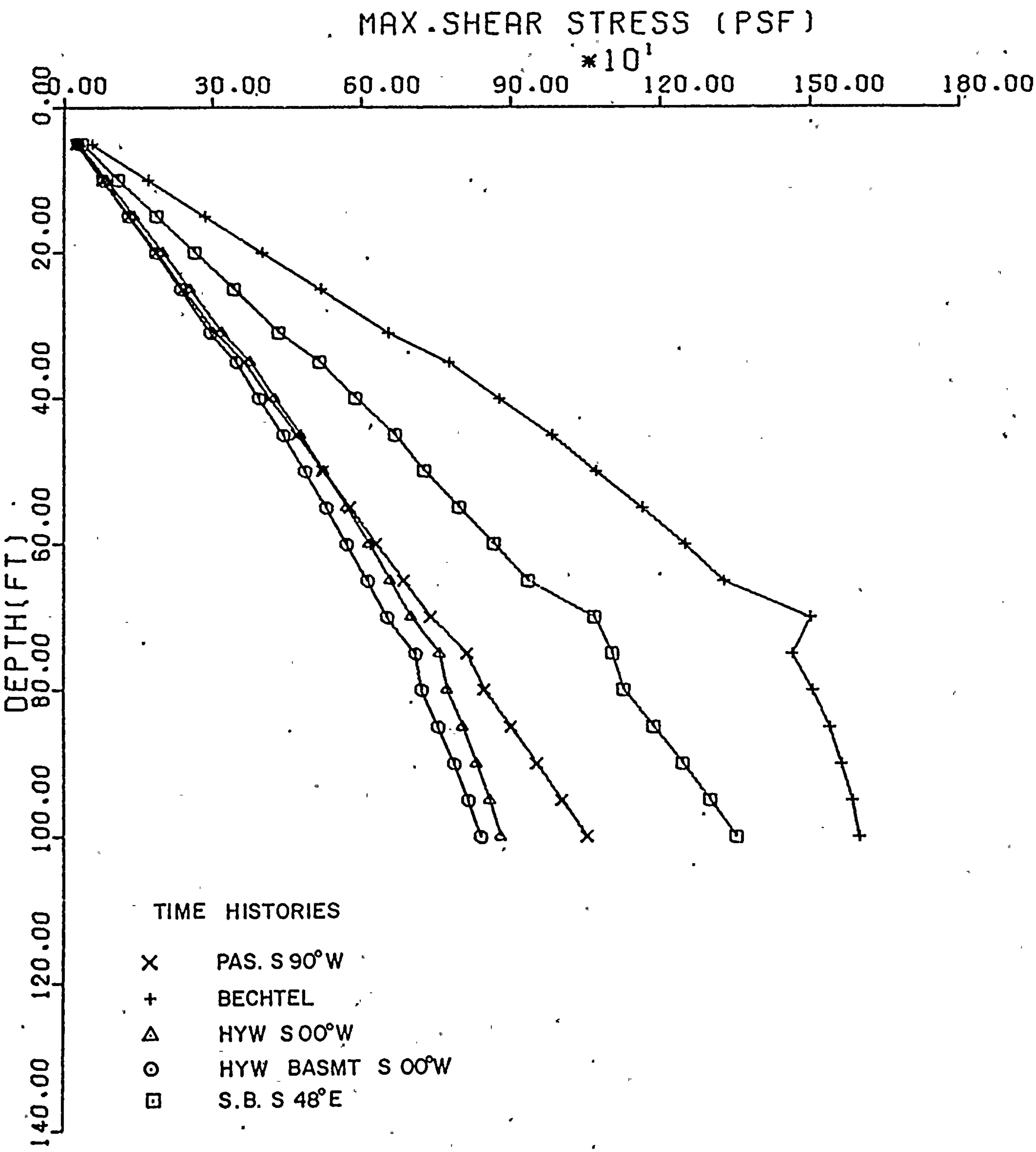
Figure 2T-9e

APPROVED BY: DATE: 10-17-75 PREPARED BY: K. Chen CHECKED BY:



MAXIMUM SHEAR STRESS VS. DEPTH
UNIT 2

Figure 2T-9f



APPROVED BY: [signature] DATE: 10-17-75 PREPARED BY: [signature] CHECKED BY: [signature]

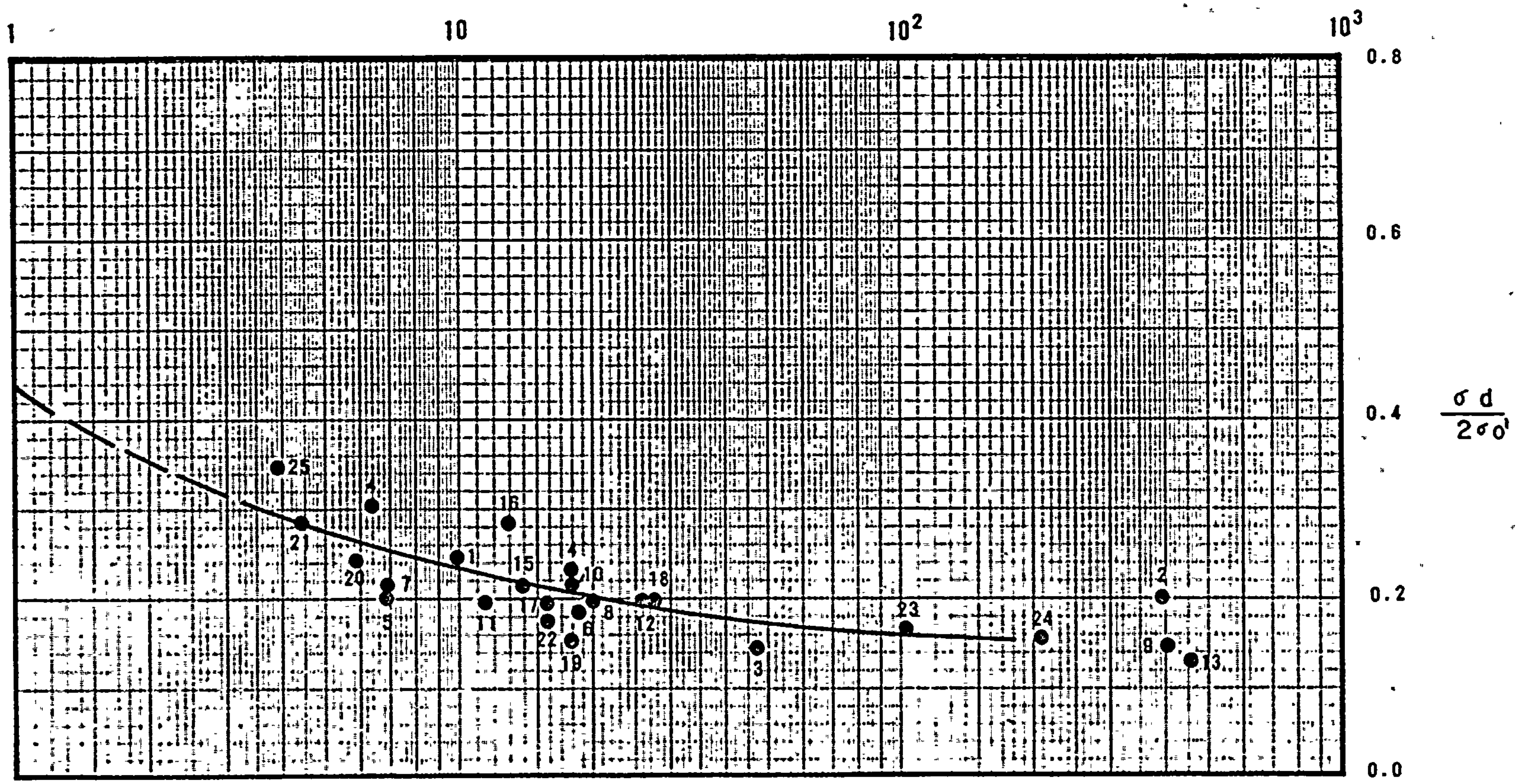
MAXIMUM SHEAR STRESS VS. DEPTH
UNIT 3

Figure 2T-9g

APPROVED BY: P. Chaney DATE: 10/14/75 PREPARED BY: P. Kornyboiled CHECKED BY: M. K. Ueda

SUMMARY PLOT — DYNAMIC SOIL STRENGTH

PROJECT NUMBER	LOCATION	DEPTH INTERVAL (ft)	SOIL TYPE	CRITERIA	SAMPLING METHOD
72-086-EG	UNIT 1, UNIT 2, UNIT 3	30 – 50	GRANULAR	$\varepsilon da = 5.0\%$ OR INTL. LIQ.	PITCHER

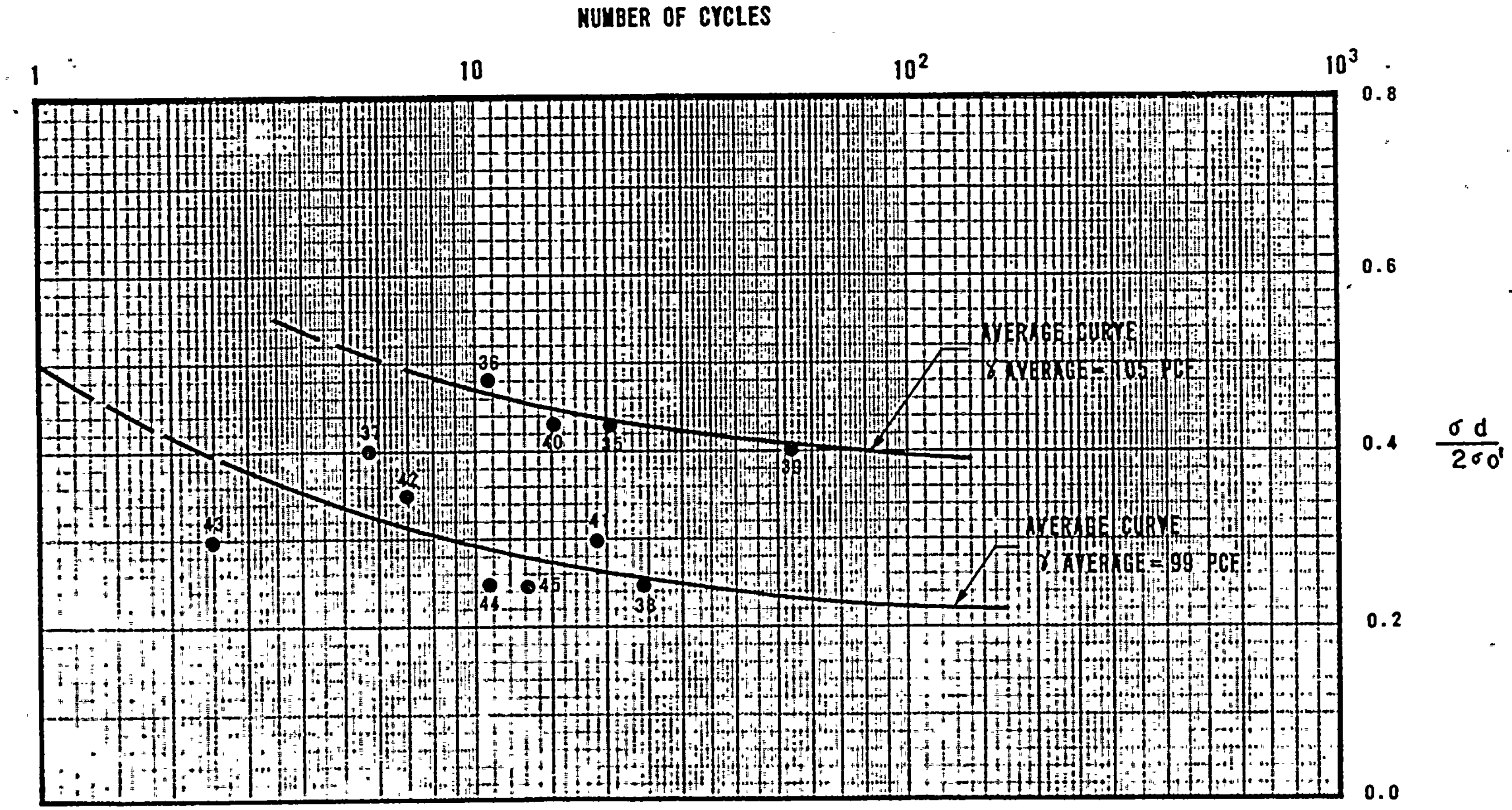


CYCLIC STRENGTHS, 30'-50'
PITCHER SAMPLES
Figure 2T-10

APPROVED BY: R. C. [illegible] DATE: 10/14/75 PREPARED BY: P. K. [illegible] CHECKED BY: [illegible]

SUMMARY PLOT – DYNAMIC SOIL STRENGTH

PROJECT NUMBER	LOCATION	DEPTH INTERVAL (ft)	SOIL TYPE	CRITERIA	SAMPLING METHOD
72-086-EG	UNIT 2, UNIT 3	30 – 50	GRANULAR	εda = 5.0% OR INTL. LIQ	BLOCK

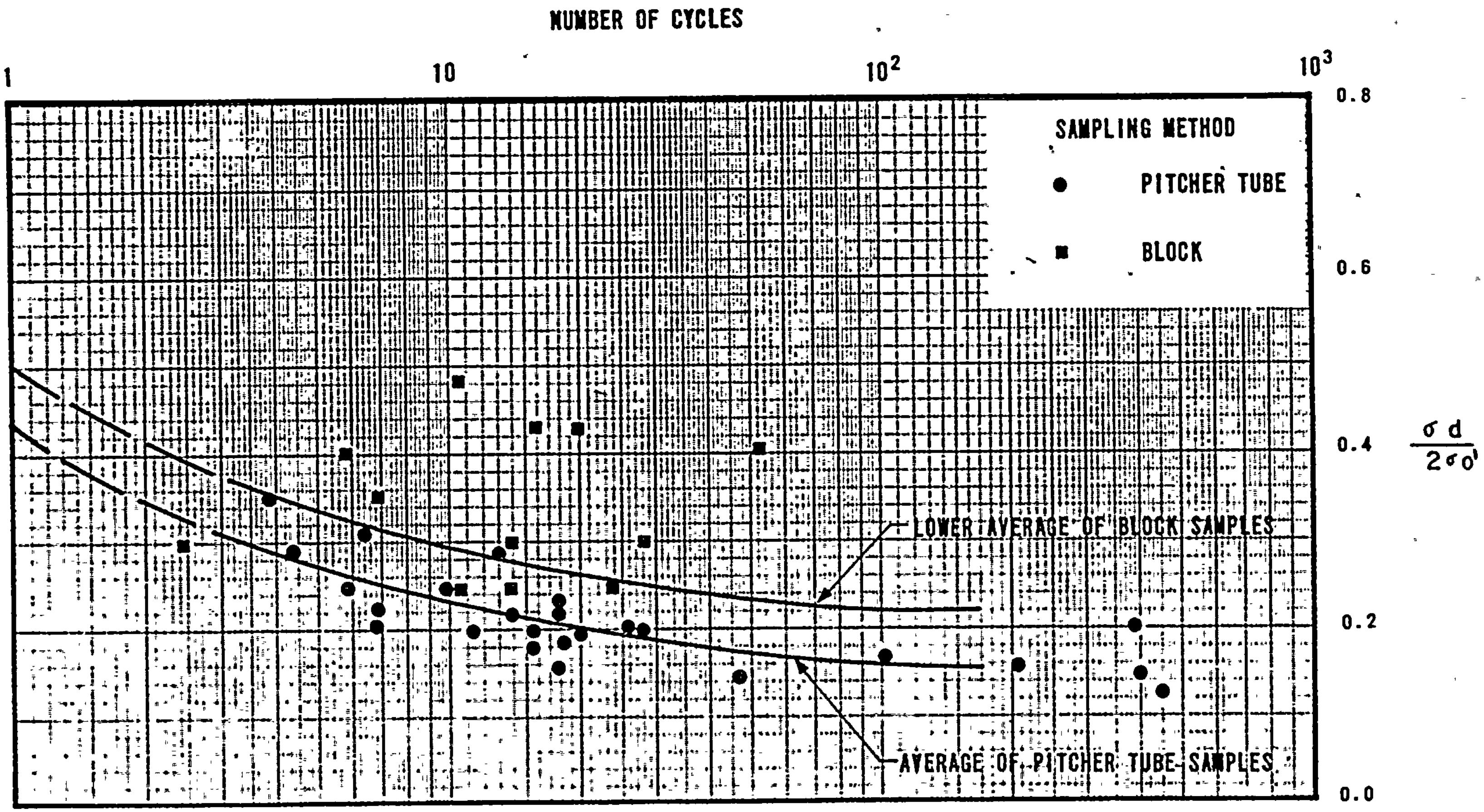


CYCLIC STRENGTHS, 30'-50'
BLOCK SAMPLES
Figure 2T-10a

APPROVED BY: R. C. Chaney DATE: 10/14/75 PREPARED BY: P. Kramph... CHECKED BY:

SUMMARY PLOT — DYNAMIC SOIL STRENGTH

PROJECT NUMBER	LOCATION	DEPTH INTERVAL (ft)	SOIL TYPE	CRITERIA
72-086-EG	UNIT 1, UNIT 2, UNIT 3	30 - 50	GRANULAR	$\varepsilon da = 5.0\%$ OR INTL. LIQ.

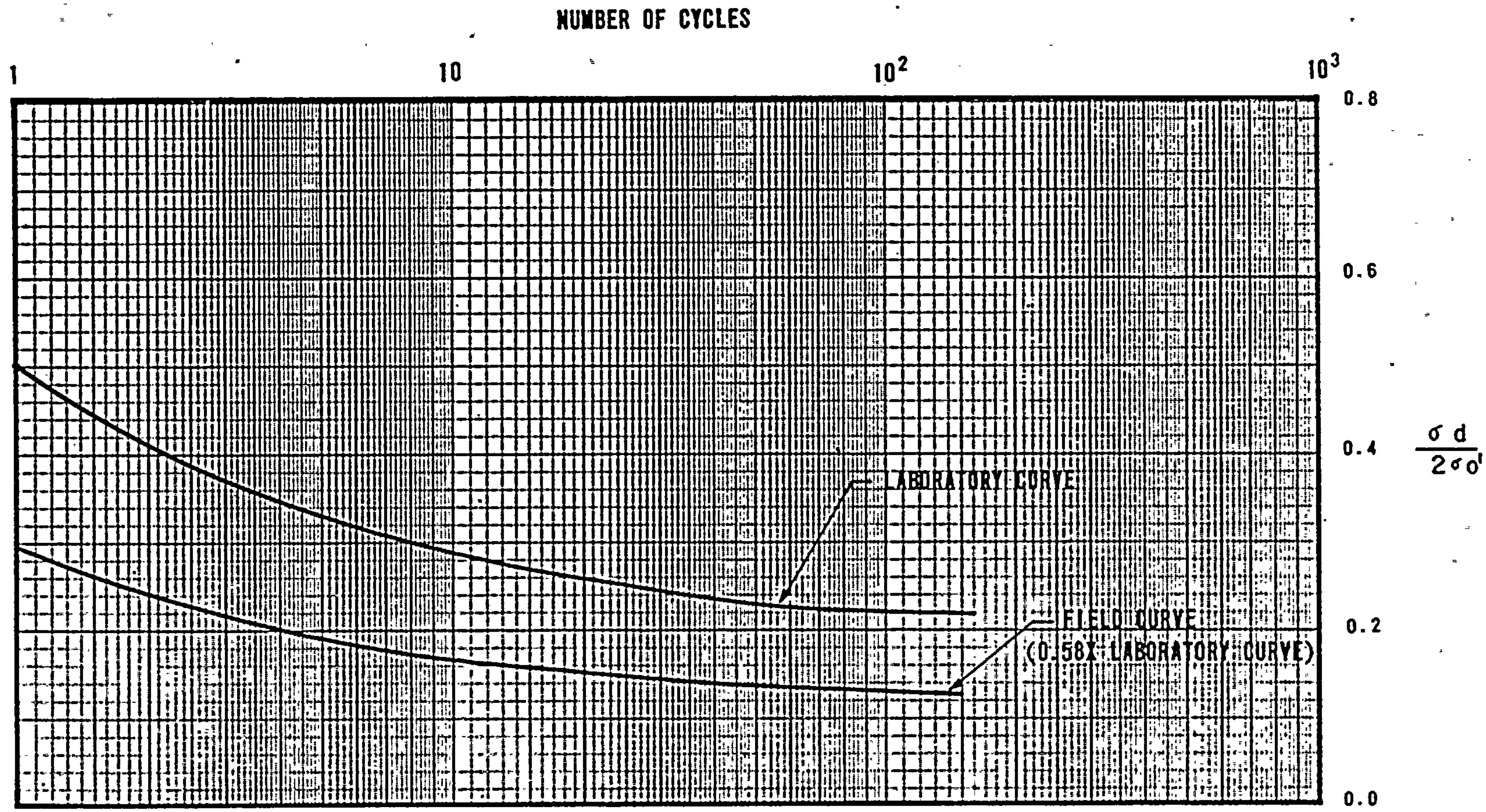


CYCLIC STRENGTHS, 30'-50'
PITCHER VS. BLOCK SAMPLES
Figure 2T-10b

APPROVED BY: R. Chaney DATE: 10/14/75 PREPARED BY: P. Kremphaibul CHECKED BY: W. K. Rueter

SUMMARY PLOT – DYNAMIC SOIL STRENGTH

PROJECT NUMBER	LOCATION	DEPTH INTERVAL (ft)	SOIL TYPE	CRITERIA
72-086-EG	UNIT 1, UNIT 2, UNIT 3	30 – 50	GRANULAR	εda = 5.0% OR INTL. LIQ.

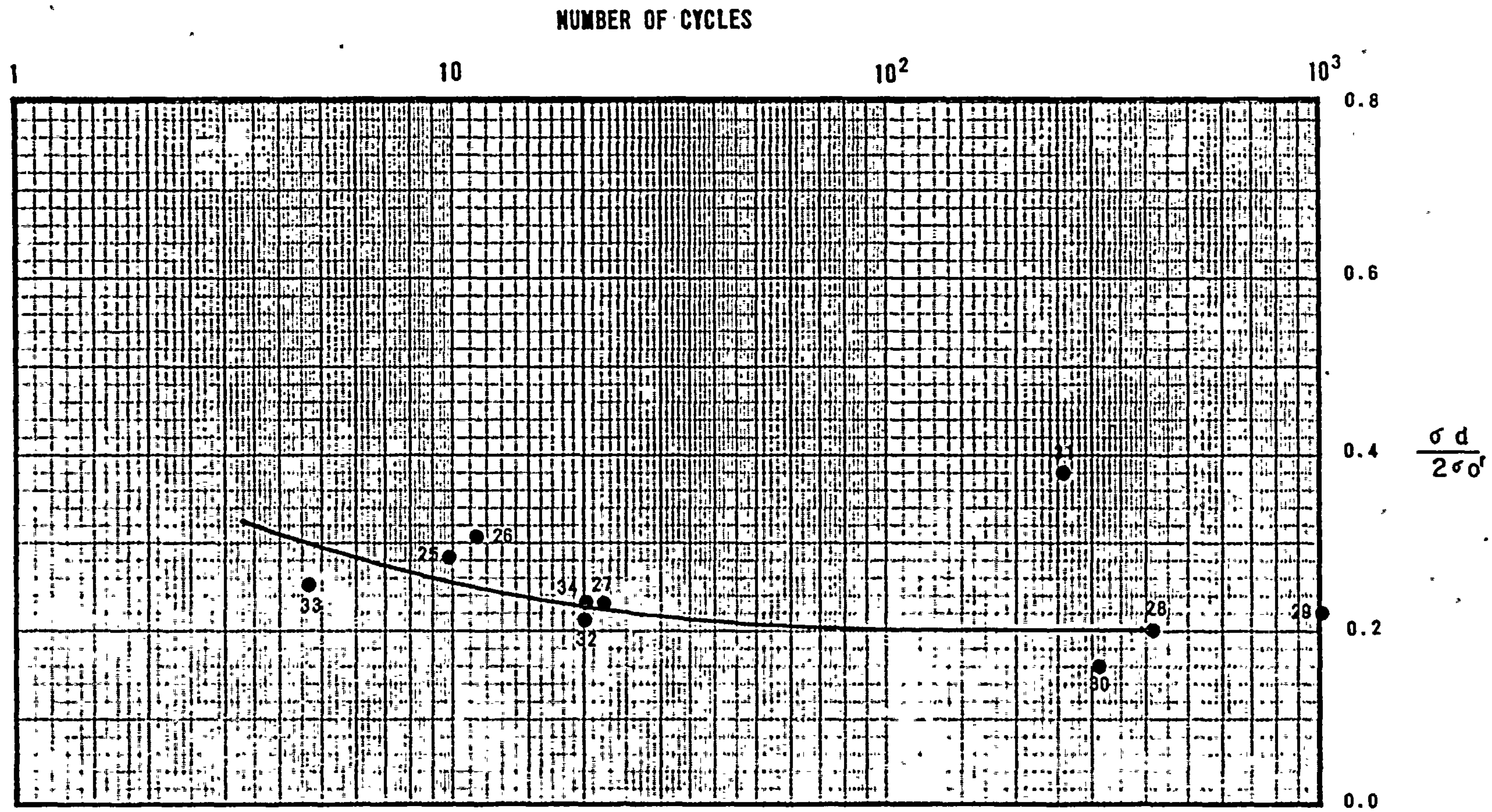


CYCLIC STRENGTHS, 30'-50'
BLOCK SAMPLES
LABORATORY VS. FIELD
Figure 2T-10c

APPROVED BY: R. C. Chaney DATE: 10/14/75 PREPARED BY: [illegible] CHECKED BY: [illegible]

SUMMARY PLOT – DYNAMIC SOIL STRENGTH

PROJECT NUMBER	LOCATION	DEPTH INTERVAL (ft)	SOIL TYPE	CRITERIA	SAMPLING METHOD
72-086-EG	UNIT 1, UNIT 2	70 – 80	GRANULAR PORTIONS ONLY	ε_{da} = 5.0% OR INTL. LIQ	PITCHER

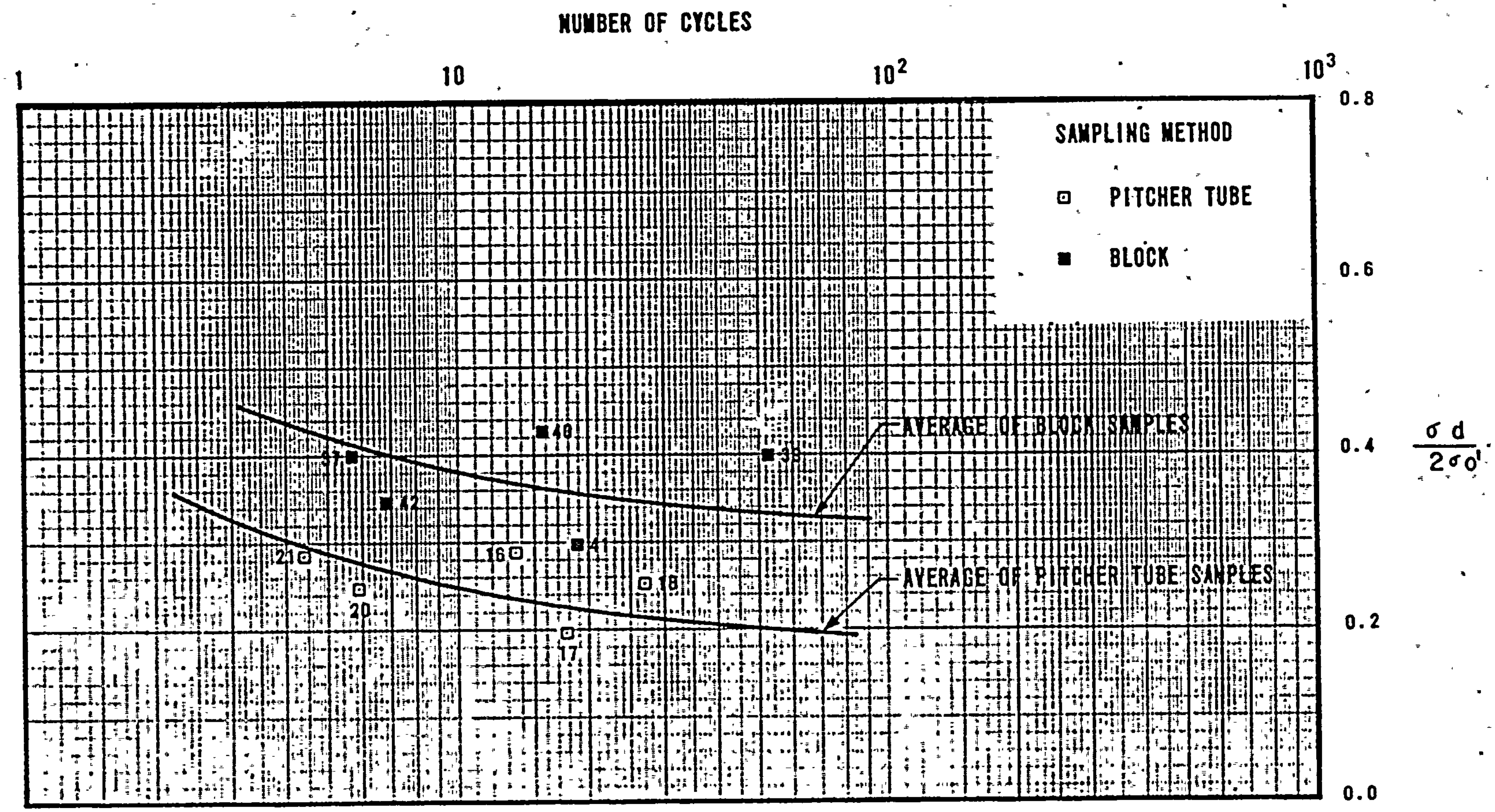


CYCLIC STRENGTHS, 70'-80'
PITCHER SAMPLES
FIGURE 2T-10d

APPROVED BY: DATE: 10/14/75 PREPARED BY: CHECKED BY:

SUMMARY PLOT – DYNAMIC SOIL STRENGTH

PROJECT NUMBER	LOCATION	DEPTH INTERVAL (ft)	SOIL TYPE	CRITERIA
72-086-EG	UNIT 1, UNIT 2	30 – 50	≥17% PASSING #200	ϵ_{da} 5.0% OR INTL. LIQ

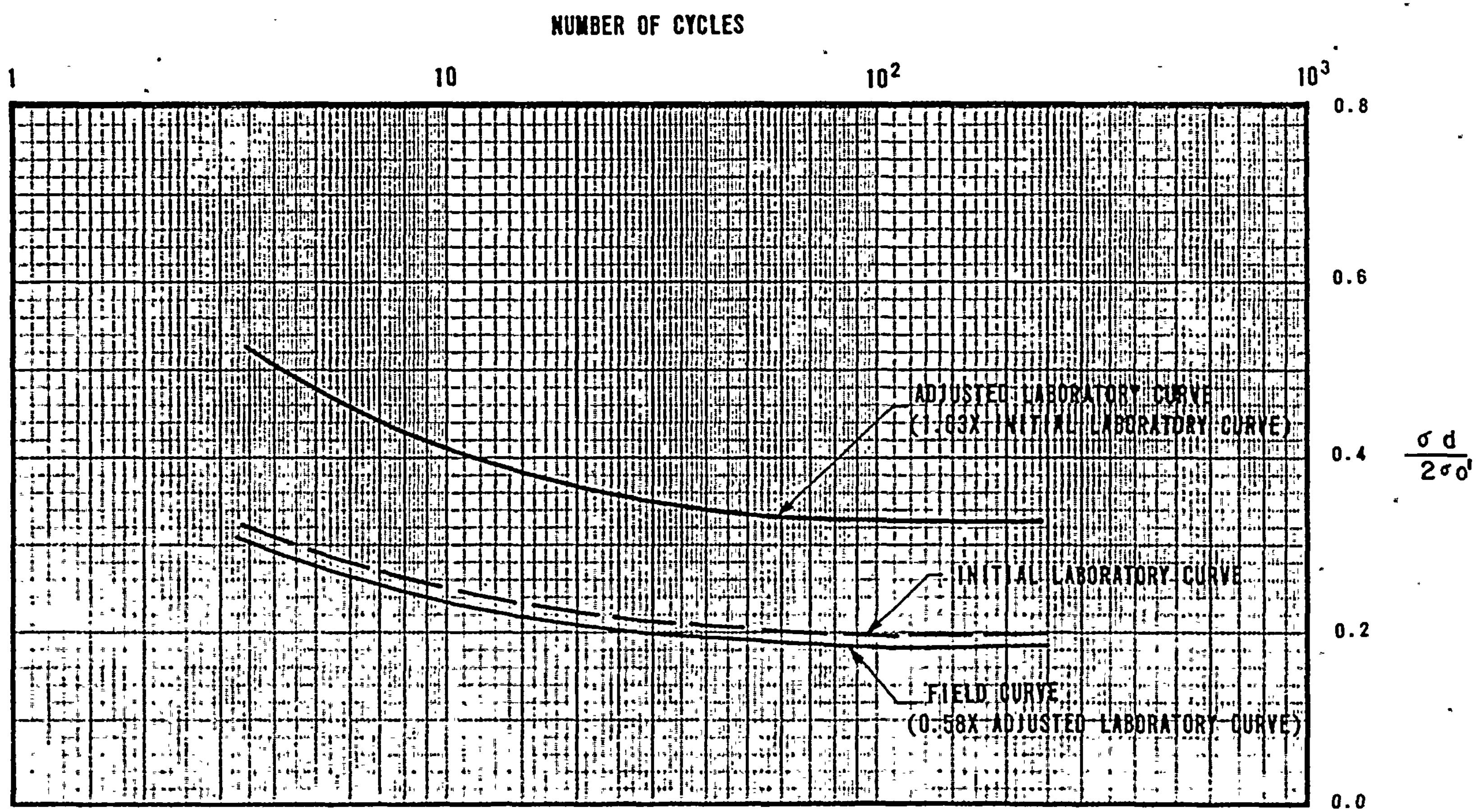


CYCLIC STRENGTHS, 30'-50'
PITCHER VS. BLOCK SAMPLES
≥ 17% PASSING #200 SIEVE
FIGURE 2T-10e

APPROVED BY: P. Chaney DATE: 10/14/75 PREPARED BY: P. Kramphaus CHECKED BY: W R Cain

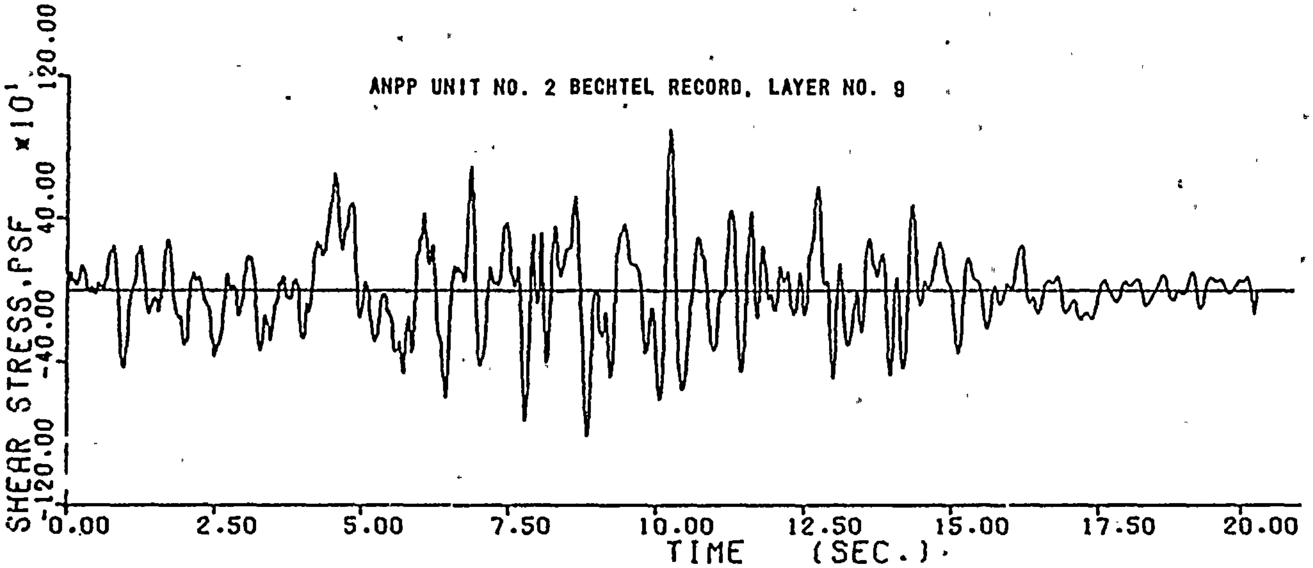
SUMMARY PLOT — DYNAMIC SOIL STRENGTH

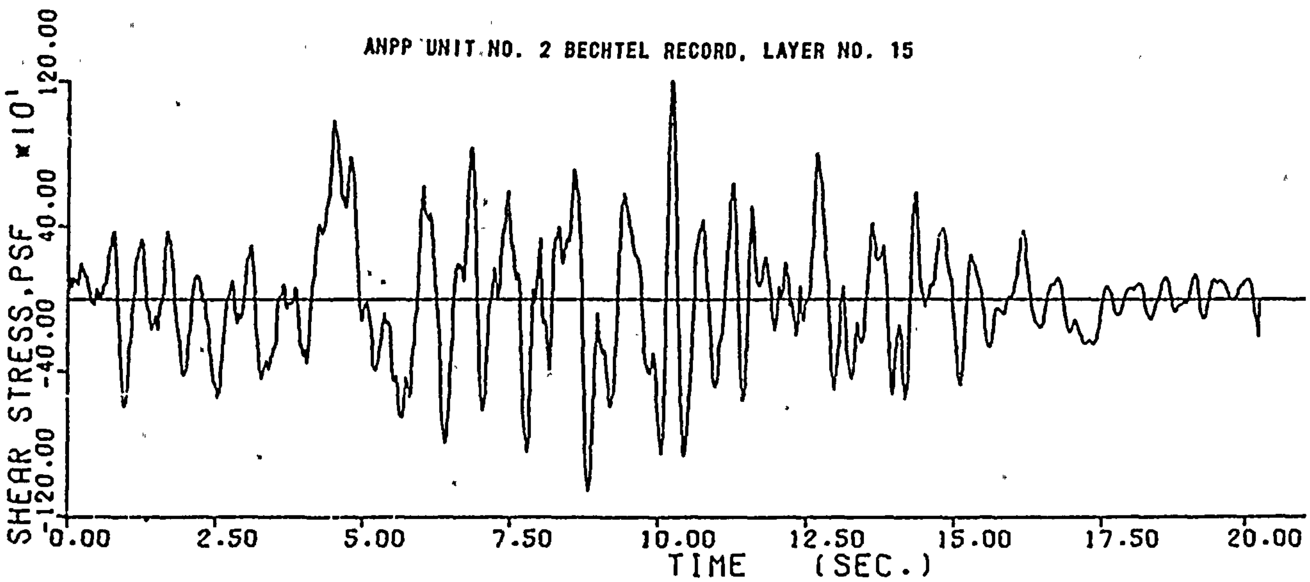
PROJECT NUMBER	LOCATION	DEPTH INTERVAL (ft)	SOIL TYPE	CRITERIA	SAMPLING METHOD
72-086-EG	UNIT 1, UNIT 2	70 - 80	GRANULAR	$\varepsilon da = 5.0\%$ OR INTL. LIQ.	PITCHER



CYCLIC STRENGTHS, 70'-80'
PITCHER TUBE SAMPLES
LABORATORY VS. FIELD
FIGURE 2T-10f

APPROVED BY [redacted] DATE [redacted] PREPARED BY [redacted] CHECKED BY [redacted]

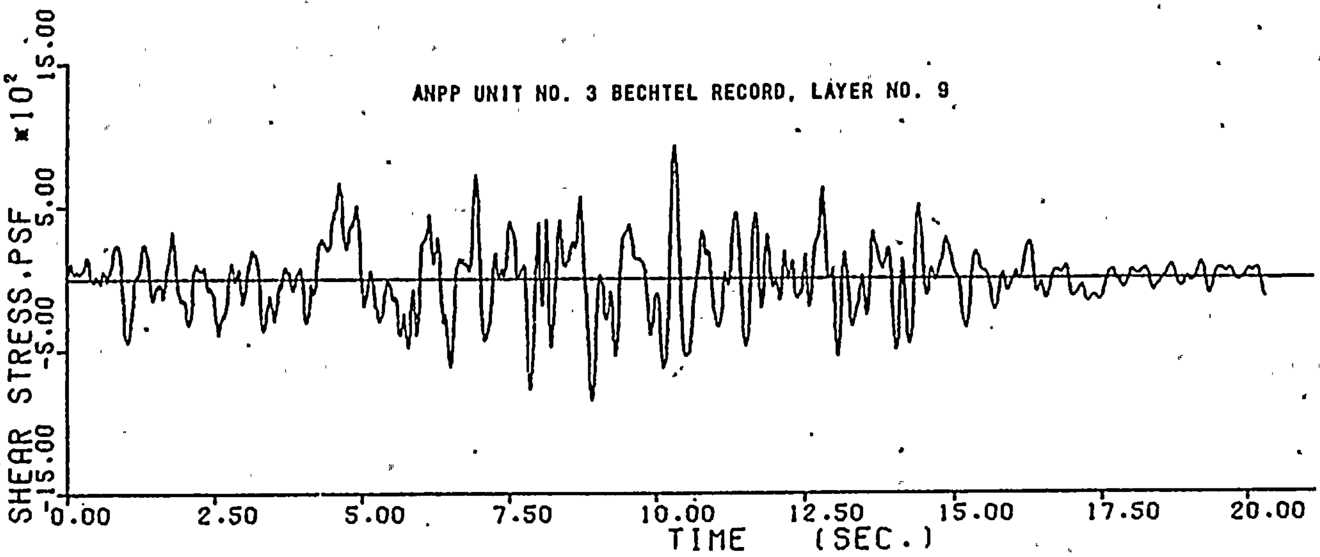


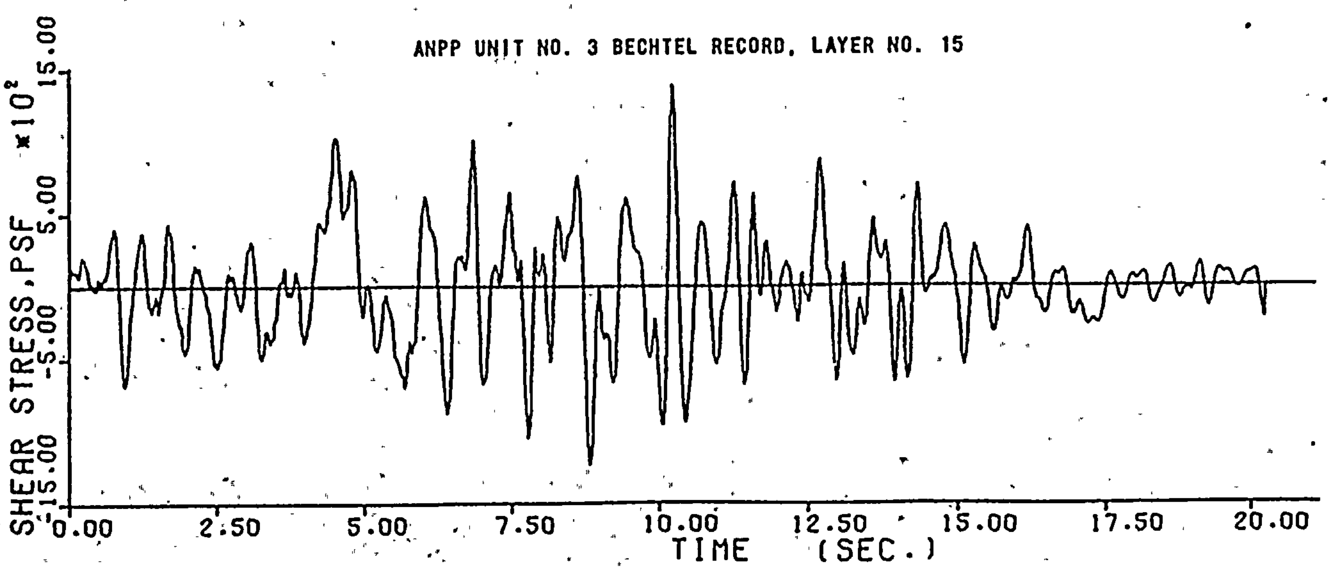


SHEAR STRESS-TIME HISTORIES
LAYER NOS. 9 and 15
UNIT 2

Figure 2T-10g

APPROVED BY: DATE: PREPARED BY: CHECKED BY:





SHEAR STRESS-TIME HISTORIES
LAYER NOS. 9 and 15
UNIT 3

Figure 2T-10h