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EVALUATION OF SLOPE STABILITY PROCEDURES

-

ALASKA SEGMENT

ALASKA NATURAL GAS TRANSPORTATION SYSTEM

TECHNICAL EVALUATION REPORT

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ALASKA SEGMENT
ALASKA NATURAL GAS TRANSPORTATION SYSTEM

09-TR-0025
DECEMBER 1981

CONTRACT NUMBER
OFI 80-0001

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TECHNICAL SUPPORT TO THE
OFFICE OF THE FEDERAL INSPECTOR

SLOPE STABILITY STUDY
-
ALASKA SEGMENT
ALASKA NATURAL GAS TRANSPORTATION SYSTEM

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1.0 INTRODUCTION

This report presents the Technical Support Contractor's (TSC) Terrain Stability Study performed in preparation for the upcoming meeting scheduled with NWA on December 15-16, 1981. The purpose of the study was to verify NWA's slope stability procedures by applying it to some selected cases.

The NWA procedures for thaw plug stability and liquefaction presented in the Pipeline Design Criteria Manual, Geotechnical Assessment, Volume 4 were followed in conducting this study. In addition, computer analyses were made using the Slope II Computer Program.

Two slopes were selected for analyses. The first case analyzed is a slope transverse to the pipeline at Mile Post (MP) 206.35, Alignment Sheet 37. At this location the NWA pipeline alignment is about 250 to 350 feet upslope of the elevated oil pipeline. Thaw plug stability was checked for NWA generalized soil conditions and for one selected borehole. Liquefaction analyses were made for two boreholes.

The second slope analyzed is a longitudinal slope along the centerline of the pipeline at MP 671, Alignment Sheet 119. At this location the pipeline route follows the Haines right-of-way about 600 feet from the Alaska Highway. Thaw plug stability analyses were performed for the centerline profile and liquefaction analysis was done for one sample.

This report should be read in conjunction with the confidential/proprietary information transmitted to Mr. W. T. Black of OFI by Mr. George P. Wuerch of NWA in a letter dated November 25, 1981. This study is based entirely on the data provided by NWA. The equations referred to in this report are found in Section 5.0 of the Pipeline Design Criteria Manual, Geotechnical Assessment, Volume 4.

2.0 FINDINGS AND CONCLUSIONS

(1) For the boreholes checked, the liquefaction analysis results presented by NWA generally agree with the results obtained by the TSC using the same procedure. For high ice content soils, however, the procedure cannot be applied because the dry densities are too low for the graphs presented (Figures 5-63 to 5-68).

(2) For the boreholes checked, the thaw plug stability results presented by NWA generally agree with the results obtained by the TSC using the same procedure. The NWA method also agreed reasonably well with the computer analysis.

(3) For the dynamic stability analysis the displacements computed by NWA, when the safety factors are below one, and the displacements computed by the TSC are compatible.

(4) NWA does not appear to use the strength parameters presented in Table 4-2 in the slope stability analysis.

(5) The "average" soil conditions chosen by NWA for each pipeline segment are generally conservative. However, these "average" soil conditions do not account for the worst condition. An example of such a case is the 25 year thaw plug stability near Borehole N 77-67 shown on Alignment Sheet 37. Adequate stability was shown using the "average" soil conditions, but a thaw plug failure resulted using the soil conditions in Borehole N 77-67 (see Sections 4.3.1 and 4.3.2 of this report).

(6) The results of the short term thaw plug stability analysis depend to a significant extent on cohesion. For the analysis performed at Borehole N 77-66, the value of C_{cu} (100 PSF) appears relatively low but it is high enough to prevent short term failure (shallow depth). On the other hand, for the deeper failure surface of the 25 year thaw depth, the stability must depend more on the friction ($\emptyset$) than the cohesion. For Borehole N 77-66, as $\emptyset$ is assumed to be only 6°, the static safety factor is less than 1.0 and the slope is considered unstable.

(7) According to the liquefaction analyses, most of the pipeline route segment between MP 670.7 and MP 671.4 would liquefy at some depth during the design earthquake, or even at much lower magnitude earthquake. For example, at the location of Borehole N3-05, total failure would occur below a depth of 10 feet at an acceleration of only six percent if the groundwater table is at the ground surface.

(8) Depending on the amount of slope displacement that can be tolerated during an earthquake, both the thaw plug and unfrozen slope analyses showed that under most conditions, the steepest slope (30%) in the pipeline segment between MP 670.7 and MP 671.4 would not fail during an earthquake. The only failure predicted is for the unfrozen slope when the water table is at the ground surface. However, for the water table located midway between the ground surface and the failure plane, only a small displacement is predicted. By comparison, the liquefaction analysis indicates total soil liquefaction even when the water table is assumed at the base of the failure zone. If liquefaction occurs on even a modest slope, surely a complete slope failure should occur. This is contrary to the results of the slope stability analyses.

3.0 LIST OF QUESTIONS

(1) In liquefaction analysis, NWA estimates the strains for a safety factor of 1.0. What is the significance of these strains? NWA has not said what is to be done when strains are particularly large (for examples 20% or 30%).

(2) In one particular instance the liquefaction analysis showed that the soils on a slope would liquefy. Yet the dynamic stability analysis indicated that the slope would have practically no movement during the design earthquake. How does NWA account for such conditions?

(3) NWA does not describe the procedure for determining the value of saturated density used in the liquefaction analysis. A single value is used for the entire hole. Is it assumed that the analysis is not sensitive to variations in density?

(4) It appears that some incompatibility exists in the parameters used in the slope stability analyses. For example, NWA has given a dry density of 60 PCF and a moisture content of 125% for the "average" soil conditions of the segment studied between MP 205.98 and MP 207.47. The saturated density computed using these values is 135 PCF. Such a value appears unrealistically high for such high ice content soils. What are the criteria for selecting compatible dry densities and moisture contents?

(5) It appears that there are various definitions of thaw plug stability. Is NWA considering thaw plug stability in both the longitudinal and transverse directions of the pipeline?

(6) For liquefaction analysis between MP 670.7 and MP 671.4, a saturated density of 110 PCF was assumed for all boreholes, except Borehole N6-08, where the saturated density was assumed to be 117 PCF. What is the reason for selecting a higher density for Borehole N6-08, when the SPT blow counts were lower than those in other boreholes? See blow counts of Borehole N6-07. It is also not clear if the use of a 2.5 in. Split Spoon and a 340 lb. hammer results in blow counts equivalent to the Standard Split Spoon Test (SPT) or if a correction factor is used.

4.0 TERRAIN STABILITY STUDY - ALIGNMENT SHEET 37

4.1 TERRAIN AND SOIL CONDITIONS

The gas pipeline route shown on Alignment Sheet 37 follows the west slope of the Dietrich River Valley, approximately 300 feet upslope from the Trans Alaska Pipeline (TAPS) right-of-way. The gas line is to be located immediately upslope from the Prudhoe Bay haul road. Slopes in the direction of the pipeline do not exceed about 4-1/2 percent; however, cross slopes are relatively steep, reaching up to 29 percent.

The pipeline is to be located in terrain unit F_s consisting of retransported, frozen organic silt, containing massive ice. The F_s overlies the terrain unit G_t, consisting of frozen mixed grained till with low to moderate ice content. The till overlies sedimentary bedrock. The F_s (silt) deposit is between 0 and 6 feet thick, although the deposit is locally thicker. Ice contents vary from 10 to 40 percent, although higher ice contents are reported in one of the boreholes (N77-67).

In the study area a total of 16 boreholes were drilled either on or close to the right-of-way by Alyeska and NWA. Most of the Alyeska boreholes are too shallow and the soil descriptions are too general to be of much benefit in assessing the soil conditions. Consequently, the terrain conditions were interpreted primarily from the four boreholes drilled by NWA in this segment.

Drawing B-1, in Appendix B, show the topography of the area studied and the location of the boreholes. A geological/soil profile along the centerline of the pipeline is presented on Drawing B-2.

4.2 LIQUEFACTION ANALYSIS

The slope is situated in an area of magnitude 5.5 earthquake, for which the design peak acceleration is 0.15. The NWA liquefaction analysis procedure consists of checking the liquefaction potential at each borehole where thawing is expected. Liquefaction analyses were performed for Boreholes N0031-046 and N77-66. The liquefaction potential is evaluated by computing the cyclic stress ratio and modified penetration resistance using the following relationships:

$$\tau/\sigma_o' = 0.65 \frac{a_{max}}{g} \cdot \frac{\sigma_o}{\sigma_o'} \cdot r_d \qquad \text{(Equation 1)}$$

$$N_1 = C_N . N \qquad \text{(Equation 2)}$$

Where:
- τ = Average horizontal shear stress induced by earthquake
- σ_o' = Effective overburden pressure on the soil layer involved
- σ_o = Total overburden pressure on the soil layer involved
- a_{max} = Maximum earthquake acceleration
- g = Acceleration due to gravity
- r_d = A stress reduction factor
- C_N = Correction factor for overburden pressure
- N = Standard penetration resistance (blows per foot)
- N_1 = Modified penetration resistance (blows per foot).

The modified penetration resistances for frozen soils are obtained from graphs presenting the relationships between frozen dry densities and corrected blow counts for various soils.

For liquefaction analysis at Borehole N0031-046, NWA Figures 5-65 and 5-66 were used to determine the modified penetration resistance while Figure 5-60 was used to determine the critical modified penetration resistance (N_1) and the critical acceleration for 0 percent, 5 percent, 10 percent, and 20 percent shear strain and total failure. These figures are presented in Appendix A of this report.

The results of the liquefaction analysis at Borehole N0031-046 are presented on Table B-1 in Appendix B. The agreement between NWA results and the TSC results are generally good. There are differences in the factors of safety, which is defined as the ratio of N_1 of the soil over N_1 critical, but these differences are believed to be due to computer roundoff.

Table B-2, in Appendix B, presents the liquefaction analysis at Borehole N77-66. This borehole was not analyzed by NWA. In addition to the figures used in the analysis of the first borehole, Figure 5-63 and 5-68 were also used. For the six foot depth sample the density was relatively low (72 PCF) and the corrected blow count could not be found using NWA figures. The corrected blow count was assumed to be five, which indicate liquefaction potential.

An attempt was made to analyze Borehole N77-66 for liquefaction potential but because of the low density soils NWA figures could not be used. If these soils are allowed to thaw they will liquefy under static conditions.

4.3 THAW PLUG STABILITY

As the ground is frozen in this segment, thaw plug stability is critical. Cross slopes are much steeper than the longitudinal slopes and hence will control stability. Of the four holes drilled by NWA, Borehole N77-67 displays ice contents that are well above the average ice content determined from the "average" conditions. See Drawing B-2 for a geological/soil profile along the pipeline centerline.

4.3.1 Thaw Plug Stability at Borehole N77-67

To permit a comparison between thaw plug stability calculations based on average and extreme soil conditions, stability analysis was performed for the slope at the location of Borehole N77-67. The NWA analytical procedure was used, incorporating soil parameters based on the log of Borehole N77-67. For the analysis, the following soil parameters were adopted:

- Moisture content of 100 percent
- Dry density of 50 PCF
- Saturated density of 100 PCF

The soil is interpreted to be organic silt, with some sand and ice. This soil corresponds to Group 4 (Table 5-1). The average cross slope at the hole location is 21 percent (11.8°) and the longitudinal slope is 0.1 percent.

From Table 4-2, undrained strength parameters are indicated as follows:

$$\phi_{cu} = 6°$$
$$C_{cu} = 100 \text{ PSF}$$

The predicted depth of thaw is determined from Figure 5-8. For a work pad of two feet thick. The depth of thaw would be one foot after two years and seven feet after 25 years. In the graded or disturbed portion of the right-of-way where there is no work pad, the depth of thaw would be 2.7 feet after two years and 10.5 feet after 25 years. The thaw consolidation ratio (R parameter) is assumed to be 0.2 after two years and 0 after 25 years.

4.3.2 Thaw Plug Stability After Two Years

(1) Within the Work Pad Area

From Figure 5-19 for one foot of thaw and a two foot thick work pad the slope angle for $\beta_{SFS=1.0}$ is 28°

From Figure 5-21 when the soil friction angle is 6° the corrected slope angle for $\beta_{SFS=1.0}$ is 4°

From Figure 5-20 the cohesion component R_2 is 0.72

The corrected R_2 to account for a cohesion different from the value

used in Figure 5-20 is $0.72 \times \dfrac{\text{Design Cohesion}}{250} = \dfrac{.72 \times 100}{250} = 0.29$

The Static Factor of Safety (SFS) is computed as follow:

$$SFS = \frac{\tan \beta_{SFS=1.0\ (C=0)}}{\tan\beta} + \frac{R_2}{\sin\beta} \qquad \text{(NWA Equation 5.1.19)}$$

where: β is the slope angle (11.8°)

$$SFS = \frac{\tan 4°}{0.21} + \frac{0.29}{0.203}$$

$$= 0.33 + 1.43 = 1.76$$

The Dynamic Factor of Safety (DFS) is computed as follow:

$$DFS = (SFS) \frac{\tan^{\beta}}{\tan\beta + K}$$ (NWA Equation 5.1.26)

$$DFS = (1.76) \frac{0.21}{0.21 + 0.15} = 1.02$$

(2) Disturbed Zone Outside the Work Pad

Using the same figures and equations as above, the following safety factors were determined:

SFS = 4.9
DFS = 2.8

4.3.3 Thaw Plug Stability After 25 Years

(1) Within The Work Pad Area

From Figure 5-17 for a thaw depth of seven feet and a two foot thick work pad the slope angle for $\beta_{SFS=1.0}$ is 20.5°

From Figure 5-21 the corrected slope angle for $\beta_{SFS=1.0}$ is 3°

From Figure 5-20 the cohesion component R_2 is 0.3

The corrected R_2 to account for a cohesion different from the value used to construct Figure 5-20 is 0.12

$$SFS = \frac{\tan \beta_{SFS=1.0(C=0)}}{\tan\beta} + \frac{R_2}{\sin\beta}$$

$$= 0.21 + 0.59$$
$$= 0.8 \text{ (unstable)}$$

(2) Disturbed Zone Outside the Work Pad

SFS = 1.04

DFS = 0.6

When the dynamic factor of safety is less than 1.0, the slope displacement must be computed. The displacement of a slope during an earthquake is determined from the following equation:

$$d = \frac{V^2}{2gN}\left(1-\frac{N}{A}\right)\frac{A}{N} \qquad \text{(Equation 3)}$$

where: d = slope displacement

V = maximum ground velocity

N = yield acceleration expressed as a ratio of g

g = acceleraton due to gravity

A = maximum ground acceleration expressed as a ratio of g.

For the particular case analyzed, A = 0.15 and V = 7 in/sec.

$$N = (SFS)\ \frac{\tan\beta}{\tan\beta + K} = (1.04-1)\ \text{Sin}\ 11.8° \ \text{Cos}\ 6° = 0.008$$

$$d = \frac{7^2}{2\ (386)(.008)}\left(1- \frac{.008}{.15}\right)\left(\frac{.15}{.008}\right) = 140.8 \text{ inches}$$

4.4 THAW PLUG STABILITY USING GENERALIZED SOIL CONDITIONS

Thaw plug stability analyses were carried out using the NWA procedure and the computer program Slope II. A transverse slope with gradients of 24 and 27 percent (MP 206.35) was selected for the analysis. The water table was assumed at the surface and the thickness of the work pad was taken as zero.

The "average" soil conditions consist of six feet of silt (ML) underlain by 10 feet of silty gravel (ML - GM). The silty gravel is underlain by schist bedrock. From Table 4-2, a friction angle of 8° and a cohesion of 150 PSF were obtained for the silt (dry density 60 PCF). For the silty gravel(dry density of 90 PCF) a friction angle of 18° and a cohesion of 350 PSF were obtained. Drawing B-5, in Appendix B, illustrates the soil profile and presents the soil parameters and the results of the analyses.

The stability of the 24 percent (13.5°) slope with six feet of thaw was computed as follows:

From Figure 5-16, for 6 foot of thaw and zero work pad the slope angle for $\beta_{SFS=1.0}$ is 19.7°

From Figure 5-21,for a soil friction angle of 6° the corrected slope angle for $\beta_{SFS=1.0}$ is 3.6°

The cohesion component $R_2 = \dfrac{C(W+2D)}{WD\gamma} + \dfrac{150 \times (100+12)}{100 \times 6 \times 125} = .224$

$$SFS = \frac{\tan \beta_{SFS=1.0\ (C=0)}}{\tan\beta} + \frac{R2}{\sin\beta}$$

$$= \frac{\tan 3.6}{.24} + \frac{.224}{\sin\beta}$$

$$= \frac{.062}{.24} + \frac{.224}{.23} = 1.2$$

$$DFS = (SFS) \frac{\tan\beta}{\tan\beta + k} = (1.2) \frac{x.24}{.39} = 0.74$$

The slope displacement is computed using the following values:

$$N = (SFS - 1) \text{ Sin}^{\beta} \text{ Cos } \Phi' = (1.2 - 1) \text{ Sin } 13.5° \text{ Cos } 8°$$

$N = .046$

$V = 7$ in/sec.

$A = .15$ g

$$d = \frac{V^2}{2gN} (1- \frac{N}{A}) \frac{A}{N} = \frac{7^2(1 - .31)}{2 \times (386)} \frac{\times .15}{.046} = 1.7 \text{ in.}$$

The static and dynamic stability of the 27 percent slope was also calculated using the same method and the same thaw depth. The slope stability analysis results are presented on Drawing B-5. The factors of safety obtained using the NWA method (using the figures) are slightly less than the values obtained with the computer.

The slope stability results for the Alignment Sheet 37 are summarized below:

CONDITIONS	ELAPSE TIME	SFS	DSF	DISPLACEMENT
Borehole N77-67 (within work pad area, water table at surface)	2 years	1.7	1.0	--
	25 years	0.8	--	failure
Borehole N77-67 (outside work pad area, water table at surface)	2 years	4.9	2.8	140 in.
	25 years	1.04	0.6	

CONDITIONS	ELAPSE TIME	SFS	DSF	DISPLACEMENT
"Average soil conditions for 6 ft. thaw depth (water table at surface)	--	1.2	0.7	1.7 in.

5.0 TERRAIN STABILITY STUDY - ALIGNMENT SHEET 119

5.1 TERRAIN AND SOIL CONDITIONS

In the study area the gas pipeline route is located adjacent to the southwest side of the Haines right-of-way (ROW) near the bottom of a broad valley. The overall slope is across the pipeline from southwest to northeast. The alignment crosses gentle to moderate slopes which are both transverse and parallel to the pipeline. The steepest slopes are approximately 30 percent in the direction of the pipeline, and 15 percent transverse to the pipeline.

Drawing B-3, in Appendix B, shows the topography of the area studied and the location of the boreholes. A geological/soil profile, along the centerline of the pipeline between MP 670.7 and 671.4, is presented on Drawing B-4. This portion of the pipeline route segment contains the critical slope considered in the stability analysis.

The predominant terrain unit, in which all of the boreholes were drilled, is eolian sand (E_s). In addition, granite bedrock outcrops at the north end of the segment and sandy retransported deposits (F_{sa}) are encountered at the south end. In one of the boreholes (N6-7), silt was encountered below the depth of 33.5 feet.

The eolian sand is fine and contains a trace of silt. The soil is unfrozen from the surface to a depth ranging from 14.5 feet to more than 50 feet. No visible ice is reported in the frozen soil portions of the boreholes. Frozen dry densities average about 98 PCF.

Groundwater levels were not presented for any of the boreholes. However, in the adjacent segment to the south, which is located in similar sandy material (F_{sa} + E_s), the groundwater table was recorded in two boreholes at a depth of less than five feet from the ground surface.

Most of the borings shown on Alignment Sheet 119 encountered unfrozen, or mixed frozen and unfrozen soils. The borings appear to have been located within the Haines ROW. As the gas pipeline may be positioned off the right-of-way in what is presently undisturbed terrain, thermal conditions at the actual pipeline location may be significantly different from what is shown in the borings located within the Haines ROW. Significantly more permafrost may exist in the presently undisturbed terrain where the pipeline would be buried. From the standpoint of liquefaction analyses, conditions interpreted from the borings along the Haines ROW would be conservative because the depth of unfrozen ground, which is potentially subject to liquefaction, is conservative. On the other hand, the extent of frozen ground that would be subjected to thaw plug instability might be underestimated.

5.2 LIQUEFACTION ANALYSIS

The area is situated in a magnitude 7.0 earthquake zone, for which the design peak acceleration is 0.35. A saturated density of 110 PCF was assumed by NWA for all of the boreholes, except Borehole N6-8, where the saturated density was assumed to be 117 PCF.

Liquefaction analysis was performed on Borehole N3-05 which is unfrozen in the upper 21 feet. The procedure adopted in the liquefaction analysis for unfrozen soil is illustrated here for the 10 foot depth sample (Sample No. 3) of Borehole N3-05.

Calculation of Modified Blow Count

The modified blow count (N_1) is the equivalent SPT value (N) for an effective overburden pressure (σ_0') equal to 1 TSF. N_1 is calculated from Equation 2 presented in Section 4.2 of this report. The relationship between C_N and σ_0' is shown on Figure 5-62.

In Borehole N3-05, at a depth of 10 feet, σ_0' is 1100 PSF for the groundwater table (GWT) at 10 feet and 476 PSF for the GWT at the ground surface (0 feet). From Figure 5-62, C_N = 1.3 (GWT - 10 feet) and 1.6 (GWT - 0 feet). Hence, for the reported field blow count of 6, the value of N_1 would be 7.8 and 9.6 for the two groundwater tables considered. For conservatism, the lower value is selected and rounded off to N_1 = 8.

Critical N_1 for Various Strains

To calculate the critical N_1, the cyclic stress ratio (CSR) must first be calculated according to Equation 1 presented in Section 4.2 of this report. Figure 5-59 shows the relationship between the CSR causing various strains and N_1 for earthquake magnitude 7.0. Figure 5-61 is a plot of r_d versus depth. For the shallow depths potentially involved in liquefaction of the pipeline, the value of r_d is close to 1.0 and does not have a significant effect on the calculation of CSR.

In Borehole N3-05 at a depth of 10 feet, the CSR values are 0.22 (GWT - 10 feet) and 0.52 (GWT - 0). The critical N_1 values determined from Figure 5-59 are as shown below:

	Critical N_1 For Strains Shown				
	0%	5%	10%	20%	TF
GWT - 10 feet	18	18	17	16	11
GWT - 0 feet	37	32	25	17	11

Since the actual predicted N_1 of 8 is less than the critical N_1 at total failure, the soil deposit would liquefy under the design earthquake.

Critical Accelerations for Various Strains

The maximum values of acceleration ($\frac{a_{max}}{g}$) that the soil deposit can undergo before exceeding the specified levels of strain are also calculated from Figure 5-59. For a N_1 of 8, and the CSR values calculated above, the critical accelerations for all levels of strain are 0.15 for GWT = 10 feet and 0.06 for GWT = 0. Thus, the soil would liquefy at a relatively small acceleration if the groundwater is at the surface.

Critical Groundwater Table for Various Strains

No data is given by NWA presumably because liquefaction is predicted for all groundwater levels.

Factor of Safety for Various Strains

The factor of safety for each level of strain is calculated by taking the ratio of N_1 to the critical N_1 for each strain. The factors of safety calculated are as follows:

	Factor of Safety for Strains Shown				
	0%	5%	10%	20%	TF
GWT - 10 feet	0.4	0.4	0.5	0.5	0.7
GWT - 0 feet	0.2	0.25	0.3	0.5	0.7

Estimated Strain

The strain is estimated by using Figure 5-59 and interpolating the strain for a factor of safety of 1.0. Since the sand deposit at a depth of 10 feet in Borehole N3-05 is predicted to liquefy, no entry is made in the estimated strain column. However, in the liquefaction analysis made by NWA at a depth of five feet, a strain of 32 percent was obtained. The derivation of such strain is not clear.

Comparison of NWA and TSC results using the NWA method of liquefaction analysis is presented on Table B-3, in Appendix B. The TSC computations agree well with NWA computations.

5.3 THAW PLUG STABILITY

The soil type is sand, with a trace of silt, corresponding to terrain type Es and to the soil classification SP. The sand contains eight percent passing the 200 mesh sieve. The deposit is considered uniform to a depth of 50 feet. The dry density is assumed to be 100 PCF for frozen conditions and 93 PCF for unfrozen conditions.

The saturated bulk density is 125 PCF. The frozen soil is classified as Group 1, which would have thermal properties specified in Table 5-1. The shear strength parameter are as follows:

Undrained Strength:	Φ_{cu} =	22°
	C_{cu} =	430 PSF
Drained Strength:	Φ' =	39.8°
	C' =	0

The above drained parameters do not match the values presented in Table 4-2. According to this table, the friction angle (Φ') would be 37° and the cohesion (C') would be 270 PSF.

In the study area (MP 670.7 to 671.4) the pipeline is located on, or adjacent to the Haines right-of-way. The cross slope gradients range between 5 and 25 percent with slopes dipping to the southwest. The proposed one foot thick work pad is on the right side of the pipe centerline. The maximum longitudinal slope in the segment is 30 percent.

The NWA estimated depth of thaw is 5.9 feet after two years and 21.4 feet after 25 years. These values were checked by using Figure 5-5, which shows the predicted thaw depth versus mile post (MP) for soil Group 1. According to Figure 5-5, the depth of thaw would be 5.0 feet after two years and 18.1 feet after 25 years. Hence, the values selected by NWA appear to be conservative.

The R-factor, which is understood to be the thaw consolidation parameter used in the prediction of excess pore pressure at the thaw front, is shown as zero.

This is reasonable, as the sand contains only a trace of silt, and would be expected to quickly dissipate any excess pore pressures associated with the thawing of segregated ice in the soil.

The stability analysis was performed for a slope of 30 percent. Analyses were conducted for two groundwater table (GWT) factors: 0.5 and 1.0. A GWT factor of 0.5 is understood to represent a GWT at the mid-point between the failure plane and the ground surface. A GWT factor of 1.0 corresponds to a GWT at the ground surface (critical conditions). The procedure presented by NWA for preliminary terrain stability analyses was followed to determine the factors of safety.

5.3.1 Thaw Plug Stability After 25 Years

As the soil is essentially homogeneous, the failure surface is assumed to correspond to the 25 year thaw depth. Drained (effective stress) strength parameters are applicable. The static factor of safety (SFS) and the dynamic factor of safety (DFS) are calculated as follows:

(1) Static Stability

- GWT = 0.5 (groundwater table at mid-point between the failure plane and ground surface)

  - From Figure 5-16, when the work pad is one foot thick and the thaw depth is 21.4 ft $\beta_{SFS=1.0}$ is 30.5°,

  - Figure 5-16 is developed for Φ = 35° and the β value has to be corrected for the actual Φ of 39.8° which was used in the analysis.

  - From Figure 5-21, $\beta_{SFS=1.0}$ is 34.7°

- The SFS is then calculated for a 30 percent grade from the equation.

$$SFS = \frac{\tan 35.3}{0.30} = 2.3$$

- This is higher than the SFS of 1.96 computed by NWA.

- GWT = 1.0

  - From Figure 5-16 $\beta_{SFS=1.0} = 21.3°$. From Figure 5-21, $\beta_{SFS=1.0}$ is 24.5

  - $SFS = \frac{\tan 24.5°}{0.30} = 1.52$

  This is close to the SFS of 1.55 obtained by NWA.

(2) Dynamic Stability

For well drained soils, the dynamic factor of safety is calculated using the following equation:

$$DFS = (SFS)\ X_d - Y \qquad \text{(NWA Equation 5.1.22)}$$

Figure 5-29 is used to determine X_d and Y. From Figure 5-29, $X_d = 0.47$ and $Y/\tan \Phi = 0.17$, for which $Y = 0.14$ and $X_d - Y = 0.33$.

Hence, DFS = 2.31 (0.33) = 0.76 for GWT = 0.5
and 1.52 (0.33) = 0.50 for GWT = 1.0.

By comparison NWA calculated DFS values of 0.63 and 0.47 for GWT = 0.5 and 1.0, respectively. The value of (X_d - Y) used was very similar, being approximately 0.32.

Because the DFS calculated is less than 1.0, it is necessary to estimate the slope displacement that would occur during an earthquake. The displacement of a slope during an earthquake is determined from Equation 3 presented in Section 4.3.3 of this report.

For the slope under consideration (GWT = 0.5) the values are:

N/A = 0.29/.35 = 0.83
A/N = 1.21
V = 17 in/sec.
g = 386 in/sec.^{2}

$$d = \frac{(17)^2}{2(386)(0.29)} (1 - 0.83)(1.21) = 0.3 \text{ inch}$$

NWA obtained a displacement of 0.6 inches for an acceleration of 0A

For the same slope but with a GWT at the ground surface (GWT = 1.0) the values are:

N/A = 0.314 ; A/N = 3.18

$$d = \frac{(17)^2}{2(386)(0.20)} (1-0.314)\ (3.18) = 7.4 \text{ inches}$$

d = 7.4 inches (same as NWA)

5.3.2 Thaw Plug Stability After Two Years

The same procedure is followed in calculating the short term stability except that the failure occurs at a much shallower depth, (5.9 feet). It is assumed that the GWT is at the ground surface (GWT = 1).

From Figure 5-16, for a 5-9 ft thaw depth and a one foot thick work pad $\beta_{SFS=1.0}$ is 21.8°

From Figure 5-21, for a soil with a fraction angle of 39.8° $\beta_{SFS=1.0}$ is 25.0

$$SFS = \frac{\tan 25.0°}{0.30} = 1.55 \text{ (same as NWA)}$$

$$DFS = (SFS)\, X_d - Y$$

$$\text{as } X_d - Y = 0.33$$

$$DFS = (1.55)(0.33) = 0.51 \text{ (NWA value = 0.50).}$$

5.4 UNFROZEN SLOPE STABILITY

The procedure followed in the calculation of unfrozen slope stability is assumed to be identical to that for the thaw plug, except that the geometry of the surface is not controlled by the dimensions of the thaw zone. Slightly higher values of undrained and drained strength parameters were assumed and presumably correspond to an unfrozen soil having an unfrozen dry density of 100 PCF. According to Table 4-2, the following parameters were selected. The NWA parameters are shown for comparison in parenthesis.

Undrained strength: Φ_{cu} = 36° (23)
C_{cu} = 330 (450)

Drained strength: Φ' = 36° (40)
C_{cu} = 220 (0)

The parameters selected by NWA do not correspond to those presented in Table 4-2.

Calculation of Stability

Stability computations were made for three GWT factors: 0, 0.5 and 1.0. The critical case is for the GWT at the ground surface. The infinite slope stability results are summarized below:

GWT	SFS	DFS
1.0	1.3	0.4
0.5	2.0	0.7
0	2.7	-

The slope displacement under dynamic conditions (GWT = 1.0) was calculated using NWA method. A value of 20 inches was obtained. For a groundwater factor of 0.5 the slope displacement is 0.7 inch. NWA indicated that the displacement for this slope (GWT = 1.0) was greater than 12 inches.

The stability analyses for the unfrozen slope and the long term (25 year) thaw plug should have given the same results because the critical surface in both cases is a shallow infinite slope failure with the groundwater table at the ground surface. The thaw plug analysis gave a higher SFS (1.5 versus 1.4) because the failure surface for the thaw plug was assumed along the 25 year thaw surface which is not the critical geometry.

The slope stability results for Alignment Sheet 119 are summarized below:

CONDITIONS	ELAPSED TIME	SFS	DFS	DISPLACEMENT
"Average" soil conditions water table at surface	2 years	1.5	0.5	7.4 inches
Water table at surface	25 years	1.5	0.5	7.4 inches
Low water table -GWT = 0.5		2.3	0.7	0.4 inch
"Average" soil conditions unfrozen	--			
water table at surface		1.3	0.4	20 inches
Low water table - GWT = 0.5	--	2.0	0.7	0.7 inch
No water table - GWT = 0	--	2.7	--	--

APPENDIX A

-

NWA TABLES AND FIGURES

TABLE 4-2

THAWED* AND UNFROZEN**
SOIL SHEAR STRENGTH PARAMETERS

UNFROZEN

Frozen dry density (γ_{df}) pcf		50	60	70	80	90	100	110	120	130
Undrained (#200>12%)	Øcu	6°	8°	11°	14°	18°	23°	28°	33°	33°
	C_{cu}(psf)	100	150	200	250	350	450	600	700	1400
Drained (#200≤12%)	Ø'	18°	22°	26°	30°	33°	38°	43°	45°	45°
	C' (psf)	120	140	170	200	230	280	330	390	450
Clean sands and gravels (Ø)'		25°	28°	31°	34	37	40	43	45°	46°

THAWED

Unfrozen dry density (γ_{du}) pcf		70	80	90	100	110	120	130	140
Undrained (#200>12%)	Øcu	6°	9°	13°	17°	23°	29°	33°	33°
	C_{cu}(psf)	100	150	230	330	450	610	1200	1900
Drained (#200≤12%)	Ø'	25°	28°	32°	36°	40°	45°	45°	45°
	C' (psf)	120	150	180	220	280	350	420	510
Clean sands and gravels (Ø')		25°	28°	32°	36°	40°	45°	47°	48°

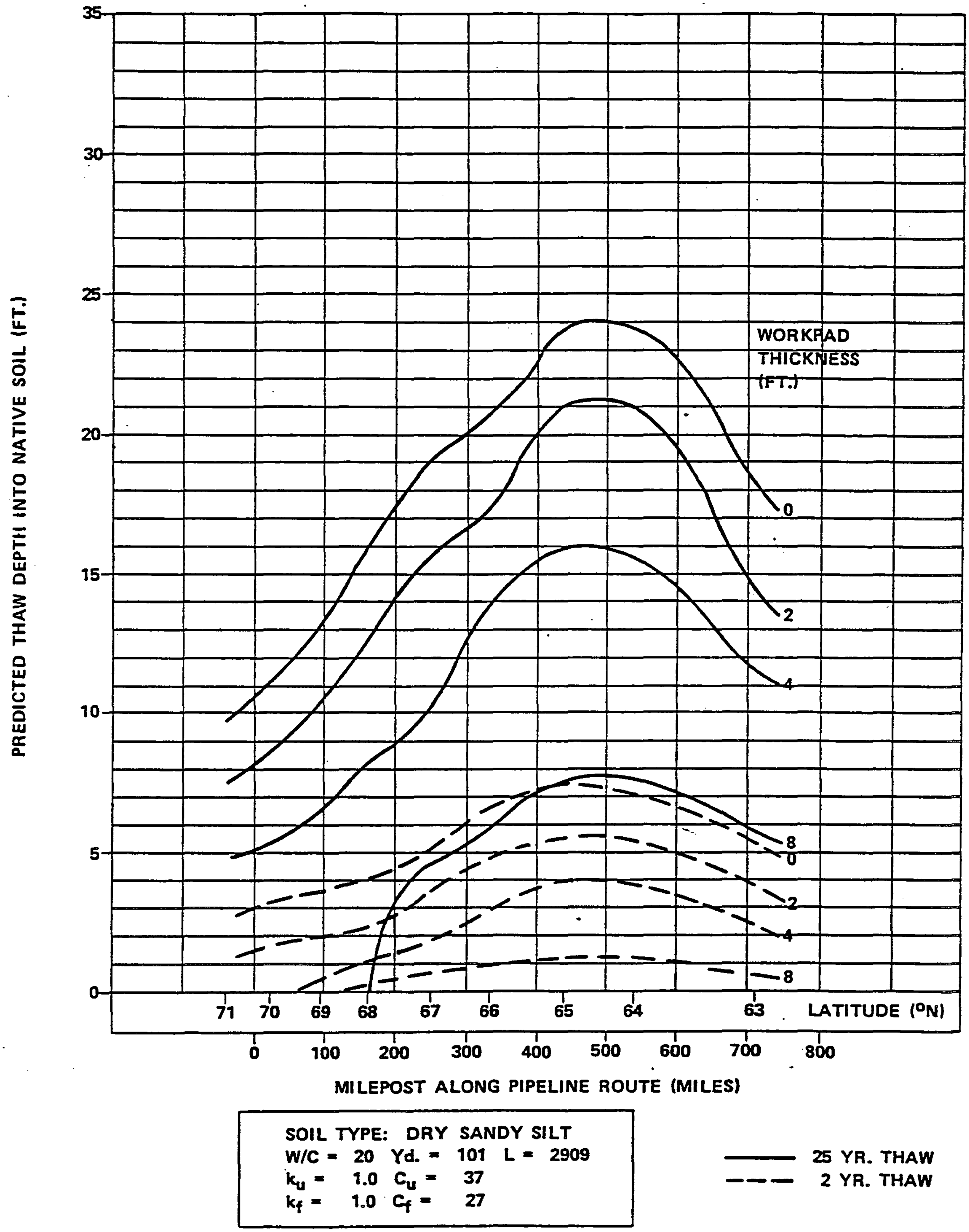
*"Thawed" implies that soils have been previously frozen and have been allowed to thaw prior to testing.

**"Unfrozen" implies that soils have never been frozen.

TABLE 5-1

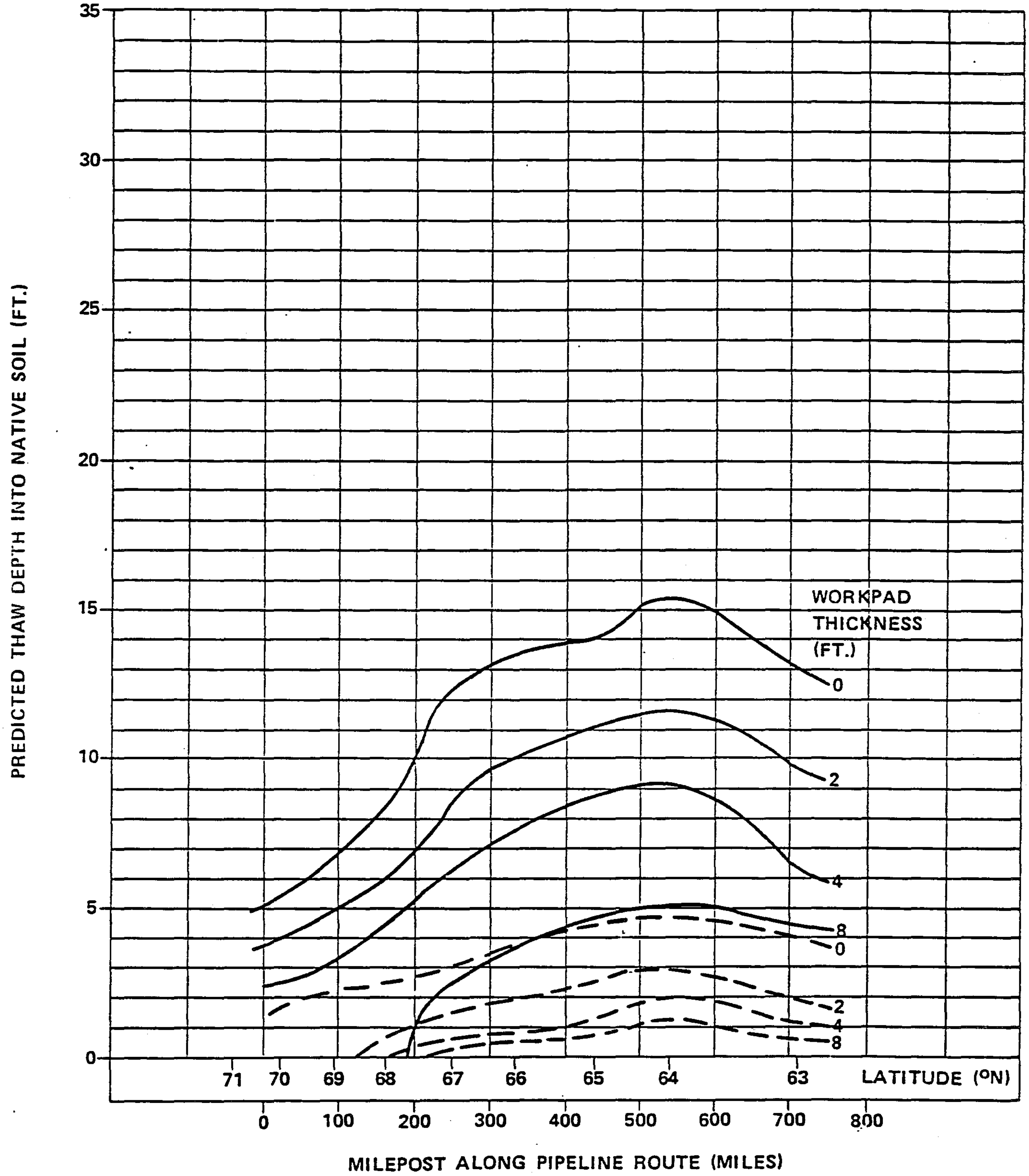
THERMAL PROPERTIES FOR SOIL THERMAL GROUPS
(for use in Thaw Plug Analysis)

SOIL GROUP	W/C WATER CONTENT %	γd DRY DENSITY (PCF)	L LATENT HEAT (BTU/FT^3)	k_f THERMAL CONDUCTIVITY (BTU/FT-HR-°F)	k_u	C_f HEAT CAPACITY (BTU/FT^3-°F)	C_u
1	20	101	2909	1.0	1.0	27	37
2	30	84	3629	0.9	0.8	27	39
3	40	72	4147	1.1	0.6	27	41
4	88	50	6336	1.26	0.4	31	52
5	2	130	374	0.7	1.0	23	25
6	5	120	864	1.0	1.1	23	26
7	10	110	1584	1.12	1.12	24	30
8	15	110	2376	1.8	1.3	27	35
Analysis Gravel Workpad	8	110	1276	1.08	1.0	27	23



PREDICTED THAW DEPTH ALONG ROUTE FOR FLAT GROUND
SOIL GROUP NO. 1

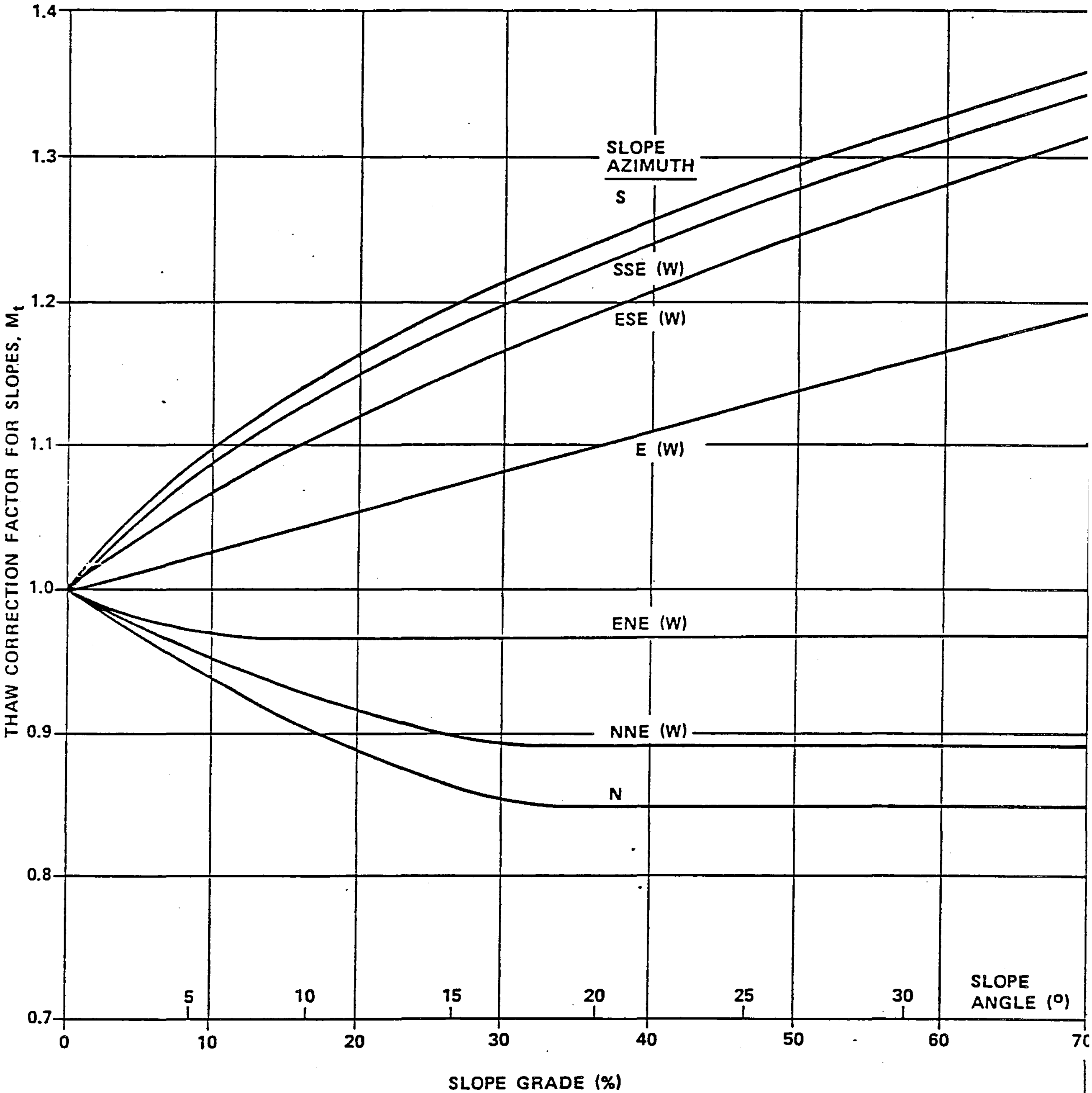
FIG. 5-5



SOIL TYPE:		ICE RICH SILT			
W/C =	88	Yd. =	50	L =	6336
k_u =	0.4	C_u =	52		
k_f =	1.26	C_f =	31		

PREDICTED THAW DEPTH ALONG ROUTE FOR FLAT GROUND
SOIL GROUP NO. 4

FIG. 5-8



THAW DEPTH (CORRECTED) = M_t x THAW DEPTH (FLAT GROUND)

THAW CORRECTION FACTOR FOR SLOPES

FIG. 5-13

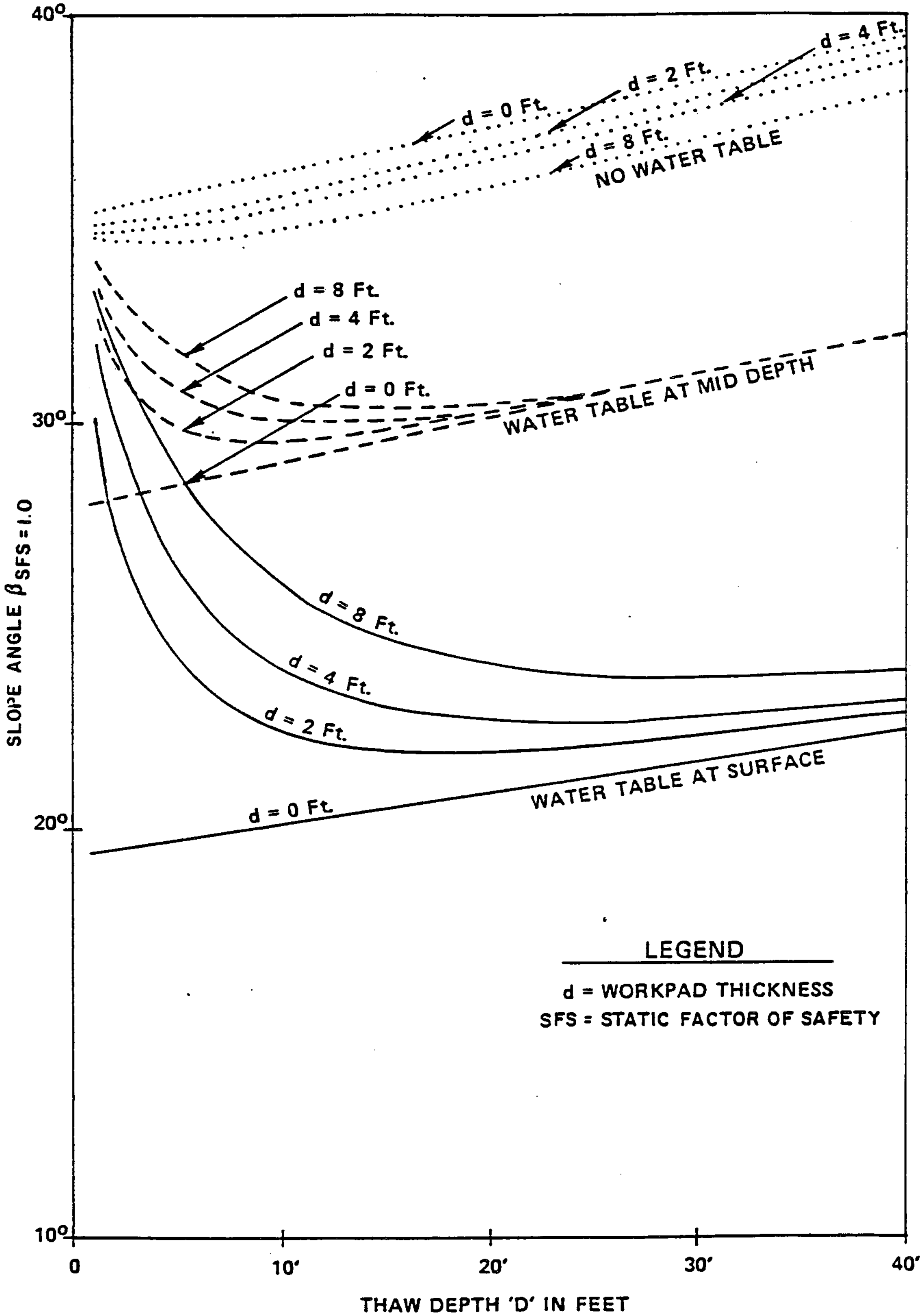


FIG. 5-16 SLOPE ANGLE FOR SFS = 1.0 vs. THAW DEPTH
(K_o = 0.45, ϕ = 35°, γ = 125 pcf, W = 100 Ft., R = 0, C = 0)

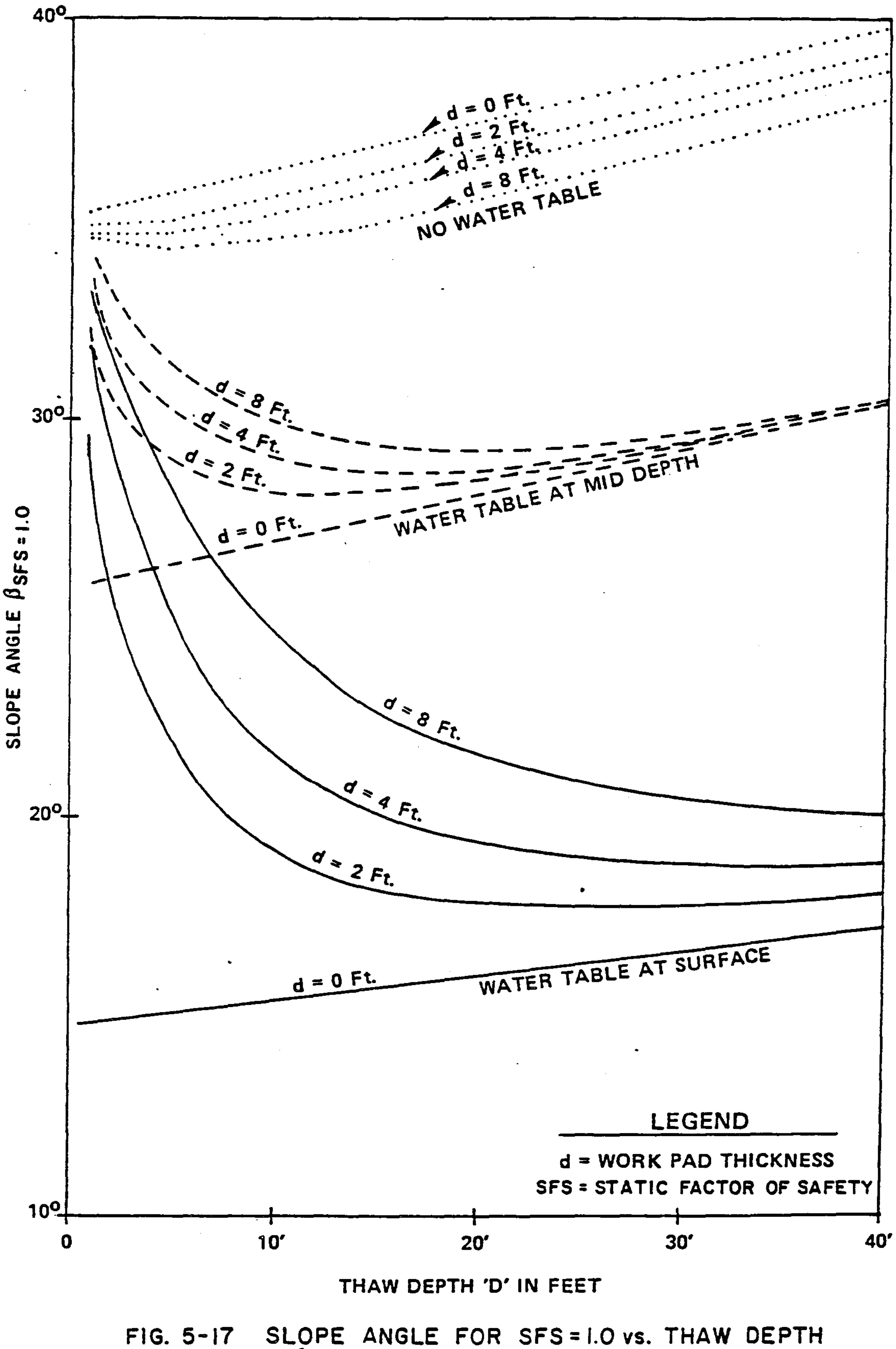


FIG. 5-17 SLOPE ANGLE FOR SFS = 1.0 vs. THAW DEPTH

(K_o = 0.45, ϕ = 35°, γ = 100 pcf, W = 100 Ft., R = 0, C = 0)

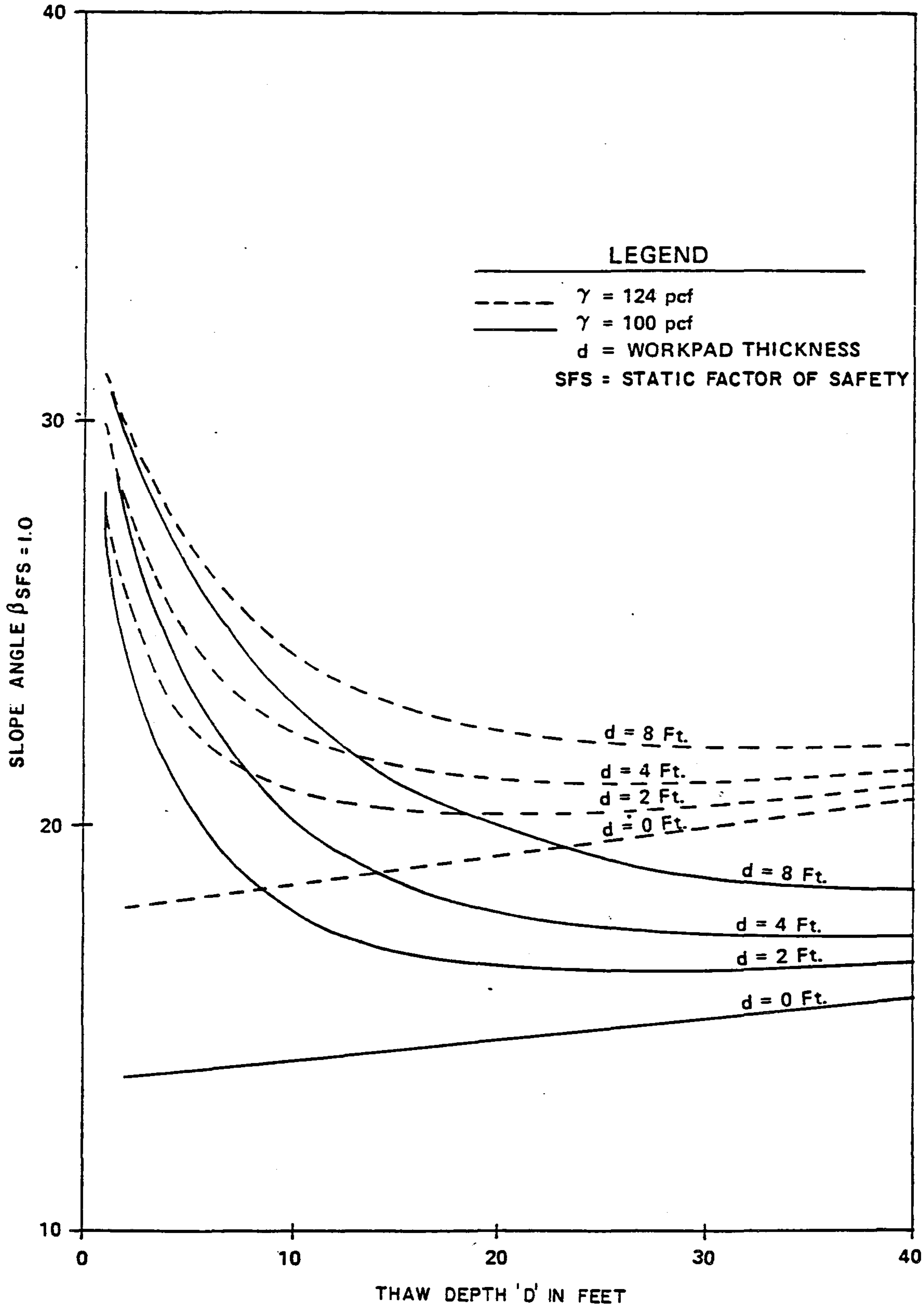


FIG. 5-19 SLOPE ANGLE FOR SFS = 1.0 vs. THAW DEPTH
(K_o = 0.45, ϕ = 35°, W = 100 Ft., R = 0.2, Z = 1.0, γ_p = 125 pcf, C = 0)

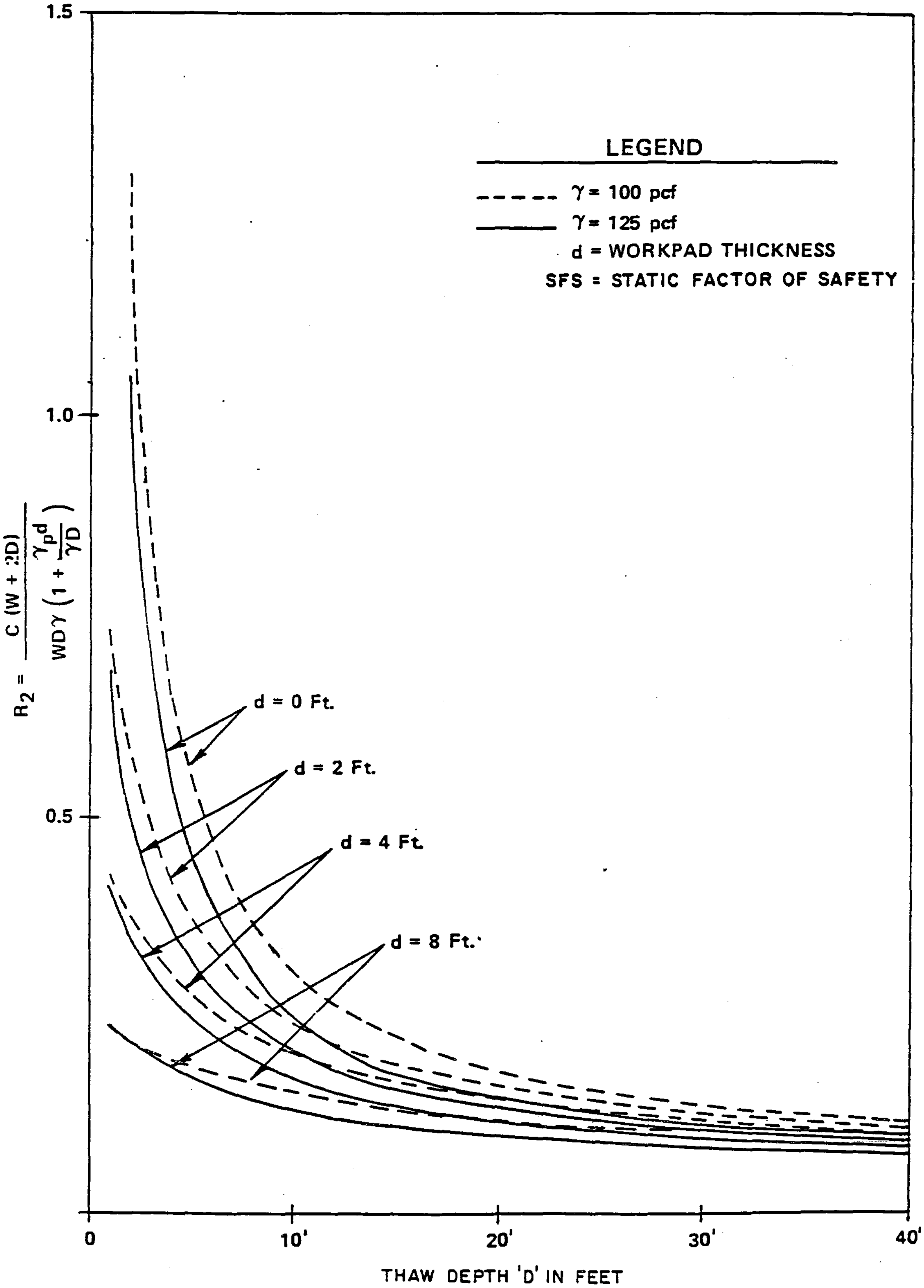


FIG. 5-20 R_2 (COHESION COMPONENT OF SFS) vs. DEPTH OF THAW
(C = 250 psf, γ_p = 125 pcf, W = 100')

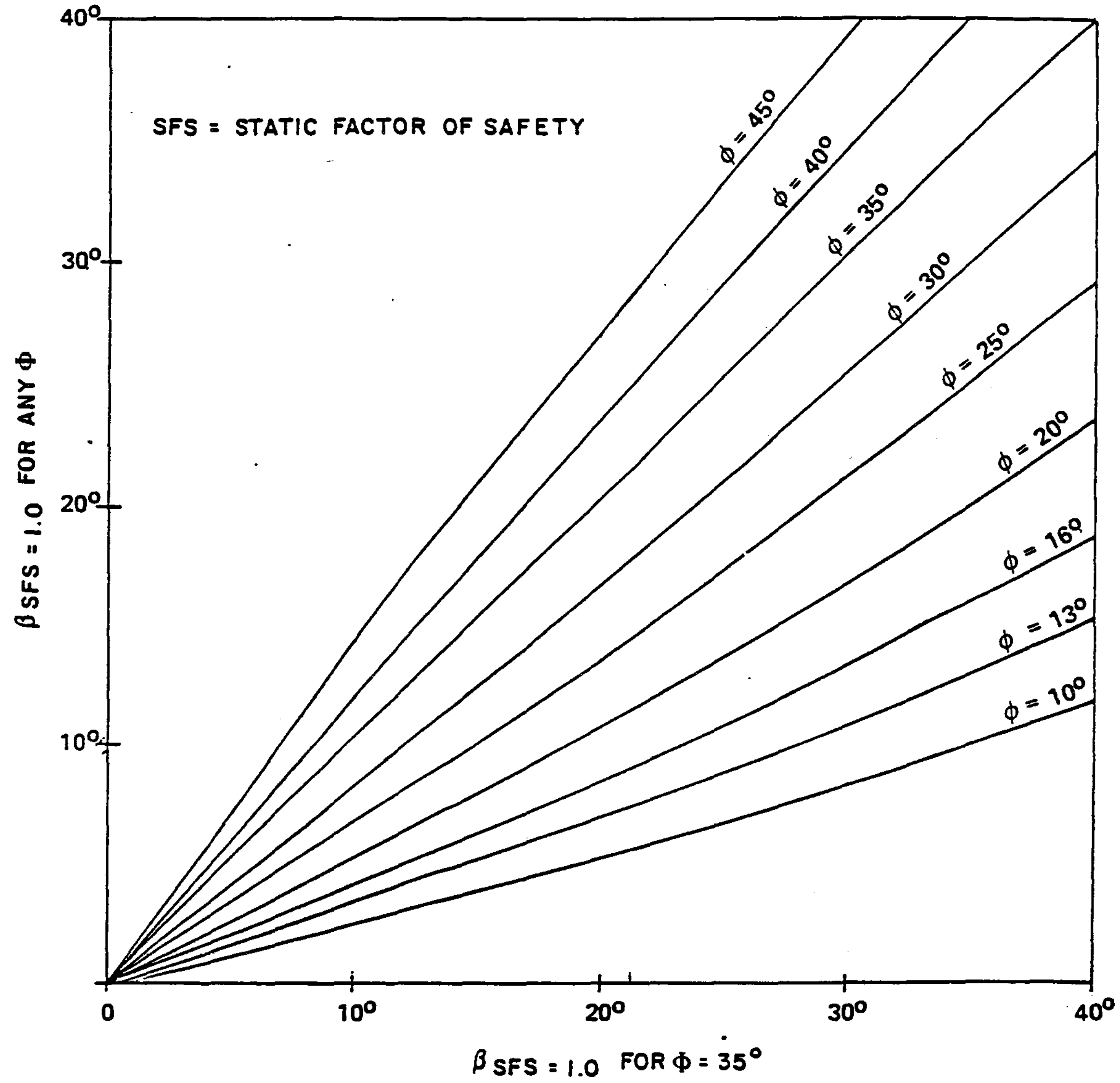
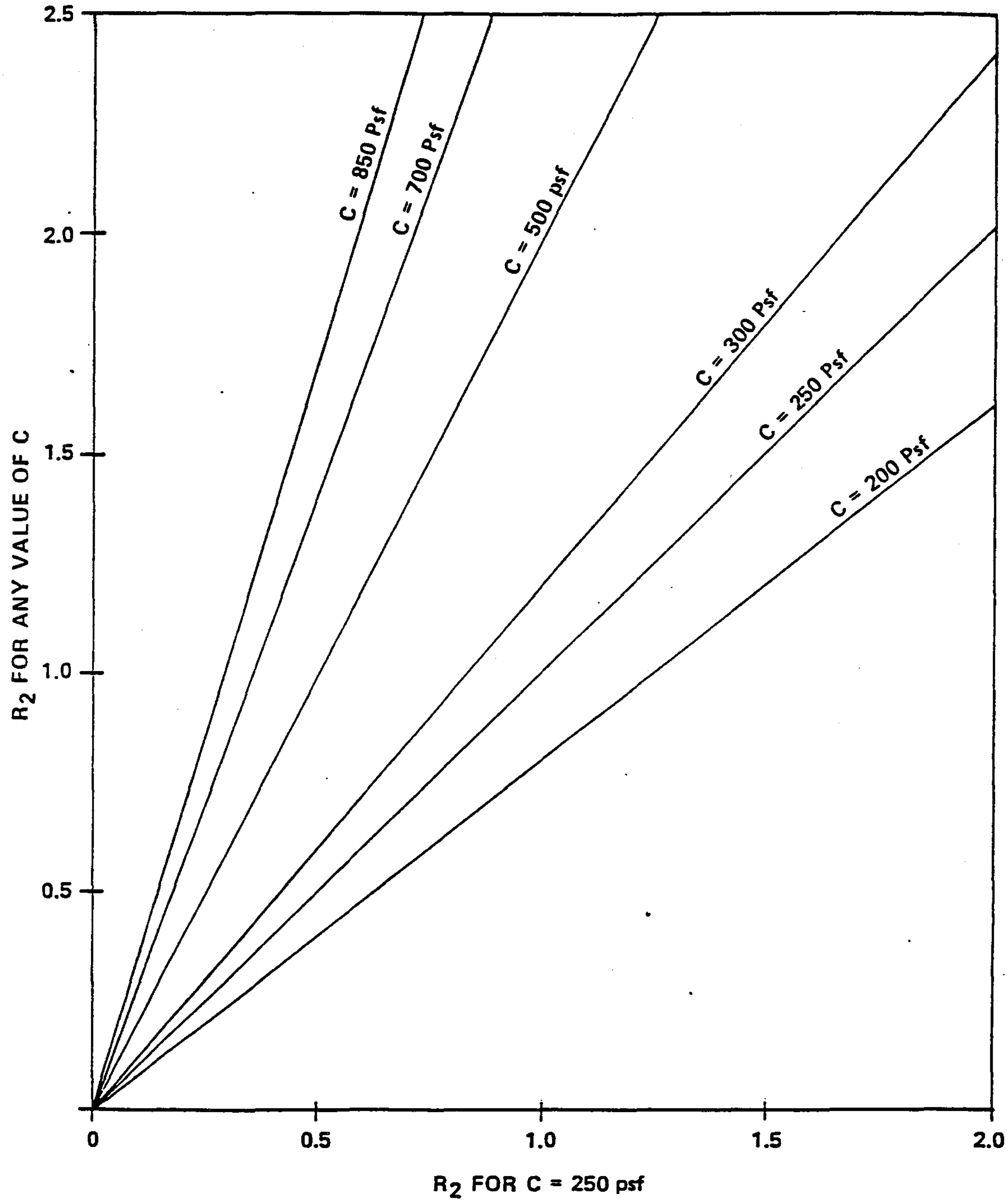
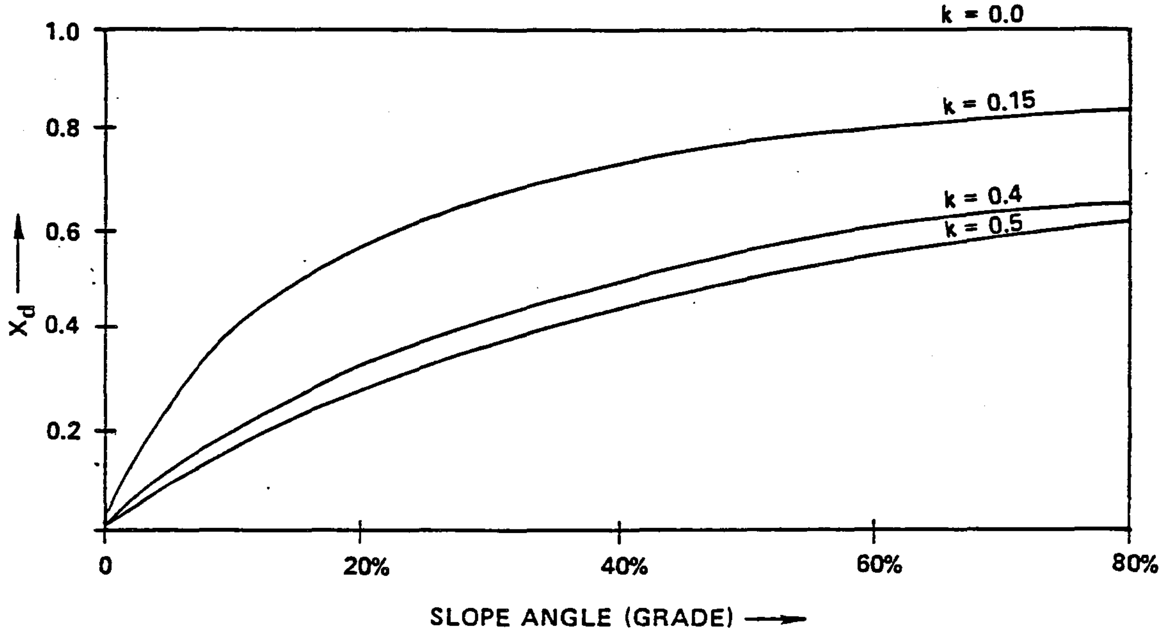


FIG. 5-21 SLOPE ANGLE FOR SFS = 1.0 FOR Φ = 35°, C = 0 vs. CORRESPONDING SLOPE ANGLE FOR ANY Φ

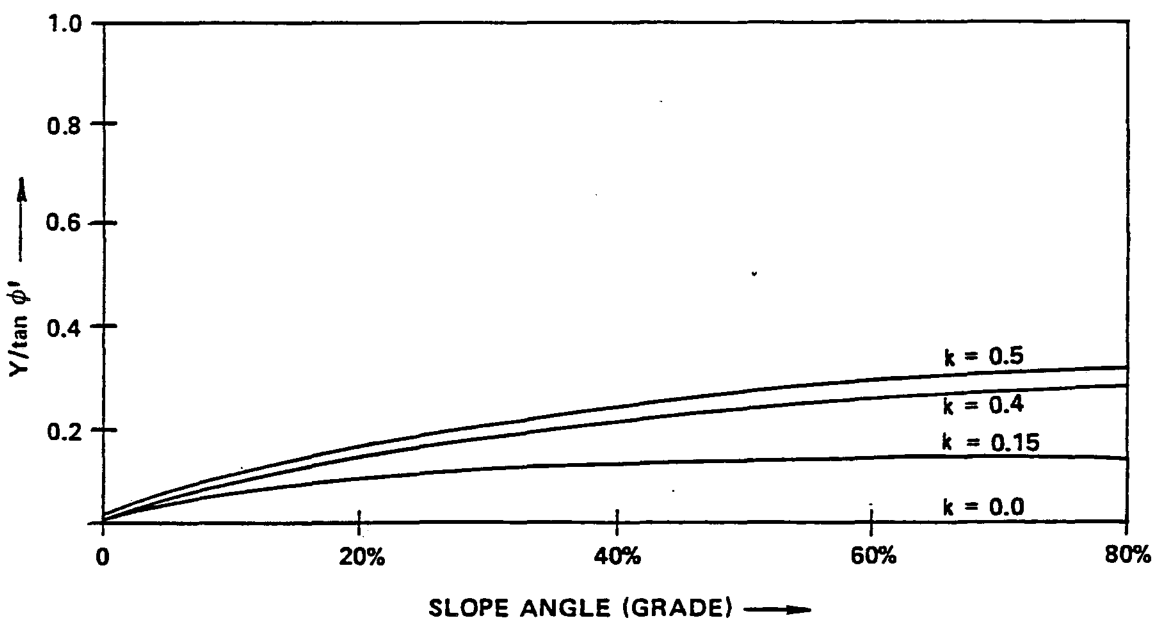


NOTE: $R_2 = \dfrac{C\ (W + 2D)}{WD\gamma\ (1 + \gamma_p d/\gamma D)}$

FIG. 5-22 COHESION COMPONENT OF STATIC FACTOR OF SAFETY (R_2) FOR C = 250 psf vs. CORRESPONDING COHESION COMPONENT FOR OTHER VALUES OF C



(a)



(b)

FIG. 5-29 COMPUTATION OF DYNAMIC FACTOR OF SAFETY

$$DFS = (SFS)\ X_d - Y$$

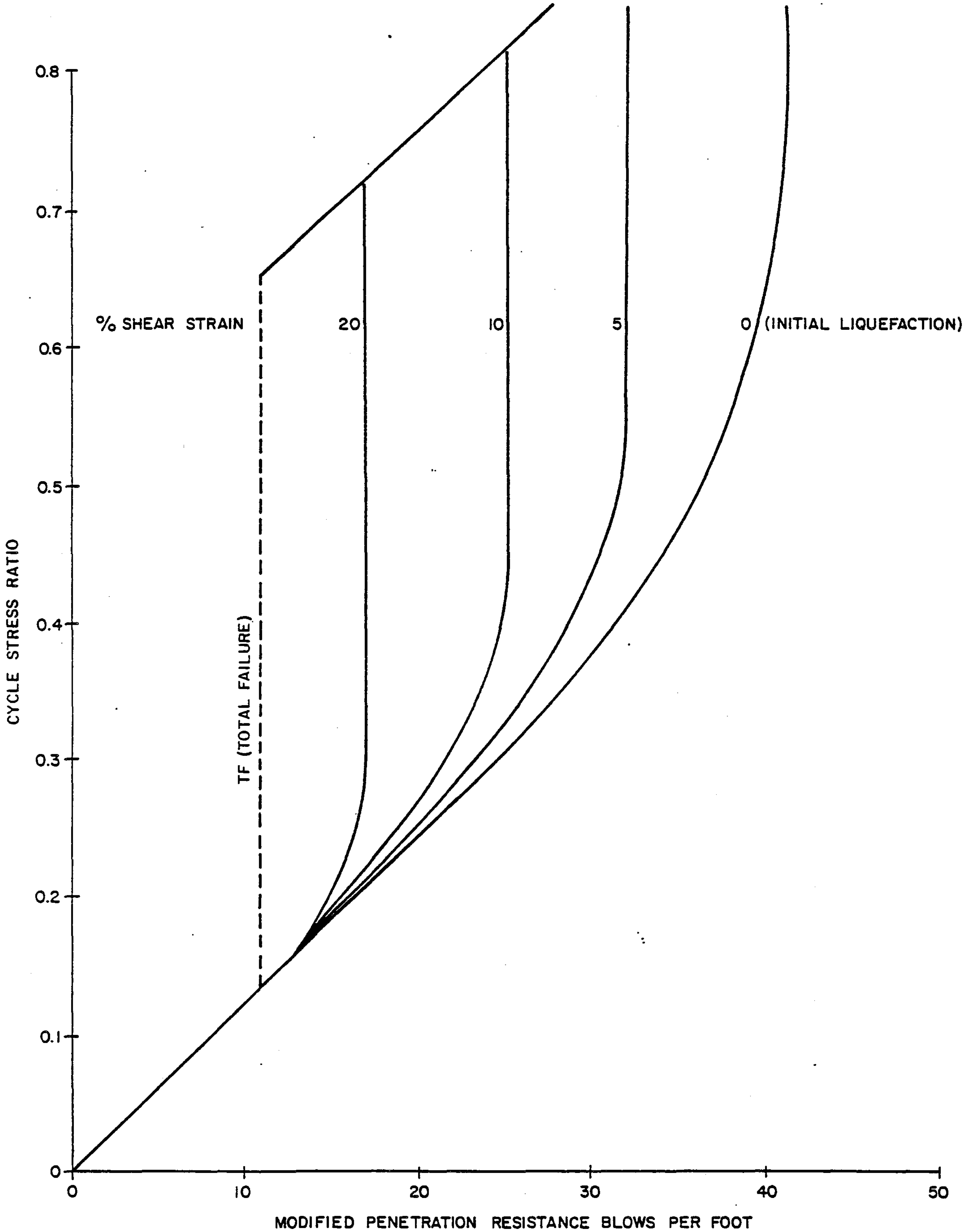


FIG. 5-59 Relationship Between Cyclic Stress Ratio Causing Different Strains and Penetration Resistance for Earthquake Magnitude = 7.0

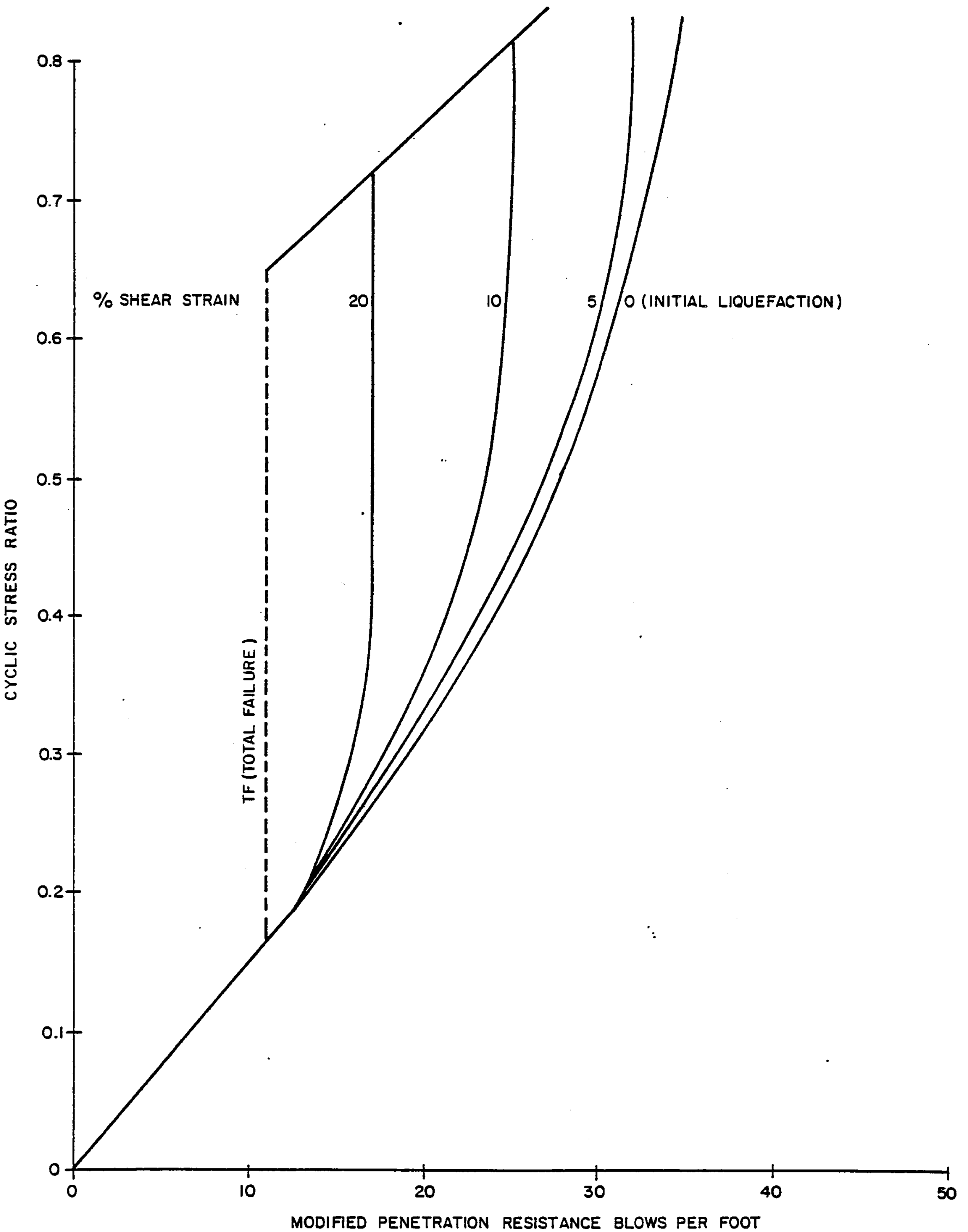


FIG. 5-60 Relationship Between Cyclic Stress Ratio Causing Different Strains and Penetration Resistance for Earthquake Magnitude = 5.5

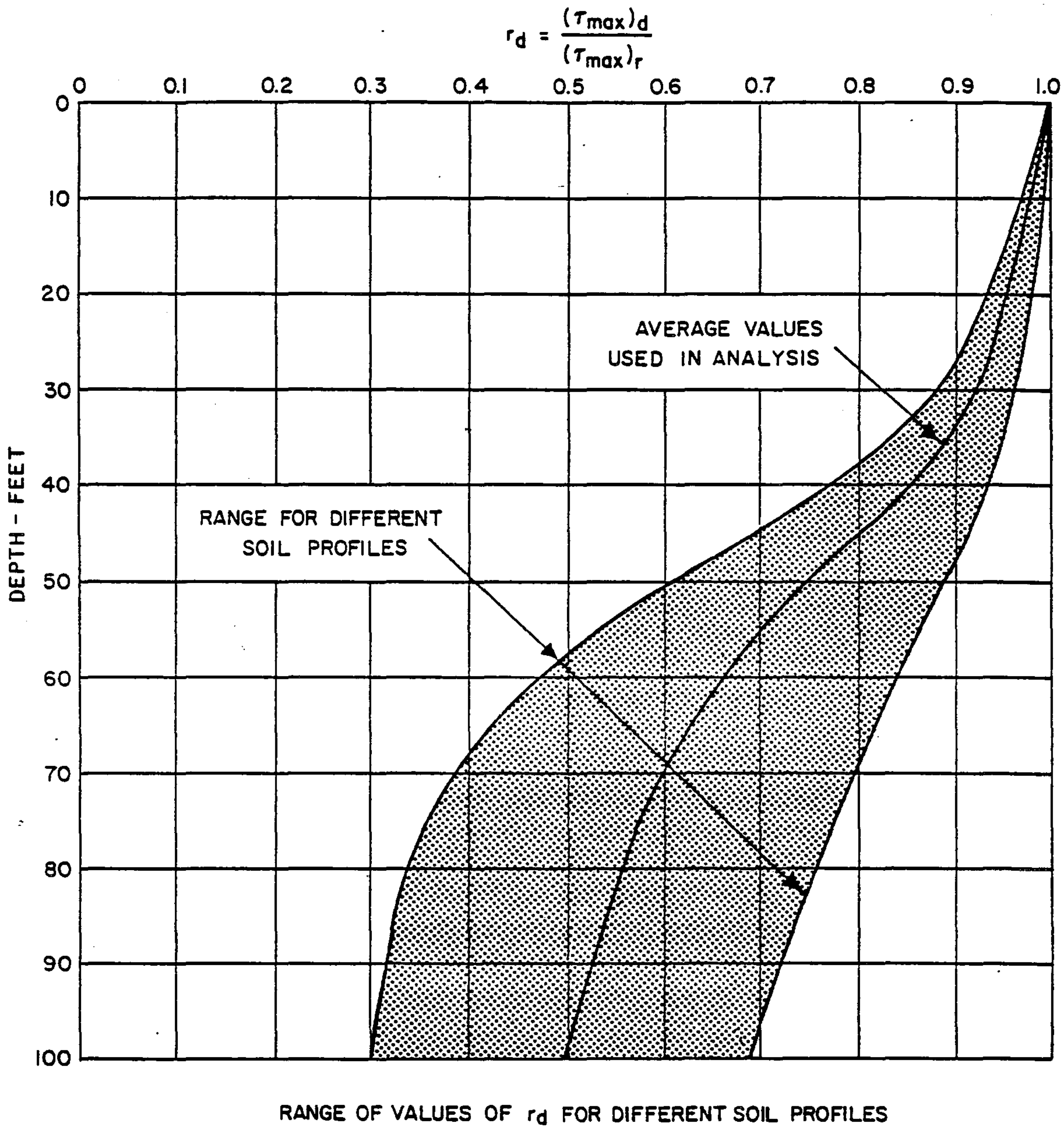


FIG. 5-61 Variation of r_d With Depth

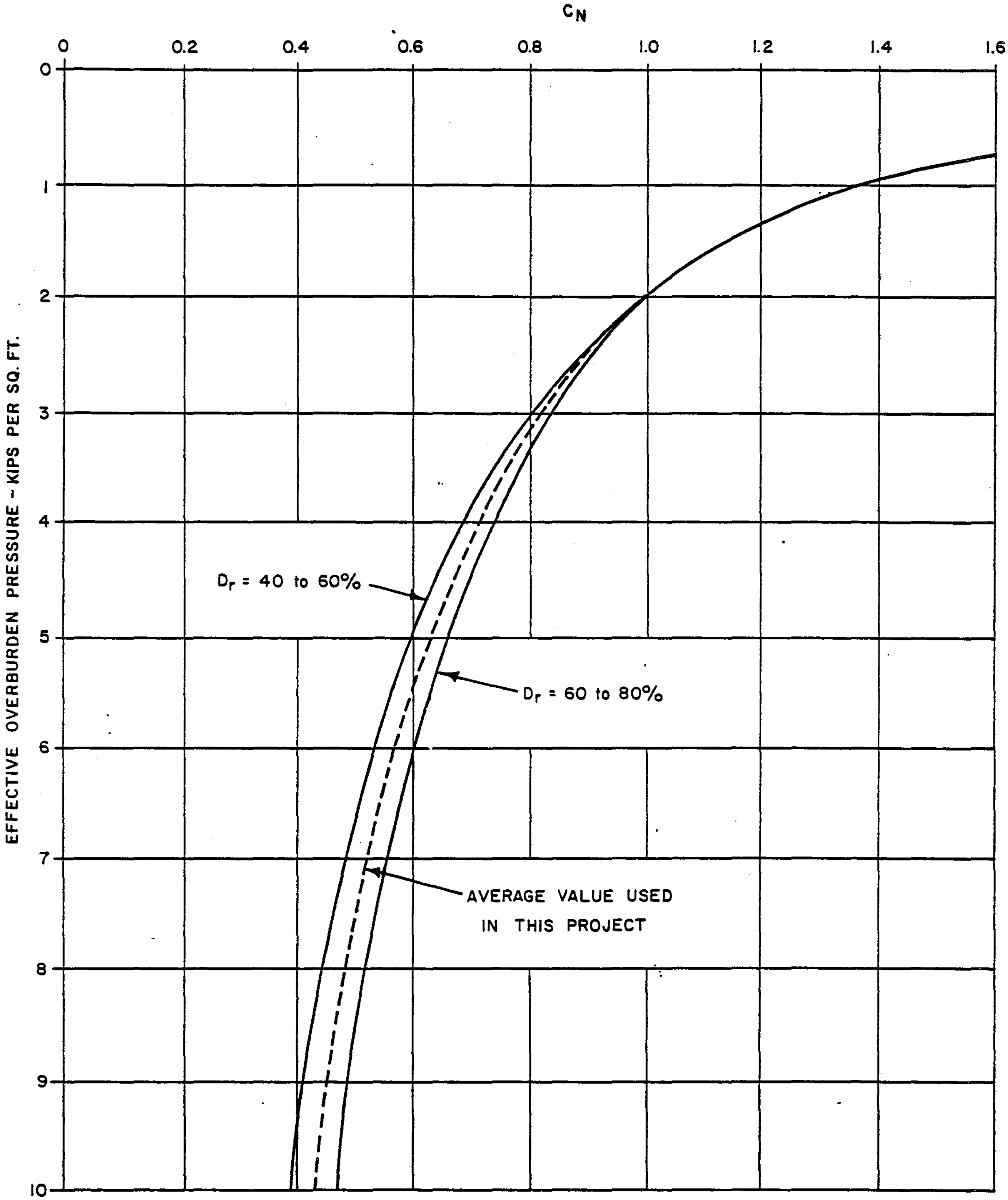


FIG. 5-62 Recommended Curves for Determination of C_N Based on Averages for W.E.S. Tests

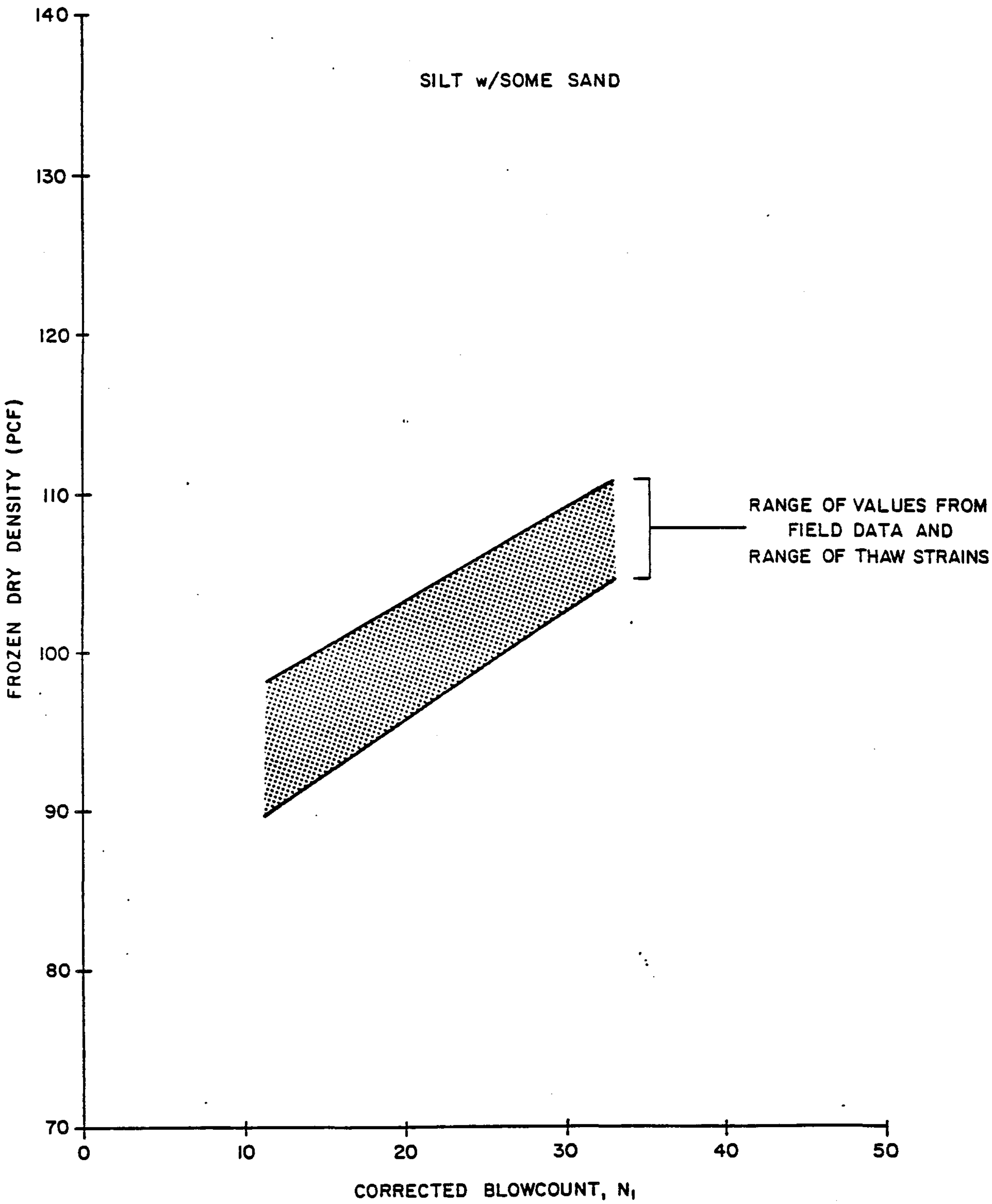


FIG. 5-63 FROZEN DRY DENSITY VERSUS CORRECTED BLOW COUNT

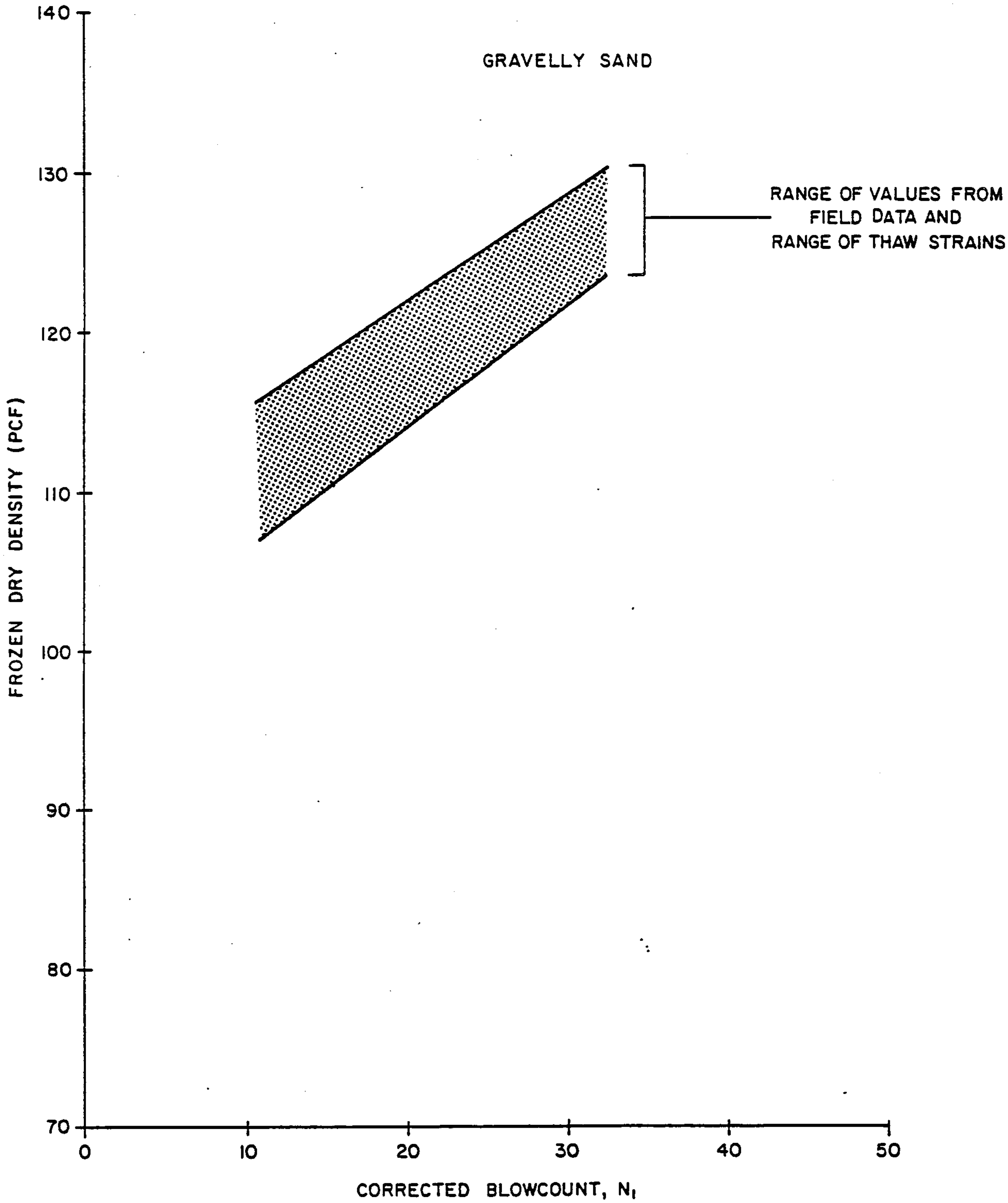


FIG. 5-65 FROZEN DRY DENSITY VERSUS CORRECTED BLOW COUNT

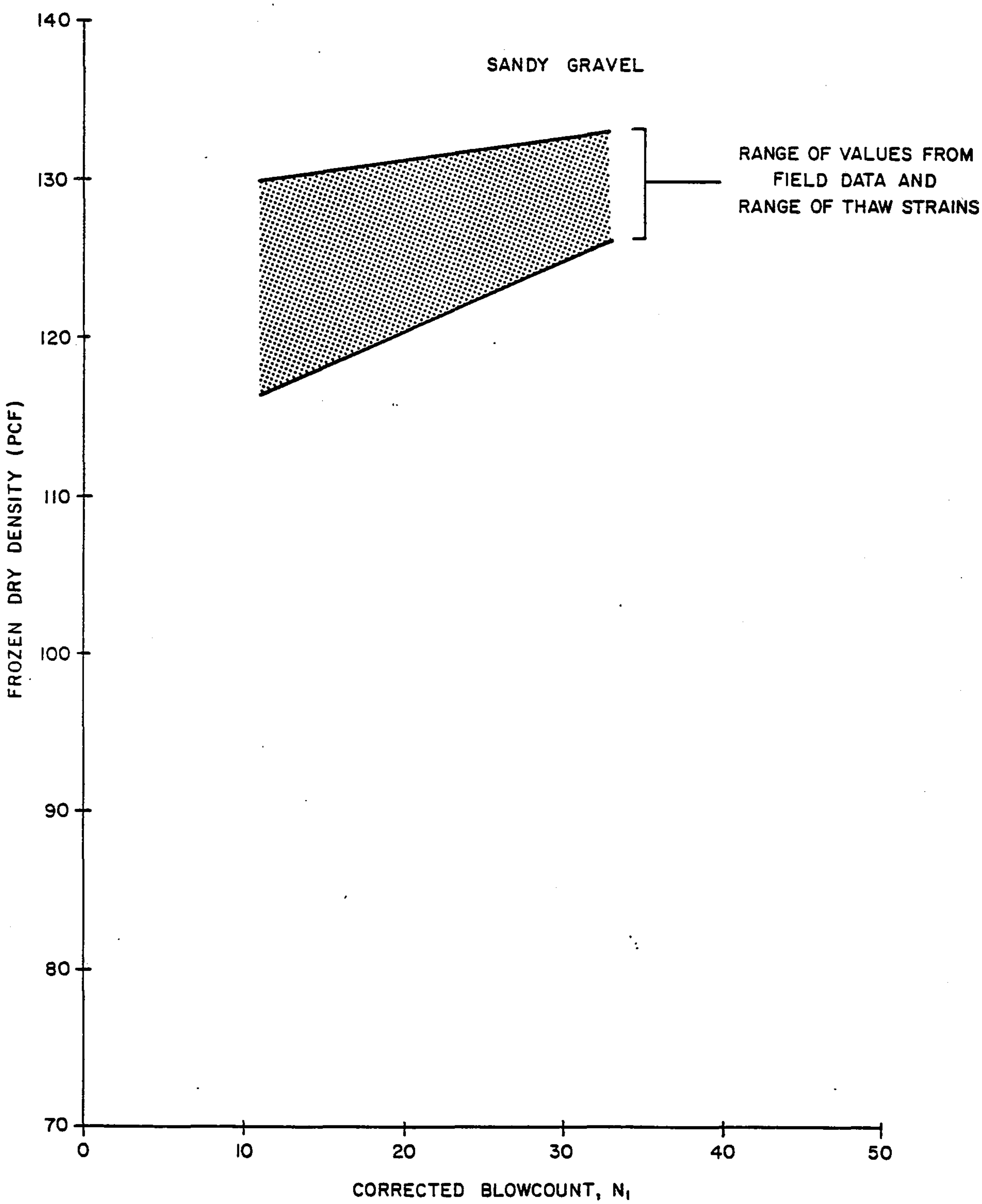


FIG. 5-66 FROZEN DRY DENSITY VERSUS CORRECTED BLOW COUNT

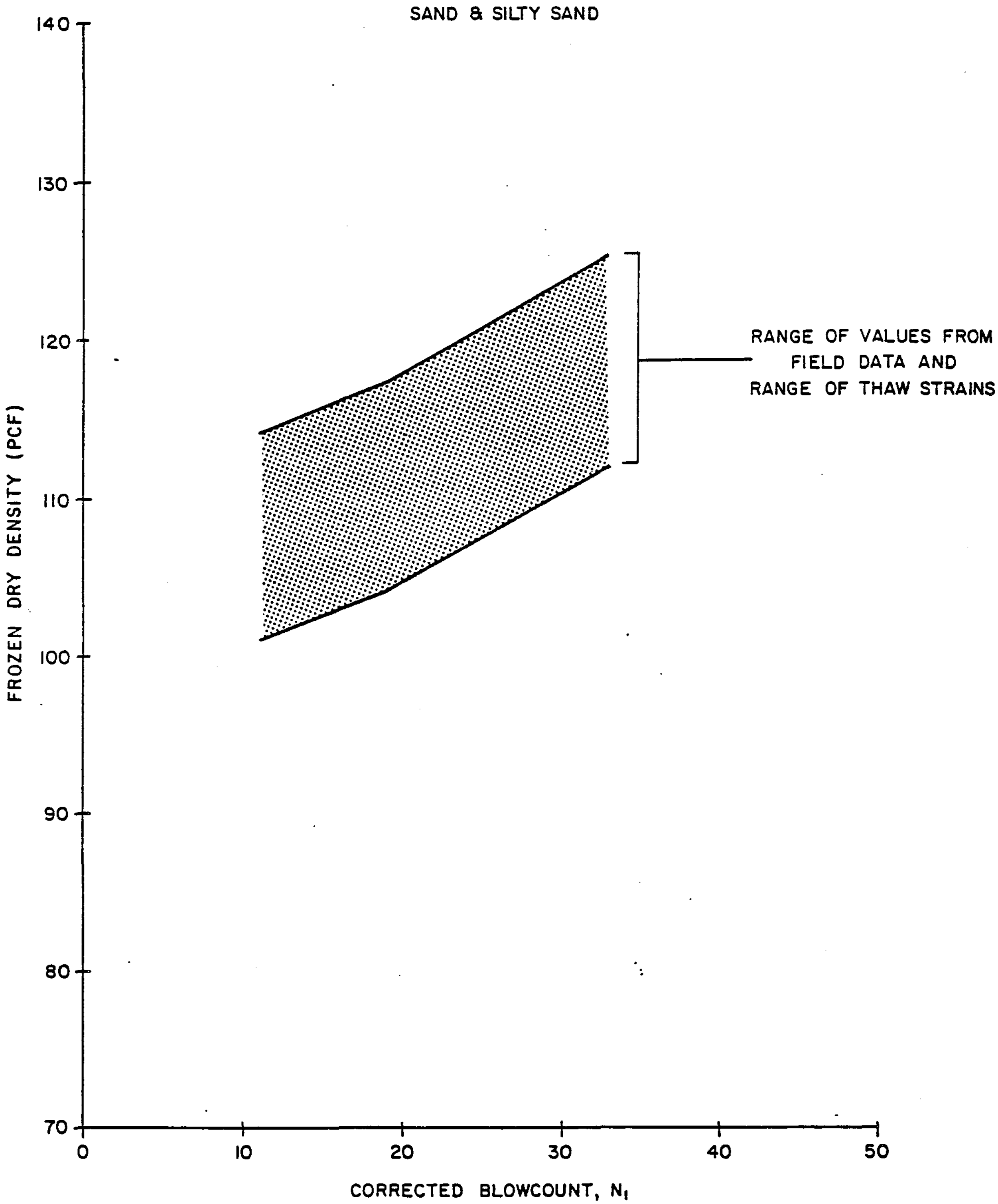


FIG. 5-68 FROZEN DRY DENSITY VERSUS CORRECTED BLOW COUNT

APPENDIX B

-

TSC TABLES AND DRAWINGS

TABLE B-1. LIQUEFACTION ANALYSIS OF BOREHOLE N 0031-046

R & M CONSULTANTS, INC.

BOREHOLE N0031-046

ALIGNMENT SHEET 37

OBSERVED GWT - ****

DESIGN ACCEL - .15

SAT DENSITY = 133.

TERRAIN UNIT FS/GT+G+/S

DRILLING DATE 11/ 8/79

EARTHQUAKE MAGNITUDE 5.5

*** - NOT APPLICABLE

TF - TOTAL FAILURE

B/F - BLOWS PER FOOT

DEPTH	SM	B/F	N1	CRITICAL N1 FOR STRAINS OF 0%	5%	10%	20%	TF	CRITICAL ACCEL FOR STRAINS OF 0%	5%	10%	20%	TF	CRITICAL GWT FOR STRAINS OF 0%	5%	10%	20%	TF	FACTOR OF SAFETY FOR STRAINS OF 0%	5%	10%	20%	TF	EST. STRAIN	
2.0	SH	14	28	7	7	7	7	7	.78	.81	***	***	***	****	****	****	****	****	4.3	4.3	4.3	4.3	4.3	0	G
GWT= 0.0				13	12	12	12	11	.42	.43	***	***	***						2.2	2.3	2.3	2.3	2.5	0	
5.0	SH	13	21	7	7	7	7	7	.54	.57	.62	***	***	****	****	****	****	****	3.3	3.3	3.3	3.3	3.3	0	G
GWT= 0.0				12	12	12	12	11	.29	.30	.33	***	***						1.7	1.8	1.8	1.8	1.9	0	

TECHNICAL SUPPORT CONTRACTOR COMPUTATIONS

DEPTH (Ft)	DRY DENSITY (pcf)	N_1	CYCLIC STRESS RATIO	CRITICAL N_1 FOR STRAINS OF 0%	5%	10%	20%	TF	CRITICAL CYCLIC STRESS RATIO FOR STRAINS OF 0%	5%	10%	20%	TF	CRITICAL ACCEL. FOR STRAINS OF 0%	5%	10%	20%	TF	FACTOR OF SAFETY FOR STRAINS OF 0%	5%	10%	20%	TF
2.0	124	28	0.098	6	6	6	6	6	0.508	0.540	-	-	-	0.78	0.83	-	-	-	4.7	4.7	4.7	4.7	4.7
			0.18	12	12	12	12	11						0.42	0.44	-	-	-	2.3	2.3	2.3	2.3	2.6
5.0	122	20	0.097	6	6	6	6	6	0.317	0.332	0.362	-	-	0.49	0.52	0.56	-	-	3.3	3.3	3.3	3.3	3.3
			0.18	12	12	12	12	11						0.26	0.28	0.30	-	-	1.7	1.7	1.7	1.7	1.8

TABLE B-2. LIQUEFACTION ANALYSIS OF BOREHOLE N 77-66

TECHNICAL SUPPORT CONTRACTOR COMPUTATIONS

ALIGNMENT SHEET: 37
DESIGN ACCEL.: 0.15

TERRAIN UNIT $\frac{Fs}{Gt} + \frac{Gt}{S}$

EARTHQUAKE MAGNITUDE 5.5

DEPTH (Ft)	DRY DENSITY (PCF)	N_1	CYCLIC STRESS RATIO	CRITICAL N_1 FOR STRAINS OF					CRITICAL CYCLIC STRESS RATIO FOR STRAINS OF					CRITICAL ACCEL. FOR STRAINS OF					FACTOR OF SAFETY FOR STRAINS OF				
				0%	5%	10%	20%	TF	0%	5%	10%	20%	TF	0%	5%	10%	20%	TF	0%	5%	10%	20%	TF
3	91	13	0.097 0.275	6 18	6 17	6 16	6 15	6 11	0.195	0.195	0.195	0.195	-	0.30 0.11	0.30 0.11	0.30 0.11	0.30 0.11	- -	2.2 0.7	2.2 0.8	2.2 0.8	2.2 0.9	2.2 1.2
6	72	5*	0.096 0.247	6 16	6 15	6 15	6 14	6 11	0.069	0.069	0.069	0.069	0.069	0.11 0.04	0.11 0.04	0.11 0.04	0.11 0.04	0.11 0.04	0.8 0.3	0.8 0.3	0.8 0.3	0.8 0.4	0.8 0.4
8	123	25	0.096 0.229	6 15	6 15	6 14	6 14	6 11	0.432	0.445	0.71	-	-	0.66 0.28	0.70 0.29	1.1 0.46	- -	- -	4.2 1.7	4.2 1.7	4.2 1.8	4.2 1.8	4.2 2.3
10.5	100	14	0.095 0.212	6 14	6 14	6 14	6 13	6 11	0.209	0.216	0.221	0.225	-	0.33 0.15	0.34 0.15	0.35 0.16	0.36 0.16	- -	2.3 1.0	2.3 1.0	2.3 1.0	2.3 1.1	2.3 1.3
13	112	33	0.095 0.207	6 14	6 13	6 13	6 13	6 11	0.73	-	-	-	-	1.2 0.53	-	-	-	-	5.5 2.4	5.5 2.5	5.5 2.5	5.5 2.5	5.5 3.0

(*) ASSUMED VALUE

NOTES

-- The N_1 values were obtained from Figures 5-63, 5-66 and 5-68.

-- The critical N_1 were obtained from Figure 5-60.

-- The critical accelerations were obtained from Figure 5-60.

-- The cyclic stress ratios were obtained from the following formula:

$$\tau/\sigma_o' = 0.65 \frac{a_{max}}{g} \cdot \frac{\sigma}{\sigma_o'} \cdot r_d$$

TABLE B-3. LIQUEFACTION ANALYSIS OF BOREHOLE N 3-05

R & M CONSULTANTS, INC.

BOREHOLE N0003-005

ALIGNMENT SHEET 119 | TERRAIN UNIT ES | *** - NOT APPLICABLE

OBSERVED GWT - **** | DRILLING DATE 6/29/76 | TF - TOTAL FAILURE

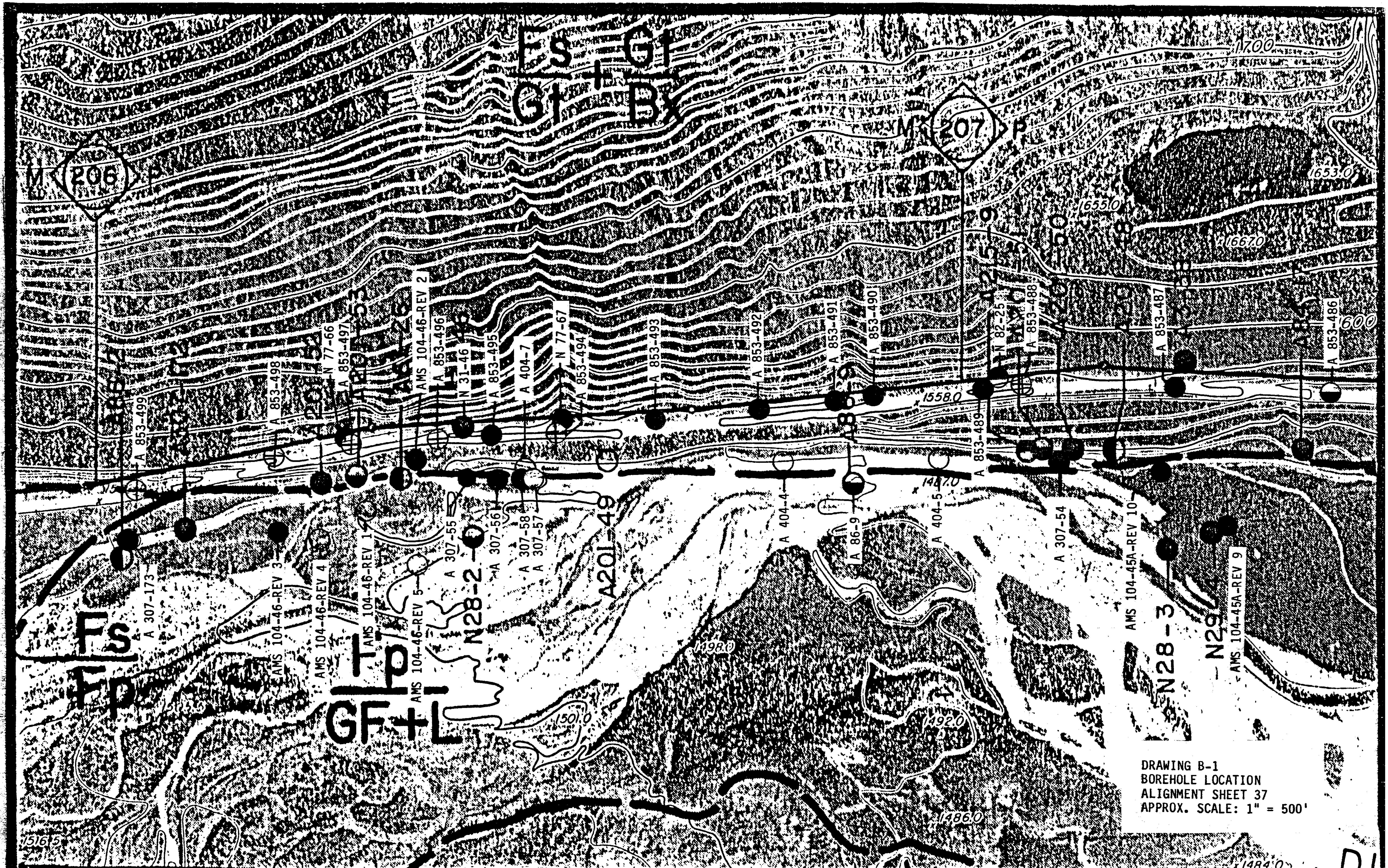
DESIGN ACCEL - .35 | EARTHQUAKE MAGNITUDE 7.0 | B/F - BLOWS PER FOOT

SAT DENSITY = 110.

DEPTH	SM	B/F	N1	CRITICAL N1 FOR STRAINS OF 0%	5%	10%	20%	TF	CRITICAL ACCEL FOR STRAINS OF 0%	5%	10%	20%	TF	CRITICAL GWT FOR STRAINS OF 0%	5%	10%	20%	TF	FACTOR OF SAFETY FOR STRAINS OF 0%	5%	10%	20%	TF	EST. STRAIN		
5.5	SH	7	12	19	18	17	16	11	.23	.23	.23	.23	***	****	****	****	****	****	.6	.7	.7	.8	1.1	32	X	O
GWT= 0.0				37	32	25	17	11	.10	.10	.10	.10	***						.3	.4	.5	.7	1.1	32	X	
10.0	SH	6	8	18	18	17	16	11	.15	.15	.15	.15	.15	****	****	****	****	****	.4	.4	.5	.5	.7	**	X	O
GWT= 0.0				37	32	25	17	11	.06	.06	.06	.06	.06						.2	.2	.3	.5	.7	**	X	
15.0	SH	7	8	18	17	17	16	11	.15	.15	.15	.15	.15	****	****	****	****	****	.4	.4	.4	.5	.7	**	X	O
GWT= 0.0				37	32	25	17	11	.06	.06	.06	.06	.06						.2	.2	.3	.4	.7	**	X	

TECHNICAL SUPPORT CONTRACTOR COMPUTATIONS

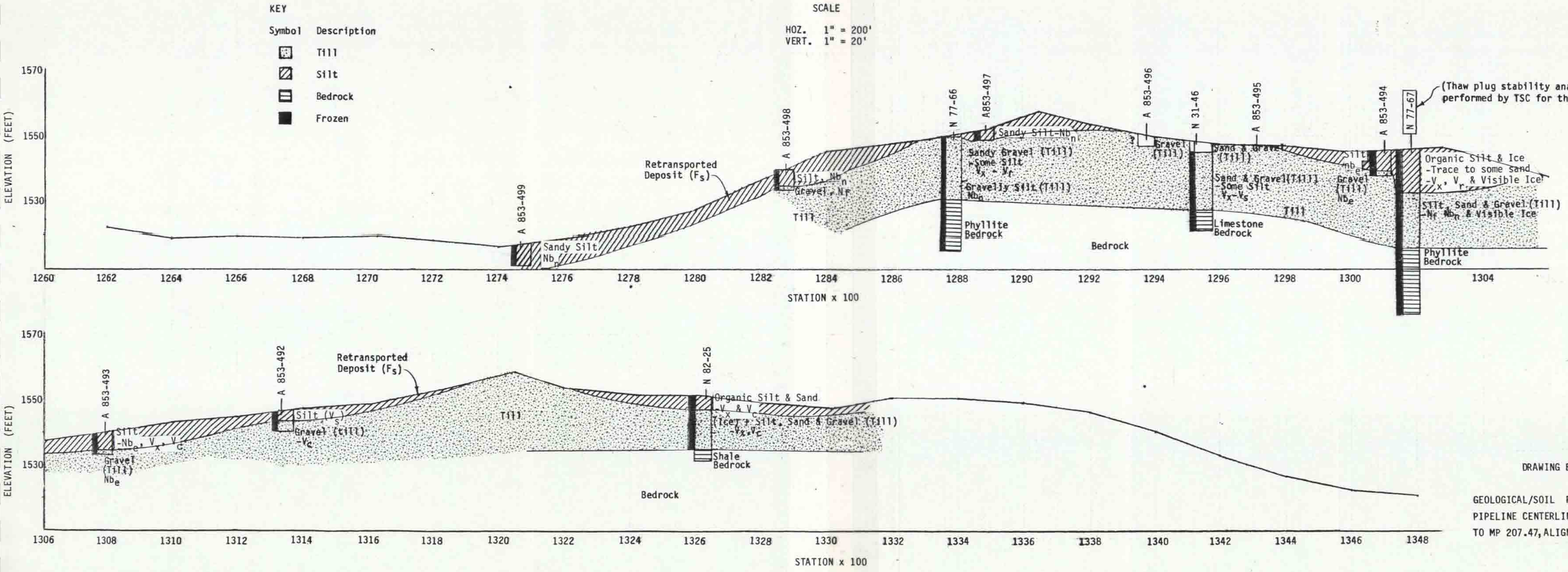
DEPTH	CRITICAL N_1 FOR STRAINS OF 0%	5%	10%	20%	TF	CRITICAL ACCEL. FOR STRAINS OF 0%	5%	10%	20%	TF	FACTOR OF SAFETY FOR STRAINS OF 0%	5%	10%	20%	TF
10.0	18	18	17	16	11	0.15	0.15	0.15	0.15	0.15	0.4	0.4	0.5	0.5	0.7
GWT 0.0	37	32	25	17	11	0.06	0.06	0.06	0.06	0.06	0.2	0.25	0.3	0.5	0.7



DRAWING B-1
BOREHOLE LOCATION
ALIGNMENT SHEET 37
APPROX. SCALE: 1" = 500'

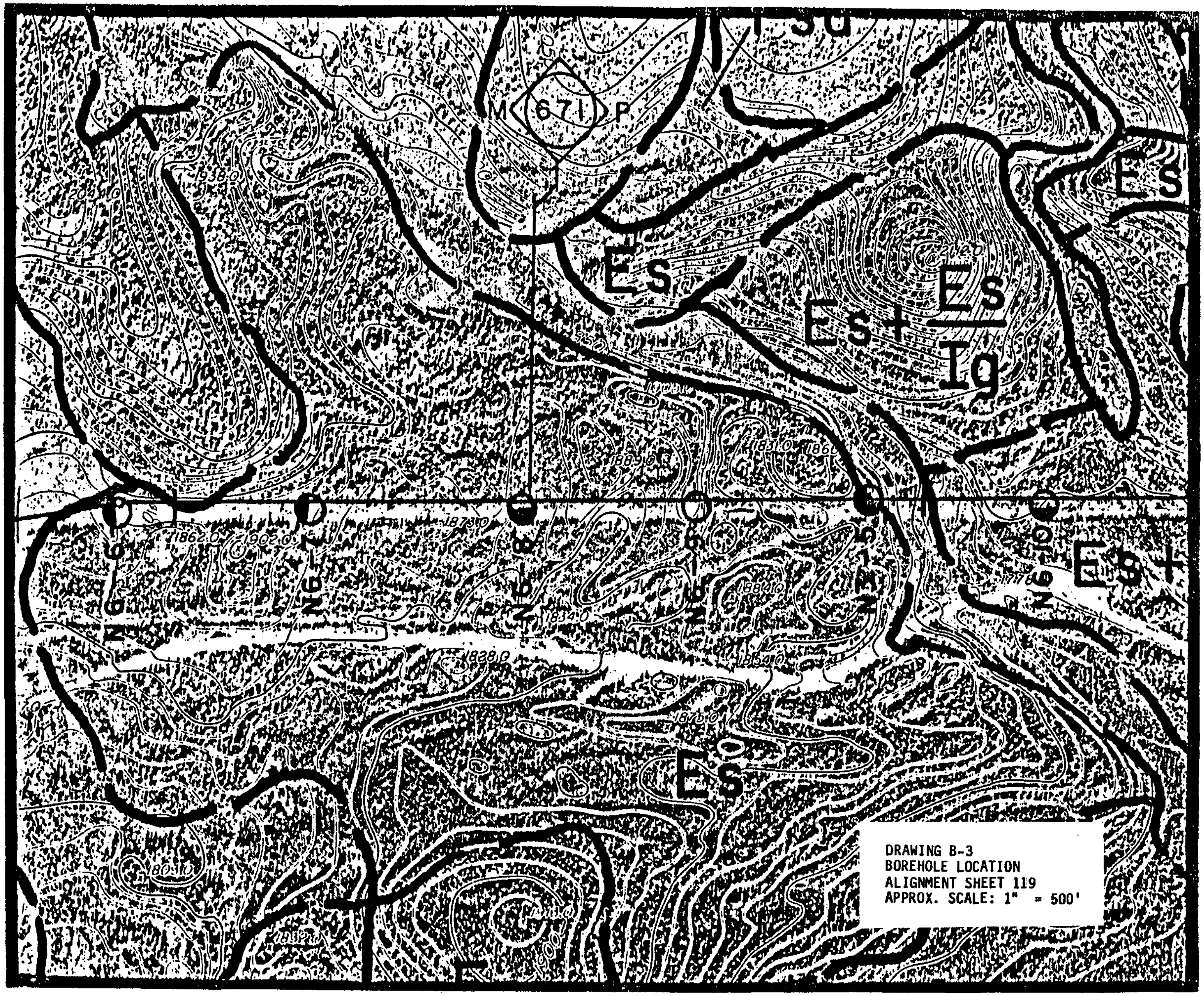
CENTERLINE PROFILE STATION 126200 THRU 134800

ALIGNMENT SHEET NUMBER 37



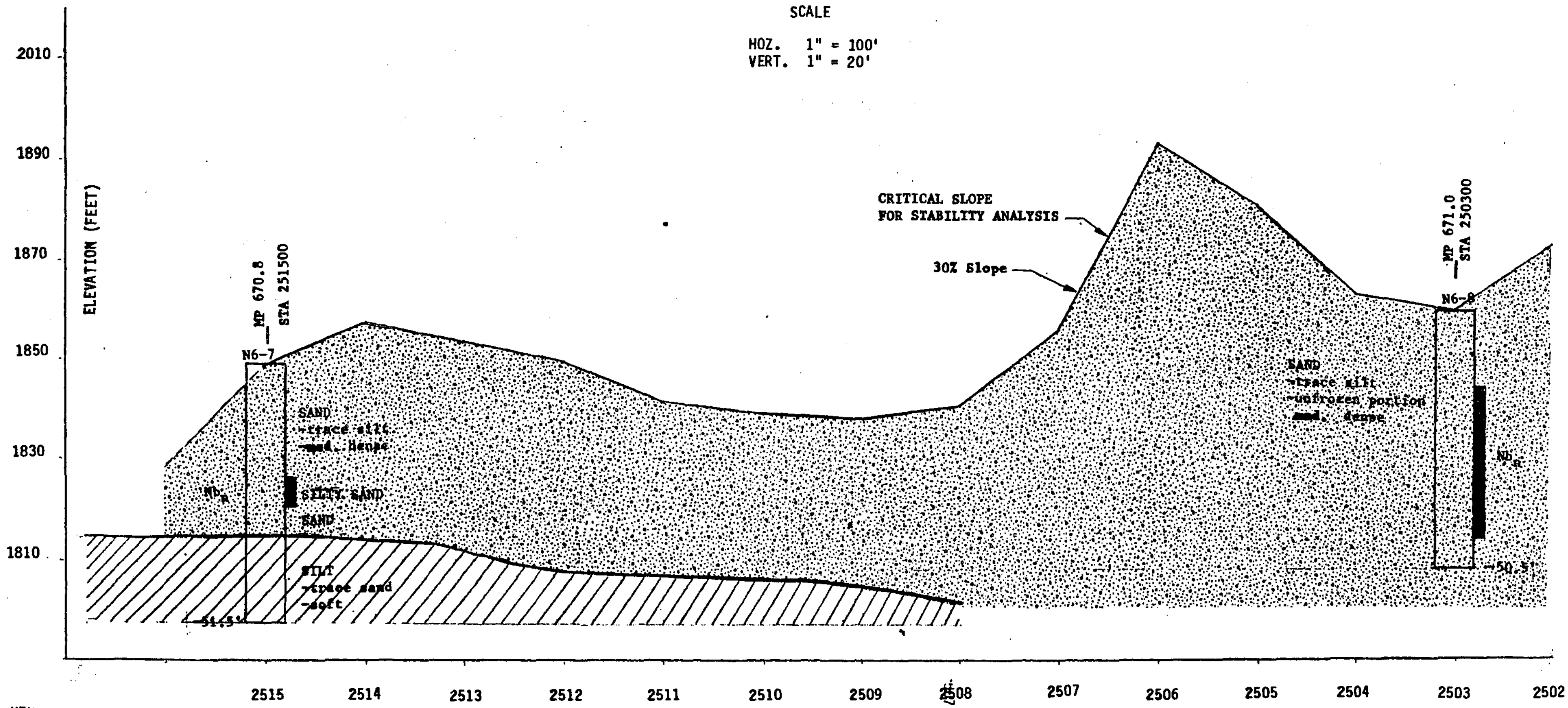
DRAWING B-2

GEOLOGICAL/SOIL PROFILE ALONG PIPELINE CENTERLINE -MP 205.98 TO MP 207.47, ALIGNMENT SHEET 37



DRAWING B-3
BOREHOLE LOCATION
ALIGNMENT SHEET 119
APPROX. SCALE: 1" = 500'

CENTERLINE PROFILE STATION 250300 THRU 251500
ALIGNMENT SHEET NUMBER 119



KEY

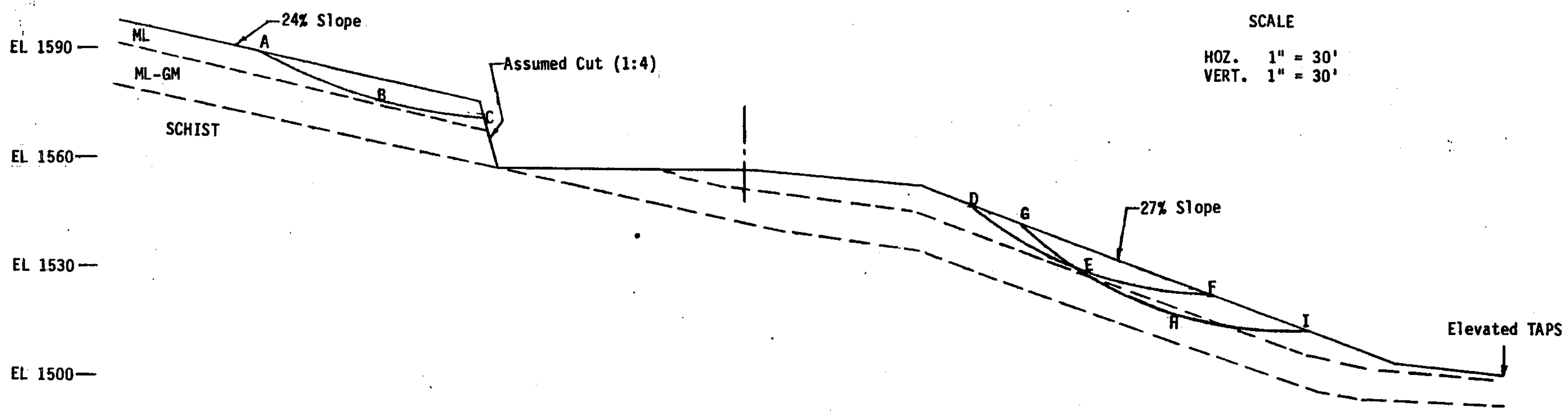
Symbol	Description
	Sand
	Silt
	Frozen Soil

DRAWING B-4

GEOLOGICAL/SOILS PROFILE
ALONG PIPELINE CENTERLINE
MP 670.7 TO 671.4
ALIGNMENT SHEET 119

ALIGNMENT SHEET 37

STATION 129042



SLOPE STABILITY ANALYSIS - UNDRAINED CONDITIONS

Soil Type (*)	W %(*)	Dry Density PCF (*)	Bulk Density PCF	Friction Degree (**)	Cohesion PSF (**)	Failure Surface	STATIC FACTOR OF SAFETY NWA's Method	STATIC FACTOR OF SAFETY Computer Result	DYNAMIC FACTOR OF SAFETY NWA's Method	DYNAMIC FACTOR OF SAFETY Computer Result
ML	125	60	125	8°	150	ABC	1.2	1.3	0.7	0.8
ML-GM	30	90	117	18°	350	DEF		~~1.5~~		~~1.0~~
						GHI		~~2.0~~		~~1.4~~

NOTES

* NWA Data
** Values obtained form Table 4-2
Profile plotted from NWA's survey data
NWA's idealized soil layers consist of 6 feet of Silt overlying 10 feet of Silty Gravel.

DRAWING B-5

SLOPE STABILITY ANALYSIS
MP 206.35