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Appendix A (NWA tables and figures) is included in the table of contents, but lacking in this draft. Those charts and graphs are included in the more complete draft report.

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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 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.

3.0 LIST OF QUESTIONS

(1) In liquefaction analysis, NWA estimates the strains for a safety factor of 1.0. The significance of these strains is not clear. NWA has not said what they plan to do when strains are particularly large (for examples 20% or 30%).

(2) In one particular instance the liquefaction analysis showed that the borehole on a slope would liquefy. Yet the dynamic stability analysis indicated that the slope would have practically no movement during the design earthquake. It is not clear how the NWA criteria 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.

(5) If 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 the boreholes, except Borehole N6-08, where the saturated density was assumed to be 117 PCF. It is not apparent why a higher density was interpreted for Borehole N6-08, as the SPT blow counts were lower than for 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 was used.

4.0 TERRAIN STABILITY STUDY - ALIGNMENT SHEET 37

4.1 TERRAIN AND SOIL CONDITIONS

The gas pipeline route 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 tills with low to moderate ice content. The till overlies sedimentary bedrock. The F_s silt deposit is between 0 and 6~~0~~ feet in thickness, 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 this area a total of 16 holes were drilled either on or close to the right-of-way by Alyeska and NWA. Most of the Alyeska holes 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 holes 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 ~~zone~~, for which the design peak acceleration is 0.15. The NWA's liquefaction analysis procedure consists of checking the liquefaction potential in each borehole where thawing is expected. The liquefaction potential is evaluated by computing the cyclic stress ratio and modified penetration resistan~~t~~ce using the following ~~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
- 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.

Liquefaction analys~~i~~es ~~was~~ were performed for Boreholes N0031-046 and N77-66. For liquefaction analysis of Borehole N0031-046 NWA's 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, 20 percent shear strain and total failure. These figures are presented in Appendix A of this report.

The results of the analysis are presented on Table B-1 in Appendix B. The agreement between NWA's results and the TSC's results are generally good. There are differences in the factor 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 of 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 Borehole N77-66. For the six foot depth sample the density was relatively low (72 PCF) and the corrected blow count could not be found using ~~the~~ NWA's figures. The corrected blow count was assumed to be five.

An attempt was made to analyze Borehole N77-66 for liquefaction potential but because of the low density soils NWA's 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, the 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, the undrained strength parameters are indicated as follows:

$\phi_{cu} = 6°$
$C_{cu} = 100$ PSF

The predicted depth of thaw is determined from Figure 5-8. For a work pad of two feet thick, ~~The~~ 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.

Thaw Plug Stability After Two Years

(1) Within the work pad area

From Figure 5-19 the slope angle for $\beta_{SFS} + 1.0$ is 28°

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

From Figure 5-20 the cohesion component R_2 is 0.45

The corrected R_2 to account for a cohesion different from the value used in Figure 5-20 is $0.45 \times \frac{\text{Design Cohesion}}{250} = 0.45 \times 100 = 0.18$

From Equation 5.1.19, the Static Factor of Safety (SFS) is computed:

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

$$= \frac{\tan 4°}{0.21} + \frac{0.18}{0.203}$$

$$= 0.33 + 0.89 = 1.22$$

From Equation 5.1.26, the Dynamic Factor of Safety (DFS) is computed:

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

$$DFS = (1.22)\ \frac{0.21}{0.21 + 0.15} = 0.71$$

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

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

where: d = slope

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.

$$N = (0.22)(0.204)(0.99) = 0.045$$

$$\text{hence, } d = \frac{7^2}{2(386)(0.045)}\left(1 - \frac{0.045}{0.15}\right)\left(\frac{0.15}{0.045}\right) = 3.3 \text{ inches}$$

Therefore, the thaw plug is considered to be marginally stable for the two year analysis.

(2) Disturbed Zone Outside the Work Pad

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

SFS = 1.95

DFS = 1.14

Thaw Plug Stability After 25 Years

(1) Within The Work Pad Area

From Figure 5-17 the slope angle for β_{SFS} = 1.0 is 21.2°

From Figure 5-21 the corrected slope angle for β_{SFS} = 1.0 is 3° (the soil friction angle is 6°)

From Figure 5-20 the cohesion component R_2 is ~~[illegible]~~ 0.20

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

$$SFS = \frac{\tan \beta SFS = 1.0(C=0)}{\tan \beta} + \frac{R^2}{\sin \beta}$$

$$= 0.21 + 0.59$$

$$= 0.80 \text{ (unstable)}$$

(2) Disturbed Zone Outside the Work Pad

SFS = 0.84 (unstable)

4.3.2 Thaw Plug Stabiity Using Generalized Soil Conditions

Thaw plug stability analysis were carried out using the NWA's procedure and the computer program Slope II. A transverse slope with gradient up to 27 percent (MP 206.35) was selected for the analysis.

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 bedrock schist. Drawing B-5, in Appendix B, illustrate the soil profile and presents the soil parameters and the results of the analyses.

The stability of a 24 percent (13.5°) slope and six feet of thaw using the NWA's method was computed as follows:

- The water table is assumed at the surface and the thickness of the work pad is zero.

go together

From Figure 5-16 the slope angle for $\beta_{SFS} = 1.0$ is ~~10°~~ 19.7°

From Figure 5-21 the corrected slope angle for $\beta_{SFS} = 1.0$ is 3.6° (the undrained soil friction angle is 8°)

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

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

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

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

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

The slope displacement is computed using the following values:

$N = (SFS - 1) \sin\beta \cos\phi' = (1.2 - 1) \sin 13.5^\circ \cos 8^\circ$

$N = .046$

$V = 7$ in/sec.

$A = .15$ g

$$d = \frac{V^2}{2gN}\left(1 - \frac{N}{A}\right)\frac{A}{N} = \frac{7^2}{2 \times (386)}\frac{(1 - .31) \times .15}{.046} = 0.14 \text{ ft.}$$

$d = 1.7$ inches

The slope displacement (d) is negligible.

The static and dynamic stability of a 27 percent slope was also calculated using the same method. The thaw depths considered were six feet and nine feet; the nine foot depth being the maximum thaw depth predicted by NWA while the six foot depth is the weakest zone in the upper soil layer from a stability standpoint. The slope stability analysis results are presented on Drawing B-5. The factors of safety obtained using the NWA method (using the figures) ~~is~~ are ~~more conservative~~ slightly less than the values obtained with the computer. ~~results.~~

add a space

5.0 TERRAIN STABILITY STUDY-ALIGNMENT SHEET 119

5.1 TERRAIN AND SOIL CONDITIONS

The gas pipeline would be 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 670.68 and 671.43 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 bore holes were drilled, is eolian sand (E_s). In addition, at the extreme ends of the segment, granite bedrock outcrops at the north end and sandy retransported deposits (F_{sa}) are encountered at the south end. In one of the holes (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 ~~nine percent~~ 98 PCF.

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

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 ~~is~~ 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 bore holes, except bore hole 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 ~~Hole~~ 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 a field blow count of 6, the value of N_1 would be 7.8 and 9.6. 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 Critical Stress Ratio (CSR) must first be calculated according to the Equation 1 ~~[illegible]~~ 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 hole 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 2 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 an 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.

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:

	Critical N_1 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 5-69 and interpolating the strain for a factor of safety of 1.0. Since the sand deposit at a depth of 10 feet in hole N3-05 is predicted to liquefy, no entry is made in the estimated strain column. However, in the liquefaction analysis made at a depth of five feet, a strain of 32 percent is shown. The derivation of such strain not clear.

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

5.3 THAW PLUG STABILITY

The soil type is sand, with a trace of silt, corresponding to terrain type Es and USC 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 frozen and 93 PCF unfrozen.

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 this area the pipeline is located on, or adjacent to the Haines right-of-way. The cross slope gradients range between five and 25 percent with slopes dipping to the southwest. The one foot thick work pad would be 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 N° 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 would correspond to a GWT at the ground surface (critical conditions). The procedure presented by NWA in the preliminary terrain stability analyses was followed to determine the factor of safety.

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 (DSF) are calculated as follows:

(1) Static Stability

- o 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 read $\beta_{SFS = 1.0} = 30.5°$ for thaw depth of D = 21.4 feet,

  - Figure 5-16 is developed for $\phi = 35°$ 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, read $\beta_{SFS} = 1.0$ equal 34.7° for $\phi = 39.8°$ ~~(= 34.7°)~~ on the ordinate, corresponding to $\beta_{SFS = 1.0} = 30.5°$ on the abscissa.

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

$$SFS = \frac{\tan 34.7°}{0.30} = 2.31$$

- This is higher than the SFS of 1.96 shown in the summary sheet.

o GWT = 1.0

- From Figure 5-16 read $\beta_{SFS = 1.0}$ = 21.3°. From Figure 5-21, read $\beta_{SFS = 1.0}$ for ϕ = 39.8° (= 24.5°) on the ordinate, corresponding to $\beta_{SFS = 1.0}$ ~~=~~ at 21.3° on the abscissa.

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

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 equation 5.1.22 as follows:

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

The design acceleration used by NWA in the analysis was 0.4, although a value of 0.35 has been indicated for this section of the pipeline.

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 of this report.

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

N/A = 0.29/0.4 = 0.725
A/N = 1.38
V = 17 in/sec.
g = 386 in/sec.^{2}

$$d = \frac{(17)^2}{2(386)(0.11)} (1 - 0.725)(1.38) = 0.5$$

This compares with a displacement of 0.6 inches calculated by NWA.

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

SFS = 1.52
N = (0.221)(0.52) = 0.11
N/A = 0.275 ; A/N = 3.64

$$d = \frac{(17)^2}{2(386)(0.11)}(1 - 0.275)(3.64) = 9 \text{ inches}$$

For A = 0.35 (instead of 0.4)

$N/A = 0.314$; $\frac{A}{N} = 3.18$

d = 7.4 inches (same as NWA Calculation)

Thaw Plug Stability After Two Years

The same procedure is followed in calculating the short term stability as for the long 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) ~~Hence~~. ~~GWT = 1.0~~.

goes together $\cancel{\dfrac{\tan 25.0°}{0.30}}$

From Figure 5-16, $\beta_{SFS} = 1.0$ is 21.8°

From Figure 5-21, β_{SFS} for 39.8 is 25.8

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

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

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

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

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

Undrained strength: $\phi_{cu} = 36°$ (23)
$C_{cu} = 330$ (450)

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

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
0	1.4	-
0.5	2.1	-
1.0	2.8	0.5

The slope displacement under dynamic conditions (GWT = 1.0) was calculated using the NWA's method. A value of 15 inches was obtained. NWA indicated that the displacement for this slope 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.

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

				CRITICAL N1 FOR STRAINS OF					CRITICAL ACCEL FOR STRAINS OF					CRITICAL GWT FOR STRAINS OF					FACTOR OF SAFETY FOR STRAINS OF					EST.	
DEPTH	SM	B/F	N1	0%	5%	10%	20%	TF	0%	5%	10%	20%	TF	0%	5%	10%	20%	TF	0%	5%	10%	20%	TF	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	

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

TECHNICAL SERVICE CONTRACTOR COMPUTATIONS

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
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						0.48	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

ALIGNMENT SHEET 37
DESIGN ACCEL.: 0.15

TERRAIN UNIT

EARTHQUAKE MAGNITUTE 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

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

TERRAIN UNIT $\frac{F_s}{G+} + \frac{G+}{S}$

EARTHQUAKE MAGNITUTE 5.5

ALIGNMENT SHEET: 37
DESIGN ACCEL.: 0.15

(*) ASSUMED VALUE

NOTES

-- The N_1 volues 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:

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

R & M CONSULTANTS, INC.

BOREHOLE N0003-005

ALIGNMENT SHEET 119

OBSERVED GWT - ****

DESIGN ACCEL - .35

SAT DENSITY = 110.

TERRAIN UNIT ES

DRILLING DATE 6/29/76

EARTHQUAKE MAGNITUDE 7.0

*** - NOT APPLICABLE

TF - TOTAL FAILURE

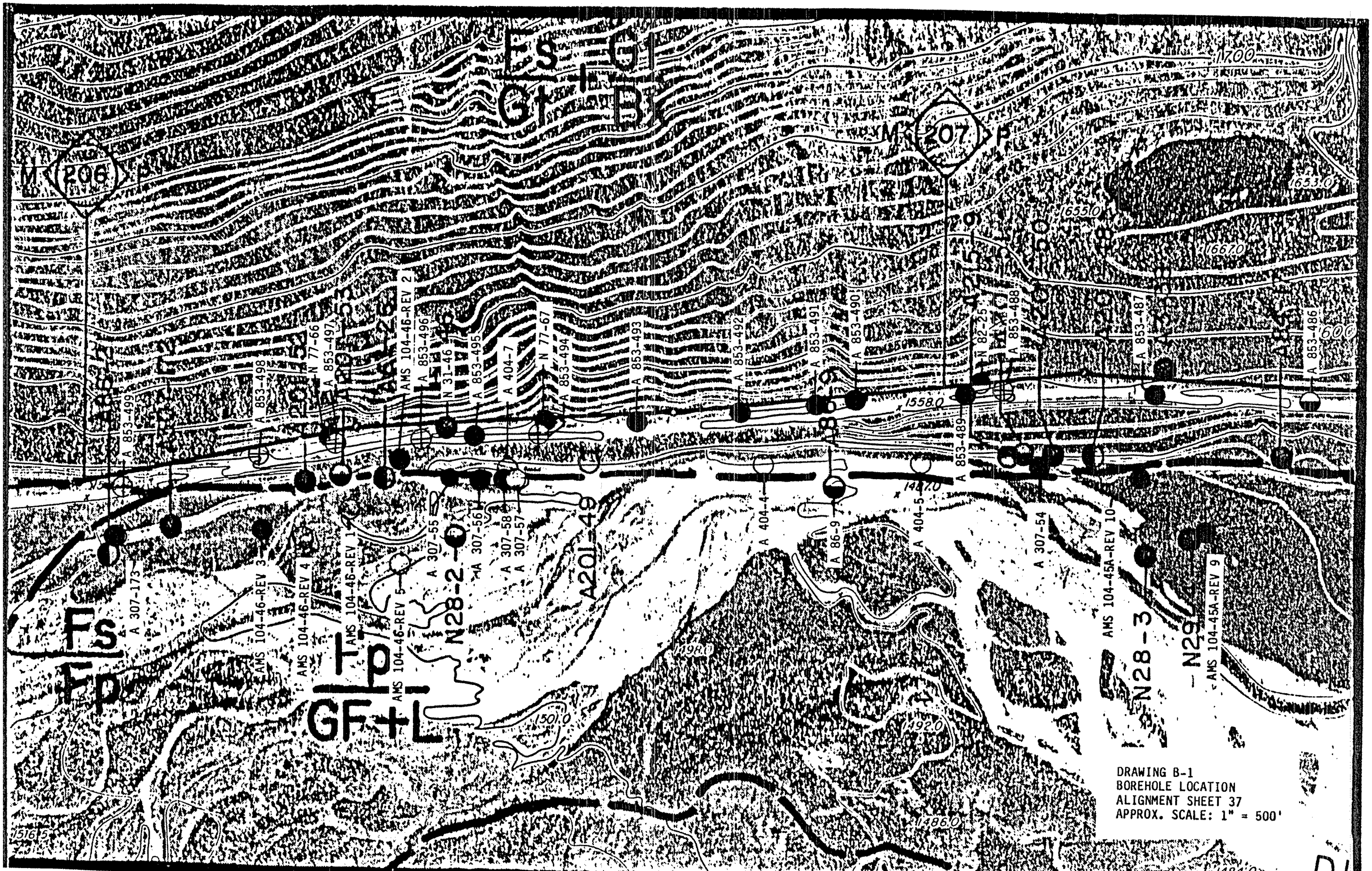
B/F - BLOWS PER FOOT

				CRITICAL N1 FOR STRAINS OF					CRITICAL ACCEL FOR STRAINS OF					CRITICAL GWT FOR STRAINS OF					FACTOR OF SAFETY FOR STRAINS OF						
DEPTH	SM	B/F	N1	0%	5%	10%	20%	TF	0%	5%	10%	20%	TF	0%	5%	10%	20%	TF	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
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
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
GWT= 0.0				37	32	25	17	11	.06	.06	.06	.06	.06						.2	.2	.3	.4	.7	**	X

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

TECHNICAL SUPPORT CONTRACTOR (TSC's) 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	10.	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



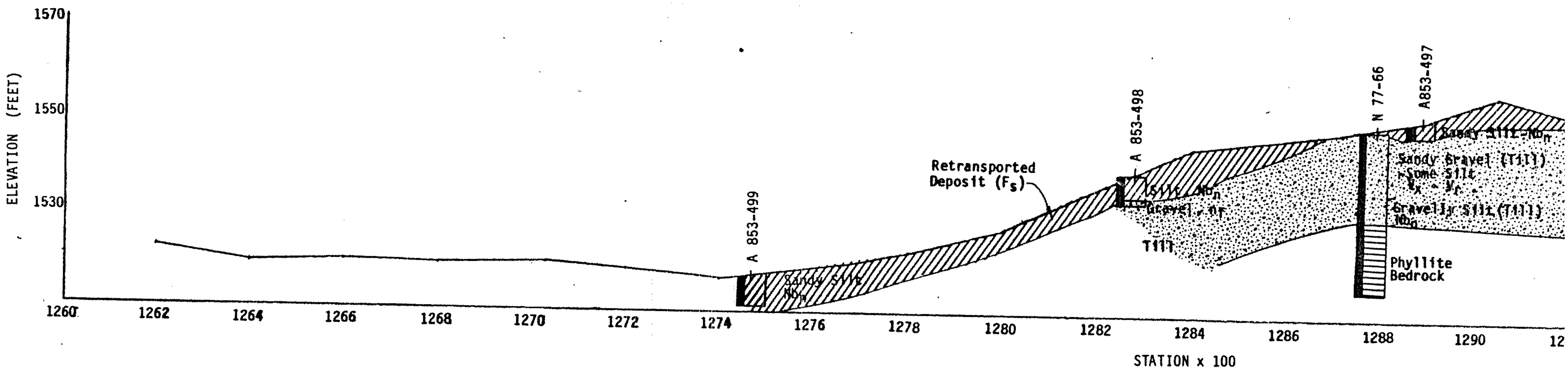
DRAWING B-1
BOREHOLE LOCATION
ALIGNMENT SHEET 37
APPROX. SCALE: 1" = 500'

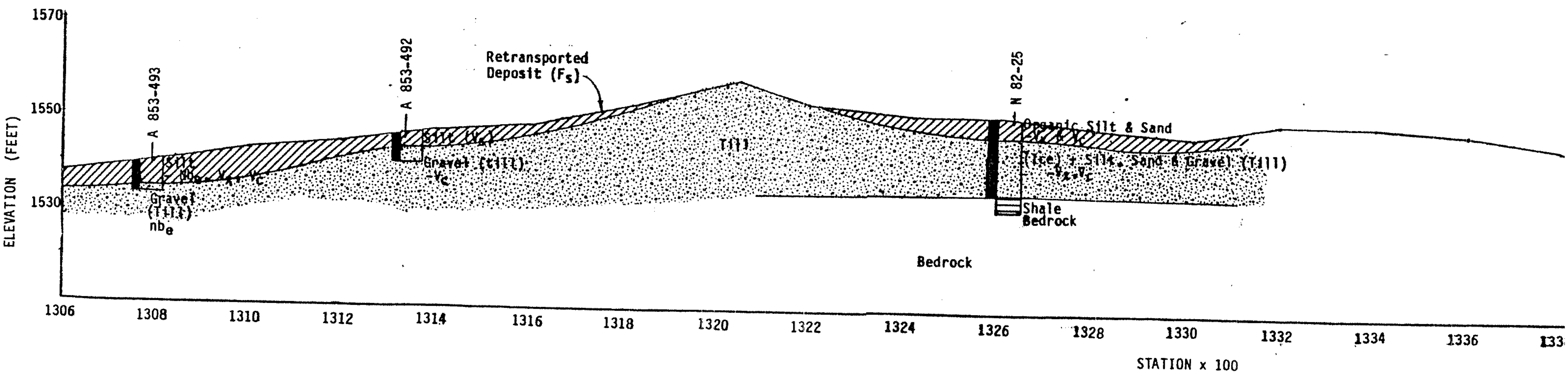
CENTERLINE PROFILE STATION 126200 THRU 134800

ALIGNMENT SHEET NUMBER 37

SCALE

HOZ. 1" = 200'
VERT. 1" = 20'



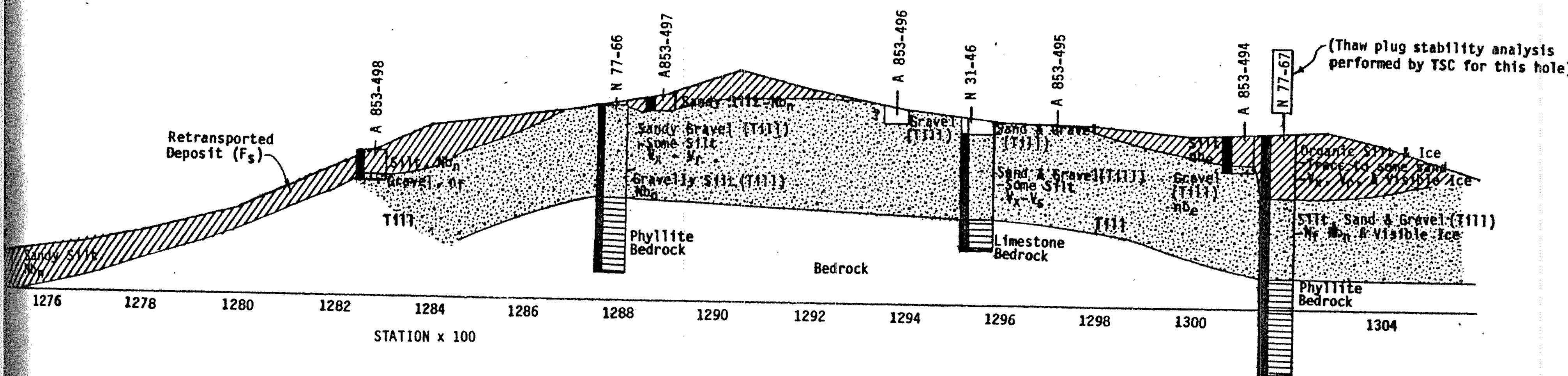


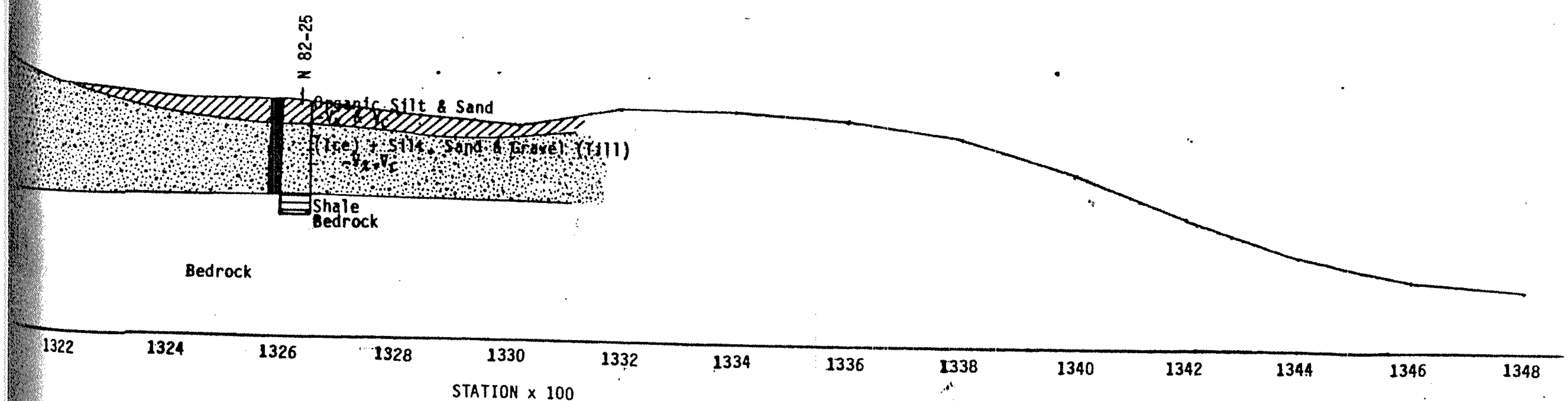
CENTERLINE PROFILE STATION 126200 THRU 134800

ALIGNMENT SHEET NUMBER 37

SCALE

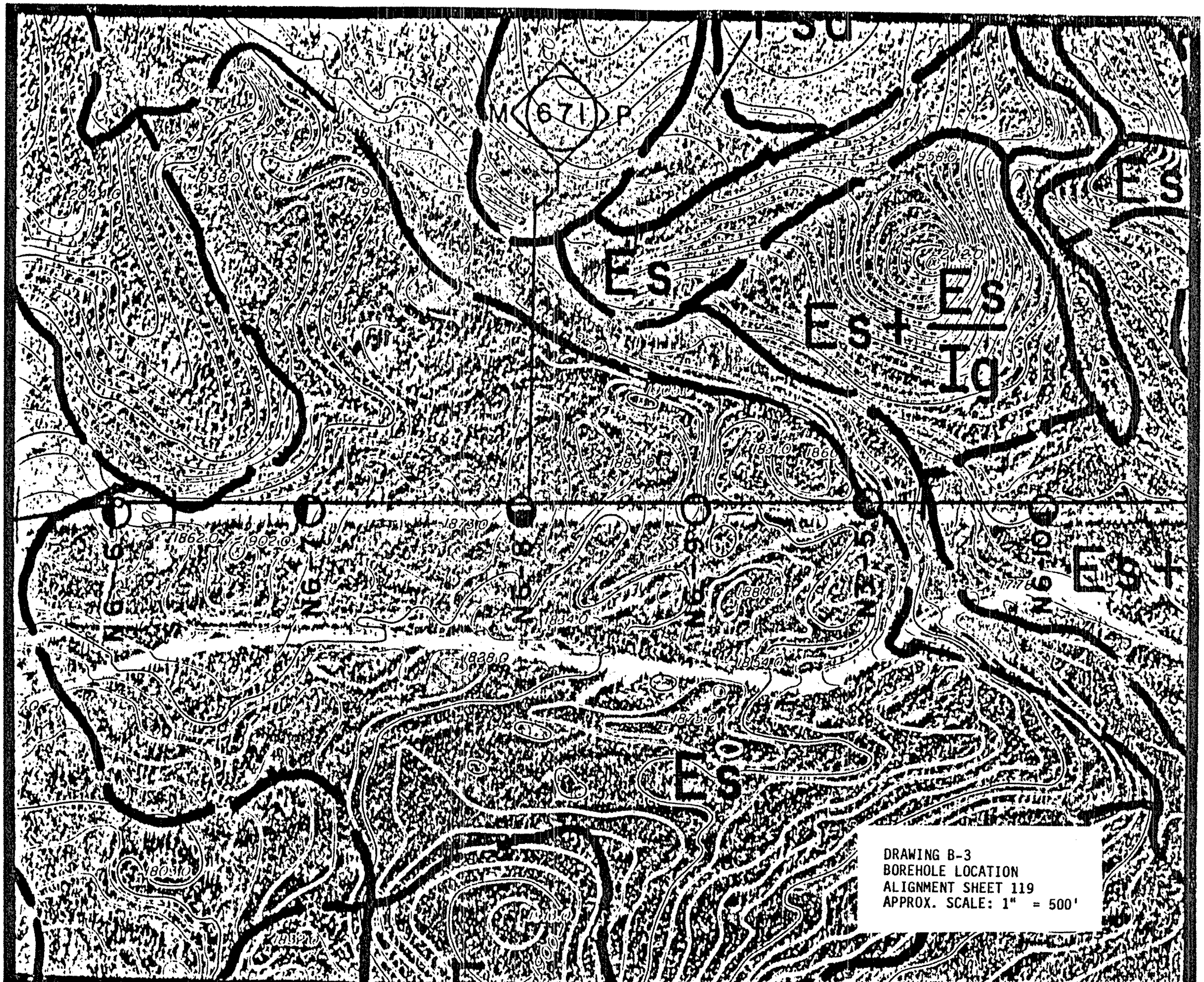
HOZ. 1" = 200'
VERT. 1" = 20'





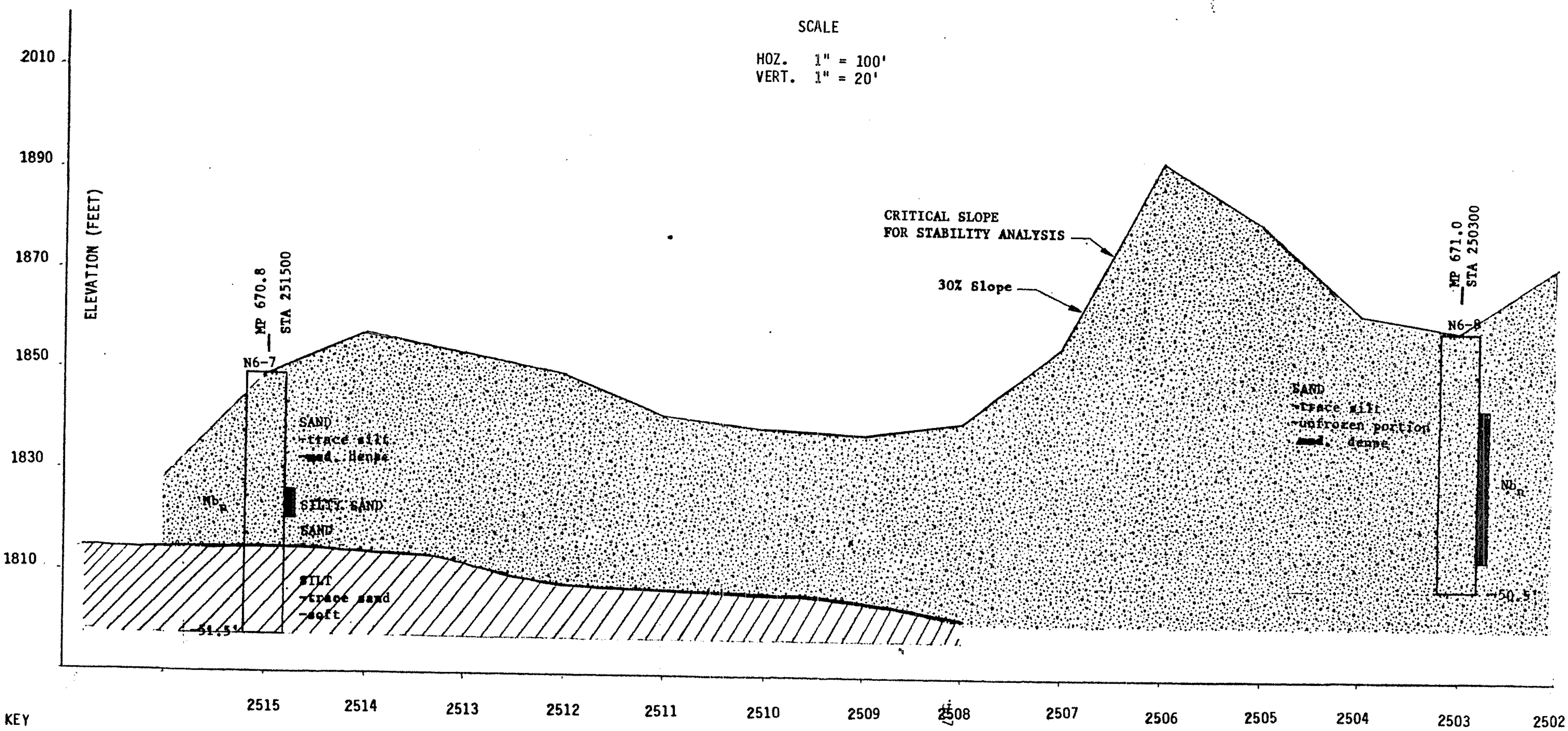
DRAWING B-2

GEOLOGICAL/SOILS 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

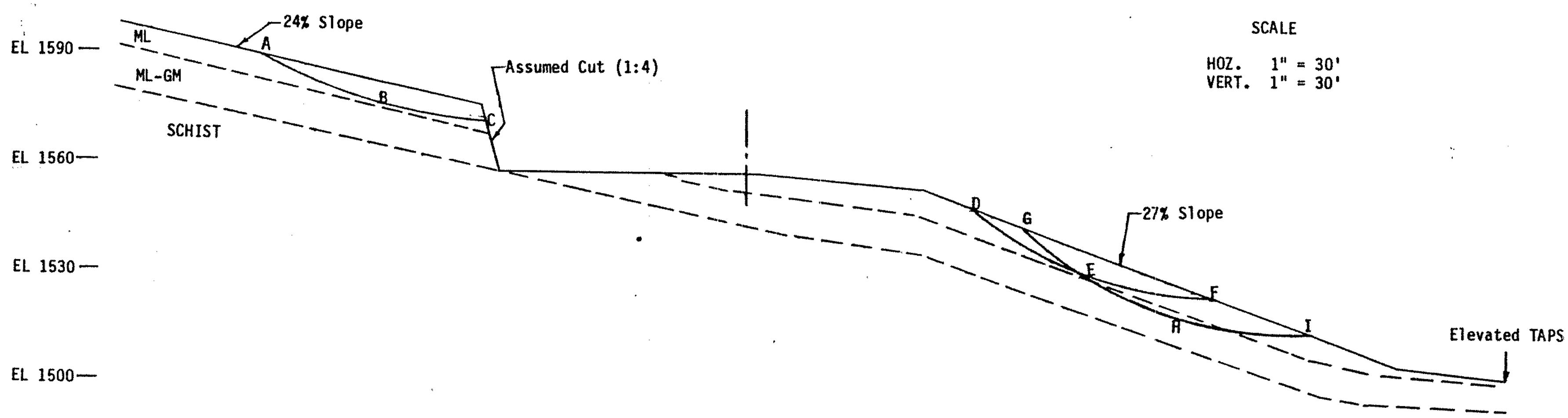
Sand
Silt
Frozen Soil

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