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Unified Industries, Incorporated

December 2, 1981
09-ML-0380
WBS 1.01.09.04

Office of the Federal Inspector
Alaska Natural Gas Transportation System
2302 Martin Drive
Irvine, CA 92715

Attention: Mr. Daniel Muslin
Contracting Officer's Technical Representative

Subject: Comparison of NWA and Alyeska Geotechnical Criteria - Slope Stability

References: (1) OFI "UII Assignment Control" Form No. U-0000152, dated October 16, 1981

(2) OFI "UII Assignment Control" Form No. U-0000153, dated October 17, 1981

Dear Mr. Muslin:

Please find enclosed three (3) copies of our draft report presenting the comparison of the NWA and Alyeska slope stability design criteria. This report is submitted as a partial fulfillment of the task authorized by and conducted in accordance with the above referenced OFI documents.

If you have further questions concerning this subject, please advise.

Sincerely,

Ashraf M. Mirza
Deputy Project Manager

AMM:JR/mlj
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DRAFT REPORT

COMPARISON OF NWA AND ALYESKA GEOTECHNICAL CRITERIA - SLOPE STABILITY

NOVEMBER 1981

CONTRACT NO. OFI 80-0001

WBS 1.01.09.07

Submitted in partial fulfillment of the Technical Support Contractor's authorized task No.s U-0000152 and U-0000153.

A. M. Mirza
Deputy Project Manager

1.0 INTRODUCTION

This report compares the design criteria and analytical procedures for slope stability proposed by the Northwest Alaskan Pipeline Company (NWA) for the Alaska Natural Gas Transportation System, and those that were previously adopted by the Alyeska Pipeline Service Company (Alyeska) for the Trans Alaska Oil Pipeline System (TAPS). The two approaches are summarized in the text while the figures and tables contained in the Appendix assist in illustrating the criteria and procedures which are discussed.

Because of the proximity of the two pipelines, it would be reasonable to expect similar criteria and design standards, particularly if a slope failure on one of the lines could affect the operation or safety of the other.

Criteria and procedures were compared for four categories of slopes:

- Thawing slopes in permafrost
- Unfrozen slopes
- Rock slopes
- Frozen slopes

In addition, criteria for determining the dynamic stability of slopes were also examined.

The thoroughness of the comparison was somewhat limited because reports were not available documenting the actual methods and criteria implemented by Alyeska in the design of the TAPS line. In describing the Alyeska approach, only two documents were available, namely, appendixes to the Criteria and Design Basis Manual, which was prepared in 1973 [5, 6]. Where the design methodology and parameters were altered during the course of the design, it has been necessay to refer to unofficial records and recollections of personnel within the TSC who were involved in the construction of the TAPS line.

2.0 FINDINGS AND CONCLUSIONS

(1) The criteria and analyses proposed by NWA and followed by Aleyska in slope stability assessments are very similar. However, some differences between NWA and Alyeska do exist in how the criteria are applied. The major differences are discussed in this report.

(2) Similar values of minimum safety factors were adopted by both NWA and Alyeska for static and dynamic stability analyses. The minimum safety factors chosen are considered to be within accepted norms.

(3) Procedures for analyzing the dynamic stability of slopes are nearly identical. Alyeska adopted a criterion governing the maximum permissible deformation of a slope as five inches. NWA also advocates the use of a limiting slope deformation criterion but does not state the maximum permissible deformation.

(4) In assessing the stability of frozen slopes which may undergo thaw, Alyeska considered a thaw plug having a width greater than the full width of the right-of-way. In comparison, NWA considers that long term thawing occurs only beneath the work pad. Until more detailed thermal analyses are conducted along the route, it seems premature to limit the thaw plug dimensions to the extent proposed by NWA. if true agree

(5) Although the procedures proposed by both NWA and Alyeska for analyzing thaw plug stability are nearly identical, neither approach adequately accounts for the effect that massive ice will have on generating excess pore pressures at the thaw front. Significance ?

(6) NWA proposes to utilize a relatively wide range of strength parameters for unfrozen and thawed soils based on correlations with dry density. By comparison, Alyeska tended to use single values of strength for respective soil types although the strength parameters for silt did bear some relationship to dry density. NWA may be justified in attempting to be more precise in its analysis by establishing

a correlation between strength and soil density. However, significant discrepancies exist between the strengths corresponding to the upper and lower limits of density and the strengths adopted by Alyeska.

(7) Alyeska adopted a conventional approach in the analysis of the stability of rock slopes. However, no reference to the analysis of rock slope is made in the NWA Pipeline Design Criteria Manual.

(8) Neither NWA nor Alyeska discussed criteria or procedures for establishing the stability of slopes in frozen ground that remains frozen. This appears to be a serious deficiency because about 75 percent of both routes are located in permafrost. Atiqun

(9) Mitigative design measures proposed by NWA and Alyeska to counteract potential slope instability are similar, although NWA refers to more measures than Alyeska does, including the use of insulation to prevent thaw degradation.

3.0 NWA'S METHOD OF ANALYSIS

NWA initially identifies potentially unstable segments along the pipeline route using average soil conditions of each segment and then conducts site-specific analyses of stability. Conditions within each segment are determined from the route soil conditions alignment sheets and from the route geotechnical characterization and classification data.

Initial analyses are performed using stability response design charts which are intended to conservatively represent the terrain and geotechnical conditions. Segments which are thus found to be potentially unstable are re-analyzed on a site-specific basis.

3.1 STATIC ANALYSES

3.1.1 Thaw Plug Stability

(1) General

Disturbances associated with pipeline construction may cause the ground to thaw beneath the pipeline and construction right-of-way. The zone of thaw is referred to by both NWA and Alyeska as a thaw plug.

NWA evaluates the thaw plug stability for two time periods: after two years (short term) and after 25 years (end of pipeline life). The two year period appears to reflect the dormant stage prior to pipeline start-up. In calculating the depth of the thaw plug, it is assumed that the presence of chilled gas in the pipeline does not affect the thermal profile. The analysis also assumes that no insulation is present in the work pad and that the work pad thicknesses presented in the Civil Drawings are valid.

Thaw plugs affecting the integrity of the pipeline must have a safety factor of more than 1.5. Thaw plugs not affecting pipeline integrity must have a safety factor in excess of 1.1. Design mitigation is necessary where the safety factor is below these levels.

(2) Depth and Rate of Thaw

In calculating the depth and rate of thaw, soils are classified into eight different groups based on latent heat. The values of latent heat considered ranged from 370 - 6300 BTU/ft^3. The thaw depths are calculated using eight design charts based on air thaw and freeze indices, and "N" factors reflecting ground surface characteristics. The thaw depth calculated from the design charts are corrected for the effect of the attitude of the slope.

The depth of thaw was calculated using a finite element geothermal program with phase change. Two year and 25 year thaw depths were computed and design graphs prepared for the eight soil groups being considered.

NWA will conduct site-specific calculations of the depth of thaw, where the preliminary analysis indicates that thaw plug instability is a possibility. If the site-specific evaluation confirms that the section could be unstable, a mitigative design would be applied.

(3) Stability Analysis

Thaw plug slope stability analyses are performed using a three dimensional equilibrium model. The limit equilibrium method of analysis, based on an infinite slope model, allows for shearing resistance on three sides of the thaw bulb. A rectangular section is assumed to represent the thaw zone beneath the work pad. The geometry and dimensions of the thaw bulb are shown in Figure 2-1 in the Appendix. A bulk density of 125 PCF is assumed for the work pad material.

For static analyses, effective stress strength parameters are used. Pore pressures due to groundwater seepage or thawing permafrost are estimated and incorporated into the analysis. The shear strength parameters used in thawed soil are presented in Table 2-1 of the Appendix.

Slopes with massive ice and which have a frozen dry density less than 50 PCF are considered as special problem areas, and significant thaw in such areas would not be permitted.

Pore pressures generated at the thaw front are calculated according to the "One-Dimensional Linear Theory of Thaw Consolidation," [1]. The normal force which is applied at the thaw front is reduced by the amount of the pore pressure calculated. The excess pore pressure generated by thaw consolidation is a function of the thaw consolidation ratio, R, which is defined as follows:

$$R = \frac{\alpha}{2\sqrt{c_v}}$$

where: C_v = coefficient of consolidation (ft^2/yr) (mean) ?

α = thaw parameter representing thaw penetration rate ($ft/yr^{1/2}$)

The thaw penetration parameter, α, is considered to range from 3.9 - 12.3 $ft/yr^{1/2}$, with a value of 10 $ft/yr^{1/2}$ adopted for design.

The coefficient of consolidation parameter, C_v, is taken to range from about 250 to 2000 ft^2/yr, although values as low as 3.4 ft^2/yr have been measured for clay. On the basis of experience with Alaskan silts, NWA considers that a C_v of 500 ft^2/yr is conservative in predicting excess pore pressures. By using the quoted "conservative" values of α and C_v, the thaw consolidation ratio, R, is calculated to be approximately 0.2. Only soils having a silt content of more than 80 percent are considered likely to develop excess pore pressure during thaw. Excess pore pressures are calculated for the short term period of two years. No excess pore pressure is considered for the long term period of 25 years or for intermediate periods.

3.1.2 Stability of Unfrozen Slopes

Conventional limit equilibrium methods are used to analyze the stability of slopes in unfrozen ground. Infinite slope, wedge type and circular failure models are all utilized; the particular model chosen depending on the geometrical and geotechnical conditions in the slope.

For single layer systems, failure is assumed to occur at a depth of 50 feet and parallel to the ground surface, when using the infinite slope analysis. For layered systems, failure is considered at the base of each layer and at a depth of 50 feet. The water table is considered at the ground surface, at the middle of the sliding mass, or absent. Effective strength parameters are used for all static analyses.

NWA would utilize effective stress analyses for static slope stability. Proposed shear strength parameters are contained in Table 2-1 of the Appendix. Shear strength is correlated with the dry density of the soil. In some cases, a relatively wide range of strengths is quoted.

3.1.3 Stability of Rock Slopes

NWA does not present criteria relating to the evaluation of the stability of rock slopes.

3.2 DYNAMIC (EARTHQUAKE) ANALYSIS

Stability of slopes under dynamic loading are computed using a pseudo-static analysis method (see Figure 2-2). The design earthquake acceleration, acting in the horizontal direction, increases the driving force and reduces the effective normal stress.

In fine-grained soils of low permeability, a total stress analysis is used to calculate the stability of a thawing slope subjected to dynamic loading. The consolidated-undrained (CU) shear strength is used in the analysis. The minimum factor of safety required under dynamic loading is 1.0. If the safety factor is less than 1.0, an analysis is undertaken to compute the amount of slope deformation that could occur under the design contingency earthquake, according to Newmark's procedure [2]. The tolerable amount of slope deformation is not stated, other than that it would be "less than the acceptable pipe displacement."

In coarse, well drained soils, excess pore pressures are not expected to occur and an effective stress analysis is performed. A minimum dynamic safety factor of 1.0 is required. Three seismic zones are considered, as defined in the companion report on Seismic Liquefaction [4].

In loose, well drained soils, it is considered that stress reversal may cause a 100 percent pore pressure response and accordingly, a slope is considered stable only when the dynamic safety factor is more than 1.0, and the static safety factor exceeds 2.0.

The displacement of a slope during an earthquake is determined from the following equation:

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

where: d = slope displacement
V = maximum ground velocity
N = yield acceleration expressed as a ratio of g
g = acceleration due to gravity
A = maximum ground acceleration expressed as a ratio of g.

3.3 MITIGATIVE DESIGNS

Design mitigation proposed by NWA to counter problems with slope stability are as follows:

- Deep burial below unstable soil, or replacement of the soil
- Insulation to prevent thaw degradation, or ground freezing
- Grading to flatten slopes, and overfill in thaw subsidence areas
- Provide subsurface drainage
- Soil densification, grouting or chemical stabilization
- Slope buttressing
- Construction scheduling (to avoid warm weather construction).

NWA would impose mitigative design measures wherever changes in design mode, construction zone geometry, or alignment cannot be implemented.

4.0 ALYESKA'S METHOD OF ANALYSIS

4.1 STATIC ANALYSES

Alyeska was concerned with evaluating the stability of "significant" slopes only. A significant slope was defined as one that was steeper than 10 percent.

A mile by mile assessment of slope stability parameters was conducted by a task group formed by Alyeska. The parameters were based on the results of laboratory tests and the experience of the task group members.

The pipeline route was divided into units of similar geological and physical characteristics. Slope characteristics were determined and stability charts prepared to plot slope angle against height of slope. Minimum strength parameters interpreted from the stability charts were then compared with parameters determined from laboratory testing.

4.1.1 Thaw Plug Stability

(1) General

As Alyeska was permitted to bury the oil line only in thaw stable soils, thaw plug instability in buried sections of the line was a significant concern. Thaw plug stability problems were anticipated primarily in above ground sections of the pipeline, within fine-grained soils (silts and clays). The behavior of thawing slopes in the field was studied and field charts prepared showing angles and condition of permafrost slopes which were thawing. Shear strength parameters were assessed from the charts. Under static loading condition, the minimum acceptable factor of safety was 1.5.

(2) Depth and Rate of Thaw

Initially, Alyeska assumed an average thaw penetration of 4.0 feet/year $^{1/2}$. This approach was subsequently modified so that thaw during a single year would be concentrated in only five months, thereby increasing the thaw rate and capacity to generate excess pore pressure. The approach resulted in the adoption of a somewhat larger thaw penetration rate of 6.2 ft/yr$^{1/2}$.

The thaw plug was estimated to reach a depth of 20 feet in ice-rich soils and 30 feet in ice-poor soils over a 30 year period. The width of the thaw bulb was assumed to be 120 feet. Alyeska considered the thaw plug depth after 30 years to be the critical case governing stability.

(3) Stability Analysis

The physical model adopted for the stability analysis for thawing permafrost, was an infinitely long cylinder, as illustrated in Figure 2-3 in the Appendix. The piezometric level at the thaw front was initially considered to correspond to the upper surface of the fine grained deposit, or to be equivalent to the groundwater level in an overlying free draining deposit. A linear distribution of hydraulic gradient was assumed with depth. This approach was later replaced by one based on the prediction of pore pressures, using a non-linear thaw consolidation theory [1]. This is the same theory that NWA is proposing to use in the analysis of thaw plug stability.

A coefficient of consolidation parameter, C_v, of 500 ft^2/yr was adopted for silts having a dry density of about 60 PCF.

Alyeska found that frozen silt having a dry density of 60 PCF and a permeability of at least 0.4 x 10^{-4} cm/sec would not develop significant excess pore pressure while thawing.

Shear strength parameters adopted in the stability analysis are shown in Table 2-2 of the Appendix.

4.1.2 Stability of Unfrozen Slopes

Conventional slip circles, or infinite slope analyses were used, unless evidence indicated that a more complex failure mechanism could occur.

For clays or clayey silts having a dry density of less that 90 PCF, consolidated undrained (CU) shear strength parameters were used in a total stress analysis. For clays denser that 90 PCF, drained shear strength parameters were adopted. Drained shear strength parameters were used for permeable sands and gravels in the analysis of both short term and long term stability.

Shear strength parameters for unfrozen soils are shown in Table 2-2, in the Appendix.

4.1.3 Stability of Rock Slopes

Alyeska analyzed the stability of rock slopes by assessing the orientations of discontinuities in the rock and by comparing the available shearing resistance along three dimensional failure surfaces with the driving forces. In at least part of the route, the method recommended by Hendron, et. al., [3] was adopted in the analysis of stability. This approach is conventional and widely accepted.

4.2 DYNAMIC (EARTHQUAKES) ANALYSIS

The factor of safety of a slope was evaluated for motions produced under the design contingency earthquake within the respective seismic zones a pseudo-static method of analysis was used.

A slope was considered stable if the dynamic factor of safety was 1.0 or more. If the dynamic safety factor was less than 1.0, Newmark's procedure [2] was applied to estimate the permanent slope displacement under the design contingency earthquake. The maximum acceptable displacement was taken as five inches.

For earthquake loading of thawed clay and clayey silt, consolidated undrained (CU) shear strength parameters were used. In coarse, free draining soils an effective stress stability analysis was performed. If it could be shown that the soil would not liquefy during the design earthquake, slope stability was analyzed using the same shear strength parameters as for the static stability analysis.

The maximum displacement during an earthquake was computed by the relation:

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

where: d = slope displacement
V = maximum ground velocity
g = acceleration due to gravity
A = maximum ground acceleration expressed as a ratio of g
N = yield acceleration expressed as a ratio of g.

4.3 MITIGATIVE DESIGNS

Mitigative design measures proposed by Alyeska to counter problems with slope instability were as follows:

- Pipeline re-routing
- Flatten cut or fill slopes
- Change construction mode, such as using thermal piles, elevated pipeline construction, refrigerated burial, or winter construction with a snow work pad
- Deep bruial in rock beneath unstable surficial soils.

5.0 COMPARISON OF SLOPE STABILITY CRITERIA AND PROCEDURES

A comparison of the main elements of slope stability criteria adopted by NWA and Alyeska is made in Table 2-3 of the Appendix. The comparison is also described in the following paragraphs.

5.1 FACTORS OF SAFETY

Both NWA and Alyeska have adopted essentially the same factors of safety for slope stability. For static analyses, the minimum safety factor is 1.5, although NWA accept a safety factor in the thaw plug as low as 1.1, where the pipeline integrity is not affected.

Both NWA and Alyeska have adopted a minimum dynamic factor of safety of 1.0. If the safety factor is less than 1.0, an analysis of slope deformation must show that displacement will not exceed the pipe tolerance.

5.2 THAW PLUG STABILITY

The NWA and Alyeska approaches are quite similar, being based on effective stress analyses of stability, wherein excess pore pressures at the thaw front are predicted using the "Theory of Thaw Consolidation."

Both NWA and Alyeska have adopted similar generalized parameters for thaw rate (α) and coefficient of consolidation (C_v). Therefore, it is anticipated that similar results would be found when analyzing a thawing slope with the two procedures.

Differences between NWA and Alyeska are primarily in the application of the analytical procedures rather than in the types of procedures adopted. NWA assumes that only two thaw depths need be considered: the thaw depths after two years and after 25 years. In calculating the thaw depths, NWA's method appears more sophisticated than Alyeska's, as it can account for meteorological, geothermal, and topographic conditions on a site-specific basis, whereas Alyeska tended to use more generalized parameters. In spite of the apparently better capability of the NWA procedure, it does not take into account the effect that ice segregations would have on pore pressure generation.

Alyeska attached some importance to observing and measuring the performance of thawing slopes in the field. Empirical correlations between field observations and theoretical predictions were attempted. NWA does not refer to such correlations.

In the aboveground section of the TAPS line, Alyeska assumed that a thaw plug would develop to a width of 120 feet, which is in excess of the full width of the right-of-way. By contrast, NWA indicates that a thaw plug will develop only beneath the work pad (Figure 2.1, Appendix). Evidently, NWA feels that the frost bulb which is formed around the pipeline will prevent thawing beneath much of the right-of-way.

5.3 STABILITY OF UNFROZEN SLOPES

Both NWA and Alyeska utilize conventional limit equilibrium methods, choosing the particular method according to the geometric boundaries and geotechnical conditions.

Drained strength parameters are used by for effective stress analyses of slopes in soils that drain freely. Where there is no free drainage (NWA is contradictory in its approach), it appears that both NWA and Alyeska use total stress analyses, incorporating consolidated undrained strength parameters.

A comparison of the shear strength parameters proposed by NWA and Alyeska is made in Table 2-4 in the Appendix. NWA varies the strength parameters according to the dry density of the deposit. By comparison, Alyeska often used single values of a parameter, regardless of the dry density. Alyeska indicated that dry density was not a controlling factor. For example, NWA indicates a range of effective friction angle $\phi' = 18\text{-}45°$ and cohesion C' of 120-450 PSF for a drained soil with less than 12 percent fines. The range applies for dry densities varying from 50-130 PCF. For a similar soil (sand), Alyeska used $\phi' = 30°$ and $C' = 0$.

5.4 STABILITY OF ROCK SLOPES

Alyeska adopted a conventional procedure for establishing the stability of slopes in rock. In the Design Criterial Manual submitted by NWA, the subject of rock slope stability is not discussed and hence a comparison cannot be made with the Alyeska procedure.

5.5 STABILITY OF FROZEN SLOPES

Neither NWA nor Alyeska addresses the analysis of slope stability in frozen soils. Alyeska did indicate that where excessive displacement was expected in thawing slopes, thermal piles would be utilized to maintain the frozen conditions (with respect to solifluction and creep).

5.6 DYNAMIC ANALYSIS

Both Alyeska and NWA follow essentially the same procedures in establishing the dynamic stability of slopes. Both companies use a pseudo-static analysis, incorporating a horizontal load to represent the force of the earthquake. For a dynamic factor of safety less than 1.0, the Newmark procedure to calculate the displacement of the slope is used [2].

Alyeska established the limiting amount of slope deformation during an earthquake as five inches. NWA has not stated what the allowable displacement will be for the gas pipeline.

In loose, well drained soils, NWA appears to be proposing a different criterion for dynamic stability than in other soils. Although the wording is ambiguous, it appears that for a slope to be deemed stable in such soils, the safety factor under static loading conditions must be at least 2.0. Alyeska did not distinguish this type of soil in establishing its dynamic stability criteria.

6.0 REFERENCES

[1] Morgenstern, N. R., and Nixon, J. F., "One Dimensional Consolidation of Thawing Soils," Canadian Geotechnical Journal, 8, P. 558-565, 1971.

[2] Newmark, N. M., "Effects of Earthquakes on Dams and Embankments," Geotechnique, P. 139-160, June, 1965.

[3] Hendron, A. J. Cording, E. J., and Aiyer, A. K., "Analytical and Graphical Methods for the Analysis of Slopes in Rock Masses," NCG Technical Report No. 36, July 1971.

[4] Unified Industries, Inc., "Comparison of NWA and Alyeska Geotechnical Criteria - Liquefaction," Contract No. OFI 80-0001, WBS 1.01.09.07, November, 1981.

[5] Alyeska Pipeline Service Company, "Criteria and Design Bases, Appendix A-3.1079, Summary Report - Geotechnical Aspects of the Trans Alaska Pipeline," September, 1973.

[6] Alyeska Pipeline Service Company, "Criteria and Design Bases, Appendix A-3.1010, Summary Report - Thermal Analysis for the Trans Alaska Pipeline System," June, 1973.

[7] Northwest Alaskan Pipeline Company, "Pipeline Design Criteria Manual, Geotechnical Assessment - Volume No. 4," prepared by Fluor Engineers and Constructors, Inc., May, 1981.

APPENDIX A

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TABLES AND FIGURES

TABLE 2-1

NWA 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.

[From Reference 7]

TABLE 2-2. ALYESKA SHEAR STRENGTH PARAMETERS

Material Description	Static Slope Stability: Drained (S) Shear Strength Parameters: Angle of Shearing Resistance $\bar{\phi}$ (degree)		Cohesion Intercept $\bar{c}$ (psf)		Consolidated-Undrained (R) Shear Strength Parameters: Angle of Shearing Resistance ϕ (degrees)	Cohesion Intercept c (psf)	Dynamic Slope Stability: Controlling Drainage Condition	Angle of Shearing Resistance ϕ (degrees)	Cohesion Intercept c(psf)	Dry Unit Weight γ_d (pcf)
	$\bar{\sigma} \leq 2$ ksf	$\bar{\sigma} > 2$ ksf	$\bar{\sigma} \leq 2$ ksf	$\bar{\sigma} > 2$ ksf						
Alluvium										
a) Gravel-Sand Mixtures	40	40	0	0	NCD	NCD	DL	40	0	NCF
b) Sand	35	35	0	0	NCD	NCD	DL	35	0	NCF
Till or Moraine *	45	45	0	0	NCD	NCD	D	45	0	NCF
Colluvium										
a) Gravel-Sand Mixtures	40	40	0	0	NCD	NCD	DL	40	0	NCF
b) Silty Sand	35	35	0	0	NCD	NCD	DL	35	0	NCF
Talus	35	35	0	0	NCD	NCD	D	35	0	NCF
Low Density Silt ML	NCD	NCD	NCD	NCD	11	250	CU	11	250	< 70
Medium Density Silt ML	NCD	NCD	NCD	NCD	13	250	CU	13	250	$70 < \gamma_d < 90$
High Density Silt, ML	35	35	0	0	NCD	NCD	CU	13	700	> 90
High Density Clay, CL, CH (Copper River Basin Clay)	35	35	0	0	NCD	NCD	CU	20	700	> 90

Notes:

1. NCD indicates a non-controlling drainage condition
2. NCF indicates that dry unit weight is not considered a controlling factor
3. DL indicates that drained shear strength parameters control dynamic slope stability analysis when soil involved is not saturated, but liquefaction considered to be more critical when deposit is saturated.
4. D indicates that drained parameters should control dynamic slope stability of such dense and basically coarse-grained soils
5. CU refers to consolidated-undrained shear strength parameters which should control the dynamic slope stability of such fine-grained soils

* These soils were considered very dense by Alyeska.

TABLE 2-3. COMPARISON OF NWA AND ALYESKA SLOPE STABILITY CRITERIA AND PROCEDURES

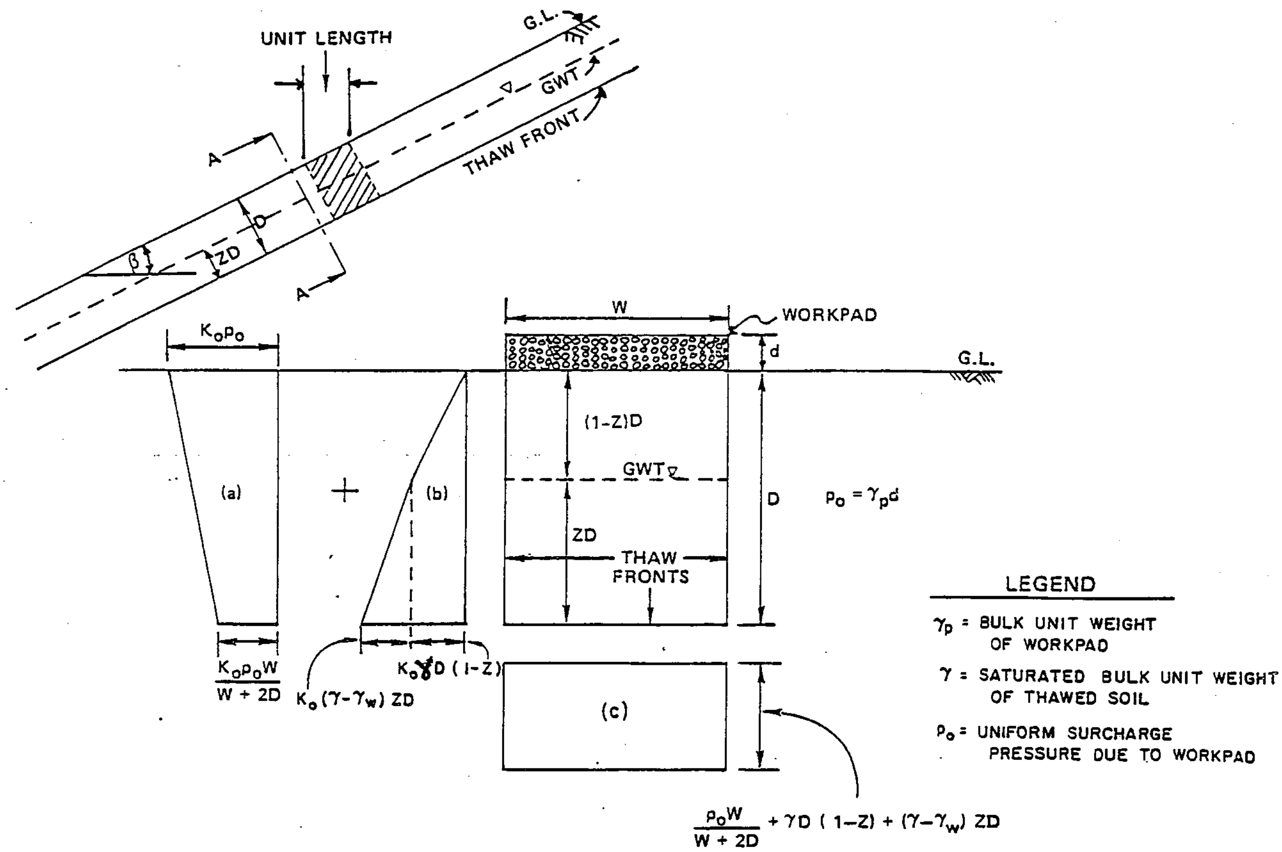
ITEM	NWA	ALYESKA
Minimum Static Safety	1.5 for slopes affecting pipeline integrity; 1.1 for other slopes.	1.5
Minimum Dynamic Safety Factor	1.0, or if <1.0, limit slope displacement to unspecified amount.	1.0, or if <1.0, limit slope displacement to maximum five inches.
Thaw Plug Stability Method	Infinite slope stability. Effective stress analysis, predict pore pressures using thaw consolidation theory.	Same, also use empirical correlations with thawing slopes along pipeline route.
Unfrozen Slope Stability Method	Limit equilibrium methods - include infinite slope, circular and wedge models; effective stress strength parameters for all static analyses. Strength parameters vary widely according to dry density.	Similar, but used consolidated undrained strength parameters for static analysis of low and medium density silt. Dry density less of a factor in assessing strength parameter.
	Use consolidated undrained strength for poorly drained soils in dynamic analysis.	Where soil is not free draining, used consolidated undrained strength in dynamic analysis.

TABLE 2-3. COMPARISON OF NWA AND ALYESKA SLOPE STABILITY CRITERIA AND PROCEDURES (CONT'D)

ITEM	NWA	ALYESKA
Rock Stability	No procedure is given.	Map orientation of geologic discontinuities; define failure boundaries and calculate minimum shearing resistance necessary for equilibrium.
Frozen Slope Stability	No procedure is given.	No procedure was given.
Dynamic Stability	For safety factor >1.0, used pseudo static analysis, incorporating a horizontal force based on % gravity acceleration. For safety factor <1.0, calculate slope displacement using Newmark's method.	Same.

TABLE 2-4. COMPARISON OF SHEAR STRENGTH PARAMETERS (PARTIAL)

SOIL TYPE	DRY DENSITY PCF	DRAINED PARAMETERS NWA ϕ' deg	DRAINED PARAMETERS NWA C' PSF	DRAINED PARAMETERS ALYESKA ϕ' deg	DRAINED PARAMETERS ALYESKA C' PSF	CONSOLIDATED UNDRAINED PARAMETERS NWA ϕ_{cu} deg	CONSOLIDATED UNDRAINED PARAMETERS NWA C_{cu} PSF	CONSOLIDATED UNDRAINED PARAMETERS ALYESKA ϕ_{cu} deg	CONSOLIDATED UNDRAINED PARAMETERS ALYESKA C_{cu} PSF
Silt or Poorly Drained Soil	50 - 130 Unfrozen					6-33°	100-1400		
	70 - 140 Thawed					6-33°	100-1900		
	> 90 Static Load			35	0				
	> 90 Dynamic Load							13	700
	70 - 90 Static and Dynamic Loads							11-13	250
Sand	50 - 130 Unfrozen	18-45	120-450						
	70 - 140 Thawed	25-45	120-510						
	All Densities			35	0				
Sand and Gravel	50 - 130 Unfrozen	25-46							
	70 - 140 Thawed	25-48							
	All Densities			40	0				



$$SFS = \frac{1}{1 + X}\left[\left\langle \frac{K_o D}{W}\left(1 - \frac{\gamma_w}{\gamma} Z^2\right) + \left(1 - \frac{\gamma_w}{\gamma} Z\right)\right\rangle + \left(\frac{X}{W/D + 2}\right)\left\langle 2K_o\left(1 + \frac{D}{W}\right) + \frac{W}{D}\right\rangle\right]\frac{\tan\phi}{\tan\beta}$$

$$= \frac{\tan\beta_{SFS=1.0}}{\tan\beta}$$

Where: $X = \frac{\gamma_\rho d}{\gamma D}$

β = slope angle in degrees

γ = saturated bulk unit weight of the thawed soil

γ_ρ = bulk unit weight of the work pad

D = depth of thaw bulb normal to the slope

W = width of the thaw bulb

d = work pad thickness

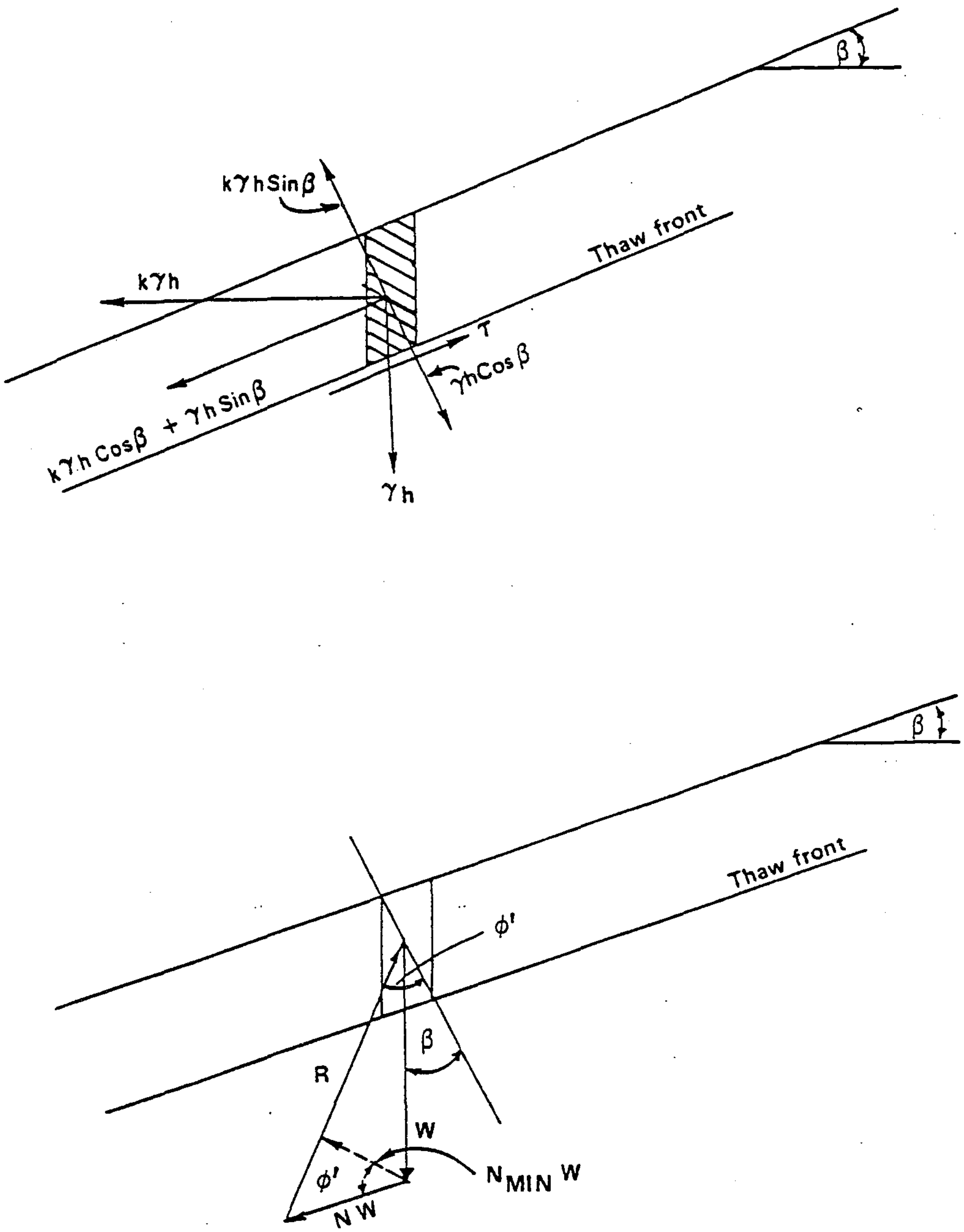
K_O = coefficient of lateral earth pressure at rest

γ_W = unit weight of water

Z = groundwater table factor

$\beta_{SFS=1.0}$ = slope angle for static factor of safety equal to 1.0 with strength parameter C = 0.

FIGURE 2-1. LIMIT EQUILIBRIUM METHOD FOR THAW PLUG STABILITY ANALYSIS. (NWA)



$$DFS = (SFS)\,\frac{\tan \beta}{\tan \beta + k} - \frac{k \tan \phi'}{1 + k \cot \beta}$$

Where: β = slope angle in degrees

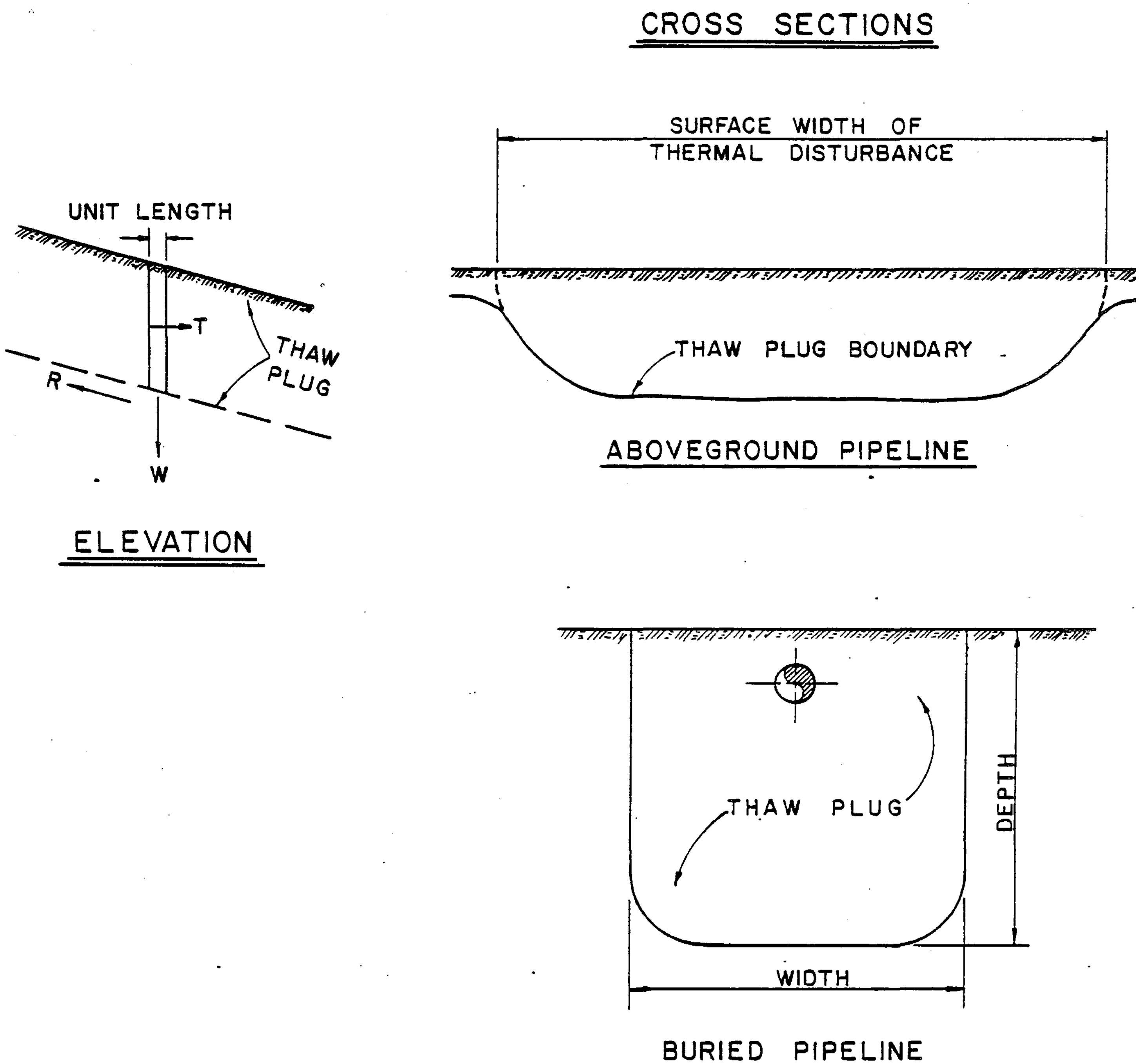
DSF = dynamic factor of safety

SFS = static factor of safety

k = seismic coefficient

ϕ' = effective friction angle

FIGURE 2-2. PSEUDO-STATIC SLOPE STABILITY ANALYSIS FOR EARTHQUAKE LOADING.



DEFINITIONS

T = FORCE ACTING ON UNIT LENGTH OF THAW PLUG DUE TO EARTHQUAKE.
W = WEIGHT OF UNIT LENGTH OF THAW PLUG.
R = AVAILABLE SHEAR RESISTANCE FOR UNIT LENGTH OF THAW PLUG.

FIGURE 2-3. MODELS FOR HYDRAULIC AND MECHANICAL ANALYSES
[From Reference 5]