PLANS
FOR DEALING WITH
SLOPE INSTABILITY
SUBMISSION 4-3

NOVEMBER, 1981

ADDENDUM
TO
THE ENVIRONMENTAL IMPACT STATEMENT
FOR
THE YUKON SECTION
OF
THE ALASKA HIGHWAY GAS PIPELINE

THE ALASKA HIGHWAY GAS PIPELINE PROJECT

Foothills Pipe Lines (South Yukon) Ltd.

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This document is one of a series of addenda prepared to meet information requirements placed on Foothills Pipe Lines (South Yukon) Ltd. by the Federal Environmental Assessment and Review Office. Addenda within the series are divided into seven sets of submissions dealing with separate subject areas:

1. Introduction to Addenda Submissions.

2. Project Description and Update for Addenda Submissions.

3. Alternative Routes.

4. Geotechnical, Hydrological, Design Mode and Revegetation Issues.

5. Fisheries, Wildlife and Scheduling Issues.

6. Issues Related to Pipeline Facilities.

7. Other Issues.

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1.0 INTRODUCTION

The matter of slope instability and consideration of it is an integral part of pipeline design. As a result, slope instability has been the topic of extended discussion between regulatory agencies and Foothills Pipe Lines (South Yukon) Ltd. during the application and planning stages for the Alaska Highway Gas Pipeline Project.

In 1976, the Project addressed the matter of slope instability at a preliminary level in its application to the National Energy Board. Material submitted included a discussion of physiographic regions traversed by the pipeline, an account of surficial geology along the route and a brief description of seismic conditions that were anticipated.

After review of the information supplied in the Project's application to the National Energy Board, the Environmental Assessment and Review Panel noted a number of concerns and information deficiencies related to slope instability arising from seismic activity, erosion and from induced frost-heave or thaw settlement, the latter arising from pipeline operation in areas of intermittent permafrost.

In 1979, the Project prepared and submitted an Environmental Impact Statement (EIS) and included in that document additional information related to slope instability. This information included further description of physiographic regions traversed by the pipeline, as well as description of surficial geology, geological features and geological processes occurring along the route. Details of terrain features were presented in tabular form with each successive terrain type traversed noted together with the lineal distance of traverse, composition and stratigraphy of the deposit, permafrost conditions, relief, slope, drainage, depth to bedrock, underlying bedrock type, borrow potential and special engineering conditions. In sections of the EIS dealing with design, instability arising from potential geothermal effects of pipeline construction and operation were dealt with through a discussion of available geotechnical information and of preliminary designs to overcome problems associated with both warm and cold gas flow. Instability arising from seismic activity was addressed separately.

Prior to hearings in 1979, the Project supplied additional information in response to deficiency questions. Information supplied during this process, relating to slope instability included:

1. a review of methods of stability analysis for permafrost terrain;

2. a response to requests for the design of rip rap which referenced standard texts on the subject; and

3. information regarding the potential for liquefaction in Kluane Lake sediments during seismic events, plus preliminary information on the nature of Kluane Lake sediments based on the results of field programs.

After review of material submitted in the EIS, of that in deficiency responses and of statements made during hearings, the Panel asked for further detailed information on methods of slope stabilization, particularly with respect to the influence of frost-heave and thaw settlement. In addition, the Panel asked for further details for seismic risks to the pipeline and information on Kluane Lake sediments. Subsequently, these requests were clarified to include "design concepts and discussion of impacts...... particularly where there is a lack of field data".

This addenda report is being submitted to meet the most recent "clarified" requirements of the Panel.

It consists of a consultant report prepared for submission to the Northern Pipeline Agency on the matter of slope stability.

The report contains two sections. The first is a description of "an analytical program" developed to draw together pertinent geotechnical information and to ensure that proper analysis of this information results in a secure pipeline installation. This program also includes the recognition and prevention of pipeline-induced instability. A second section involves "seismic liquefaction studies" aimed at ensuring that all elements of the pipeline are designed to avoid soil liquefaction hazards arising from seismic events.

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ALASKA HIGHWAY GAS PIPELINE PROJECT

(YUKON SECTION)

PROCEDURES FOR
SLOPE STABILITY ANALYTICAL PROGRAM
AND SEISMIC LIQUEFACTION STUDIES

Prepared For

FOOTHILLS PIPE LINES (SOUTH YUKON) LTD.

By

HARDY ASSOCIATES (1978) LTD.

and

ROBINSON DAMES & MOORE

January, 1982

K5501E

HARDY ASSOCIATES (1978) LTD.
CONSULTING ENGINEERING & PROFESSIONAL SERVICES

Our Project No CG05501E
Your Reference No

January 29, 1982

Foothills Pipe Lines (South Yukon) Ltd.
Esso Plaza, East Tower
425 - 1 St. S.W.
Calgary, Alberta
T2P 3L8

Attention: Mr. J.R. Ellwood, P.Eng.,
Manager, Geotechnical Services

Dear Sir:

Re: Alaska Highway Gas Pipeline Project (Yukon Section) - Procedures for Slope Stability Analytical Program and Seismic Liquefaction Studies

We are pleased to enclose herewith twenty-five (25) copies and one (1) unbound copy of a report describing the procedures that will be utilized to complete the slope stability analytical program and seismic liquefaction studies for the Yukon Section of the subject project. The former is given in Part I of the report, while the latter constitutes Part II and was prepared by Robinson Dames & Moore.

The report contains information specific to design criteria, input parameters and analytical methods. The computer programs that will be utilized have been verified and these verifications are also included. Mitigative measures that may be employed to improve the safety factor in the slope are also discussed.

The results of detailed slope stability and seismic liquefaction studies will be presented in separate reports, for each construction section.

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GEOTECHNICAL MATERIALS & METALLURGICAL ENGINEERS —
CALGARY DAWSON CREEK EDMONTON ELKFORD FORT McMURRAY LETHBRIDGE MEDICINE HAT PRINCE GEORGE RED DEER SASKATOON VANCOUVER WINNIPEG

If you have any further questions or comments regarding the report, I would be pleased to meet with you at your convenience to discuss them.

We appreciate having the opportunity to participate in this project.

Yours truly,

HARDY ASSOCIATES (1978) LTD.

Per: J-Marie Chevallier

J-M. Chevallier, M.Eng.,P.Eng.

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EXECUTIVE SUMMARY

The South Yukon portion of the Alaska Highway Gas Pipeline Project is approximately 830 kilometres long and extends from the Alaska Yukon border (K.P. 0), near Beaver Creek, to the Yukon-British Columbia border (K.P. 830), near Watson Lake. For construction scheduling purposes, these 830 kilometres have been subdivided into thirteen sections ranging in length from 54 to 79 kilometres.

The first 379 kilometres would consist of 1219 mm diameter pipe and the remaining 451 km of 1422 mm pipe. Three design construction modes have been adopted to accommodate the various soil conditions encoutered along the route. The Standard and Deep Burial modes would be implemented where the depth to thaw/frost stable soils is relatively shallow. The above grade mode would be utilized where settlement would cause excessive pipe movements.

In certain locations, the pipeline route traverses slopes that are longitudinal and/or perpendicular to the pipe, and potentially liquefiable areas. Slopes steeper than 14 percent (8^{o}) are identified in the slope catalogue, and a sample of this catalogue is presented in Appendix A.

This report discusses procedures that will be utilized to achieve the objectives of the Slope Stability Analytical Program (Part I) and complete the Seismic Liquefaction Studies (Part II). The analytical methods are reviewed in detail and all the equations or procedures that will be utilized in slope stability analyses and seismic liquefaction studies are presented. The slope stability and seismic liquefaction computer programs are described and verification of these programs is also given.

The results of slope stability analyses and seismic liquefaction studies together with general and specific construction recommendations will be presented in separate reports. Recommendations on specific mitigative measures will also be included, as appropriate.

This work was conducted under the direction of Mr. J.R. Ellwood, P.Eng., Manager, Geotechnical Services, of Foothills Pipe Lines (South Yukon) Ltd. Hardy Associates (1978) Ltd. prepared the slope stability analytical program procedures (Part I), and Robinson Dames & Moore prepared the seismic liquefaction studies (Part II).

PART 1

SLOPE STABILITY ANALYTICAL PROGRAM

1.0 INTRODUCTION

1.1 General

The pipeline route includes slopes that are longitudinal and/or perpendicular to the pipe, and their instability may affect pipeline operation. In addition, movement of slopes comprising sections of the right-of-way or skin flow of natural slopes near the right-of-way may hinder access to the pipeline. It follows that a geotechnical assessment of the stability of slopes along the route is required.

Failure of a previously stable slope may result from the following:

1. Change in static equilibrium due to construction operations and/or variations of the groundwater conditions;

2. Earthquake loading; and

3. Liquefaction of the materials constituting the slope.

This latter mode of failure is considered in the seismic liquefaction studies described in Part II of this report.

1.2 Objective of the Study

The slope stability analytical program has been engineered so that:

1. The integrity of the pipeline structure and support system is maintained against all adverse potential ground movement during and subsequent to construction.

2. Any adverse effects of construction and pipeline operation on the natural slopes can be recognized and prevented.

1.3 Report Organization

Section 1.0 relates Part I and Part II of this report and outlines the objectives of the study.

Section 2.0 discusses the rationale followed to meet the objectives of the study and explains the importance and utilization of the background data.

The classification of slopes based on the geometry of the pipe and soil state, is described in Section 3.0.

The design criteria set to successfully achieve the objectives of the study are summarized in Section 4.0.

Input parameters, common to all slope stability analyses, are reviewed in Section 5.0 in conjunction with the factors affecting them.

Section 6.0 provides a detailed discussion of the analytical methods utilized to determine the stability of slopes along the pipeline route.

A complete description of Hardy Associates (1978) Ltd. slope stability computer program is given in Section 7.0, and verification of the program is also included.

Section 8.0 describes various mitigative measures that may be employed to augment the stability of a slope, if required.

2.0 OVERVIEW OF DESIGN PROCESS

2.1 Field Reconnaissance Program

During 1976 to 1980 the pipeline route has been flown by experienced Arctic geotechnical personnel in order to select an alignment that would present minimal slope problems. Subsequently, prior to completion of the preliminary geotechnical assessment, a field reconnaissance program was undertaken by the slope design consultants to obtain a better understanding of the general topography and perform a visual evaluation of all slopes along the route. Particular emphasis was placed on slopes where:

1. No detailed design was considered necessary;
2. Special design and construction considerations would be advisable.

The field reconnaissance program in conjunction with slope and subsurface conditions will form an intergrated part of the preliminary geotechnical assessment indicated in the slope catalogue, given in Appendix A. Areas where more subsoil information is required have been isolated and field investigations are presently underway.

2.2 Slope Catalogue

The information summarized on the slope catalogue is presented on Figure 1, and this information was taken from the Foothills Pipe Lines Geotechnical Atlas. The definition of symbols utilized together with a sample of the slope catalogue is enclosed as Appendix A.

All terrains comprising the slope have been listed under the heading of "terrain type" in the slope catalogue shown in Appendix A.

BACKGROUND INFORMATION																		PRELIMINARY GEOTECHNICAL ASSESSMENT	
SLOPE NO.	A.S. NO.	K.P.	SLOPE CONDITIONS							EXISTING DATA BASE			F OR U	TERRAIN TYPE	SUBSURFACE STRATIGRAPHY	GROUNDWATER CONDITIONS	PROPOSED DESIGN MODE	DESIGN PROBLEMS	REQUIRED ANALYSIS
			ALONG ℄			CROSS ℄			ASPECT AZIM.										
			AVE (MAX) %	LENGTH (m)	HT (m)	AVE (MAX) %	LENGTH (m)	HT (m)		BOREHOLES	TESTS	INSTR.							
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

1. SLOPE NUMBERING SEQUENCE (V-I: section - slope number)
2. ALIGNMENT SHEET WHERE THE SLOPE IS LOCATED
3. KILOMETER POST AT START OF SLOPE OR PIPE AFFECTED
4. AVERAGE (MAXIMUM) SLOPES, IN PERCENT ALONG CENTRELINE OF PIPE. SLOPE IS POSITIVE WHEN SLOPE HEIGHT IS INCREASING CONCOMMITANTLY WITH CHAINAGE.
5. LENGTH OF SLOPE IN METRES MEASURED HORIZONTALLY
6. HEIGHT OF SLOPE IN METRES
7. AVERAGE (MAXIMUM) SLOPES, IN PERCENT, ACROSS CENTRELINE OF PIPE. SLOPE IS POSITIVE WHEN, LOOKING IN DIRECTION OF INCREASING CHAINAGE, THE SLOPE IS UPHILL TO THE RIGHT.
8. LENGTH OF PIPE THAT MAY BE AFFECTED BY THE SLOPE, IN METRES.
9. HEIGHT OF SLOPE IN METRES
10. AZIMUTH OF THE DOWNHILL DIRECTION OF THE SLOPE, IN DEGREES.
11. DESIGNATION OF AVAILABLE BOREHOLES PERTINENT TO THE SLOPE.
12. SUMMARY OF SPECIALIZED LABORATORY TESTING CONDUCTED ON SAMPLES OBTAINED AT THE BOREHOLE LOCATIONS.

    TS- THAW STRAIN DS- DIRECT SHEAR Qu - UNCONFINED COMPRESSION
    TXCU- TRIAXIAL CONSOLIDATED-UNDRAINED TXUU- TRIAXIAL UNCONSOLIDATED-UNDRAINED

13. INSTRUMENTATION PLACED AT THE BOREHOLE LOCATIONS.

    T-THERMISTOR S-STANDPIPE PIEZOMETER PC- CASAGRANDE PIEZOMETER
    P- PNEUMATIC PIEZOMETER EMR - ENERGY MINES RESOURCES THERMISTOR
    N- NO INSTRUMENTATION

14. SOIL CONDITION : FROZEN OR UNFROZEN
15. TERRAIN TYPE BASED ON AIRPHOTO INTERPRETATION. FOR DETAILS REFER TO GEOTECHNICAL ATLAS.
16. GENERALIZED SUMMARY OF APPLICABLE BOREHOLE LOGS.
17. GROUNDWATER CONDITIONS:

    (M)- MEASURED FROM PIEZOMETERS (E)-ESTIMATED
    (A)- ESTIMATED AVERAGE BASED ON PIEZOMETER READINGS

18. PIPE DESIGN MODE
19. IDENTIFICATION OF POTENTIAL FAILURE MECHANISMS, PRIMARY AND SECONDARY (IF APPLICABLE), AND RECOGNITION OF THE ADVERSE EFFECTS ON THE PIPE.
20. METHOD OF ANALYSIS EMPLOYED TO MODEL THE IDENTIFIED POTENTIAL FAILURE MECHANISM.

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DEFINITION OF SYMBOLS USED IN
SLOPE CATALOGUE INFORMATION SHEETS

No. K 5501 - E

Fig 1

A review of previous geotechnical investigation reports in the period 1976-1980 by Hardy Associates (1978) Ltd., Klohn Leonoff Consultants Ltd., EBA Engineering Consultants Ltd. provided the background information summarized on the slope catalogue.

The pipe design mode, design problems, and required analyses are tentative and may be changed in light of new information.

All slopes steeper than 14 percent (8°) have been recorded from the Geotechnical Atlas. The 14 percent cut-off has been selected utilizing previous experience and engineering judgement. In addition, both static and pseudostatic methods of analyses, described in more detail later, have shown a slope with a 14 percent grade to be stable. For the assumption of worst case conditions, the following data was considered appropriate:

1. For the static analysis of a thawing longitudinal slope in a permafrost area, the parameters utilized were:

   - Friction angle 30 degrees;
   - Coefficient of consolidation 3×10^{-3} cm^2/sec.
   - Ratio of effective to total unit weight = 0.5.

2. For the pseudostatic analysis, the values given to the input parameters are summarized below:

- Slope angle 8 degrees (14%);
- Friction angle 11 degrees;
- Cohesion Intercept = 12 kPa (250 psf);

These parameters, input to the analyses, represent lower bound values based on test results for the Yukon route and Alyeska pipeline route (Donovan and Krzewinski, 1978). The rationale for the selection of drained and undrained parameters for static and pseudostatic analyses, respectively, is dicussed in subsection 5.1.

The safety factors obtained for a thawing longitudinal slope in permafrost using the above data gave:

1. FS (static) = 1.57
2. FS (pseudostatic) = 1.22 with an acceleration of 0.35 g

For cross slopes with water table at the ground surface, a pseudostatic safety factor of 0.87 (Displacement: 12.0 mm) has been calculated for the standard and deep burial construction modes, when traversing a 14 percent (8°) cross slope. This safety factor was predicted using the Janbu2 computer program, which is discussed in more detail later.

It can be concluded that the results of the above preliminary parametric studies indicated that a 14 percent (8°) limit was a reasonable lower bound to the slopes that would require inclusion in the slope catalogue for further study.

2.3 Selection of Slopes Requiring Analyses

To obtain a preliminary geotechnical assessment of the stability of slopes the background information provided in the catalogue was reviewed. The procedure described in the following was followed.

1. The length and height of a specific slope in conjunction with its grade provide an indication of the likelihood of instability and the potential adverse effects of failure.

2. Evaluation of the borehole logs provide a qualitative knowledge of the materials constituting the slope. The index properties of these materials provide an indication of their strength and behavioural characteristics. Observations noted during the field investigations permit an estimation of groundwater conditions.

Slopes that had lower height, length and grade, low groundwater table and good strength characteristics were regarded as not requiring analyses. This was ascertained during subsequent field reconnaissance which verified slope geometry and provided a visual examination of surrounding terrain.

2.4 Design Flow Chart for Slope Stability Analytical Program

The procedure proposed to achieve the objective outlined in Subsection 1.2 is summarized on the flow chart, Figure 2, and the sequence is described in the following.

1. Preparation of slope catalogue.

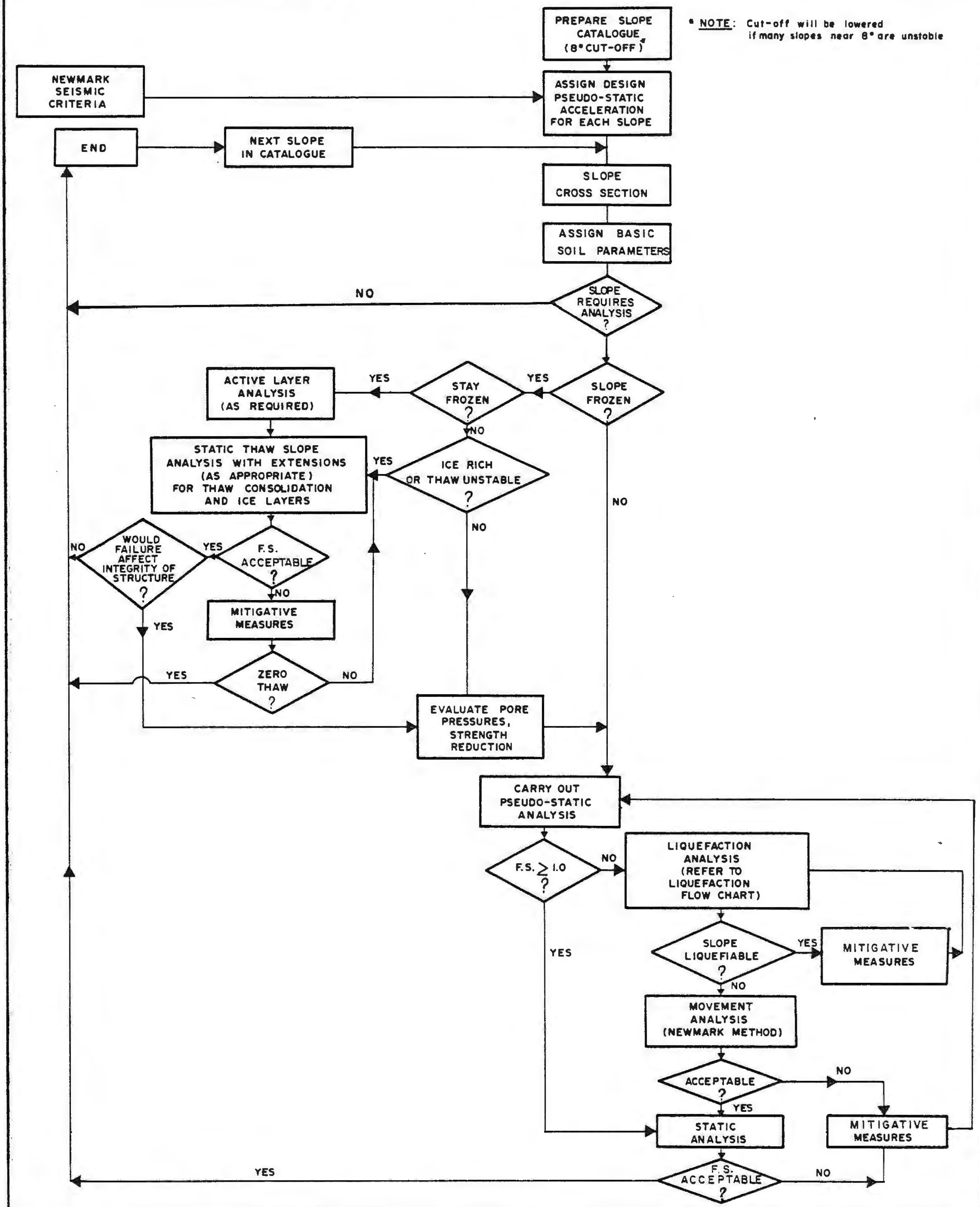
2. Assign earthquake accelerations for each slope based on Newmark Seismic Design Criteria.

3. Preparation of cross sections for each slope.

4. Assign soil parameters required for slope stability analyses.

5. Determine which slopes required analyses, as discussed in Subsection 2.2 and 2.3.

6. Conduct stability analyses for slopes consisting of non-liquefiable soils. Potential liquefaction will be determined by the seismic liquefaction studies and appropriate mitigative measures will eliminate this potential. The slopes considered for slope stability analyses are classified as follows:



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FLOW CHART FOR SLOPE STABILITY ANALYTICAL PROGRAM

K 5501 – E

FIGURE 2

a) Frozen Slopes.

b) Thawing Slopes.

c) Unfrozen Slopes.

All stabilized slopes will be re-analysed to ascertain the adequacy of the mitigative measures undertaken.

7. Estimate total ground movement for slopes subjected to earthquake accelerations when the pseudostatic factor of safety is less than unity.

8. Carry out static stability calculations for all slopes requiring analyses, to ensure that an adequate safety factor is maintained for slopes not subjected to a design earthquake event.

2.5 Methods of Analyses

The analytical methods described in detail in Section 6.0 fall into two main categories:

1. The static methods of slope stability analyses are aimed primarily at predicting the long term behaviour. These methods are also utilized to ascertain stability at various stages of construction, as appropriate.

2. The pseudostatic methods are an extension of the static methods. Earthquake loads may be imposed as a constant static force in the downslope direction. The result of these analyses predict the factor of safety during a seismic event.

2.6 Data Inputs

Input parameters required for each analytical method are reviewed in detail in section 5.0 and consist of the following:

1. Slope geometry
2. Shear strength parameters (drained or undrained)
3. Total unit weights
4. Pore pressures.

Slope geometry is obtained from the geotechnical atlas and/or field surveys, as required. The triaxial consolidated-undrained test with pore pressure measurements provides both drained and undrained shear strength parameters. Drained shear strength parameters are also obtained from direct shear test results. Both total and dried unit weights are also computed from direct shear and triaxial sample data. Groundwater conditions are measured using standpipes and piezometers installed in the field. Pore pressures resulting from thaw of frozen soils are predicted utilizing the coefficient of consolidation obtained from thaw strain tests.

3.0 SLOPE CONDITIONS

3.1 General Geometry

3.1.1 Cross Slopes

Cross slopes are those whose face crosses the pipeline right-of-way. The distance considered in recording average and maximum cross slopes extends to 100 m upslope and 25 metres downslope of the pipe. In this respect, the geotechnical atlas has also been reviewed recognizing that there could be steep and high slopes outside the boundary considered that may possibly affect the integrity of the pipeline and support systems, and this will be reflected in the cross section drawings. Cross slope grades are considered positive, when looking in the direction of increasing chainage, the slope increases in height to the right.

3.1.2 Longitudinal Slopes

Slopes that are parallel with the pipe are known as longitudinal slopes. Slope height and length were recorded along the pipe. Longitudinal slope grades are considered positive when, advancing in the direction of increasing chainage, they increase in height.

3.2 Classification and Methods of Analysis

3.2.1 Frozen Slopes

Overall shear failure in frozen soils could only occur in high and steep (40 m and 60°) frozen slopes composed of fine-grained soils at temperatures near their melting point. For these frozen slopes, although the frozen soils possess a higher shear strength, they may still only be in a state of limiting equilibrium (Factor of safety near 1.0). Along the Foothills pipeline route, frozen slopes encountered are such that failure is unlikely. As a consequence, no stability analyses will be conducted on frozen slopes that are expected to remain frozen. In addition, the frozen soil shear resistance will be greater during earthquake events because of the sudden and temporary nature of earthquake loading and the high strength of frozen soils under rapid rates of loading.

3.2.2 Thawing Slopes

This classification comprises all slopes where thawing of the permafrost is expected to occur as a result of construction and/or pipeline operation. As a consequence, it is anticipated that these slopes will be restricted to the general right-of-way area. Stability analyses for these slopes will account for potential seismic activity.

At some locations the active layer may have to be analysed to ensure that skin flow failure of slopes will not occur. Normally, only the static stability of shallow thawing slopes will be evaluated for the general right-of-way area. In cases where skin flow failure will affect operation of the pipeline and support system, earthquake loading will also be considered.

3.2.3 Unfrozen Slopes

Pseudostatic analyses will be undertaken for these slopes, in which the effects of earthquake loading are incorporated. There is no permafrost or depressed permafrost table present to form a hard stratum to limit the shapes that can be assumed by a potential surface of sliding. Potential failure surfaces can be shallow planar surfaces, or can be deeper circular or non-circular surfaces depending on the position of weaker soil layers within the slope.

4.0 DESIGN CRITERIA

4.1 Liquefaction

Liquefaction of the slope materials will result in a mode of failure not related to the limit equilibrium techiques employed here. As a consequence, only non-liquefiable slopes will be analysed. Liquefaction analyses will be conducted as described in Part II of this report - "Seismic Liquefaction". For potentially liquefiable soils, appropriate mitigative measures would be undertaken so that liquefaction will not occur, or will not be detrimental to pipe operation. Subsequently, stability analyses will be conducted as shown on the flow chart for the slope stability analytical program, given on Figure 2.

4.2 Erosion

Slope instability can result from erosion and can only be circumvented by preventative and/or mitigative measures comprising erosion control.

Revetments to prevent erosion or design of the pipe to accommodate shifting of some river banks will be studied by Foothills hydrological consultants.

4.3 Factor of Safety

In slope stability practice, the factor of safety is that factor by which the strength parameters can be reduced in order to bring the soil mass into a state of limiting equilibrium along a given failure surface.

Factors of safety are set in geotechnical practice having regard to the nature of the geotechnical conditions, the type of structure, the economic implications of failure, and the danger to public safety. Large dams for example are usually specified to have a computed safety factor of 1.5 for the full reservoir condition, but lower factors are often considered acceptable for reservoir drawdown. On the other hand, there is no well established practice for setting safety factors for pipeline slopes. The major determining factor for pipelines is usually economics and environmental factors as public safety is normally not threatened to the same degree as for dams or nuclear powerplants. Also, as the Foothills system is of tremendous length traversing a wide variety of soil, groundwater, permafrost and seismic conditions, it is economically unreasonable to have fixed criteria set for the entire system.

The following approach in establishing safety factors is therefore recommended:

4.3.1 Desirable Factor of Safety

The desirable target for static loading conditions not involving earthquake effects is to have a safety factor in the range of 1.25 to 1.5. At the same time dynamic/earthquake loading conditions should result in a pseudostatic factor of safety equal to or greater than unity.

If such measures are undertaken then the slope and the pipeline will be safe and will moreover, easily withstand the design ground motion event with the pipeline remaining in safe operating condition.

4.3.2 Minimum Desirable Factor of Safety

There will be circumstances of soil conditions, geometrical constraints and reasonable economics for which we recommend adopting the following approach. The minimum static factor of safety for slopes where a critical failure surface intersects the buried pipeline must, after construction, be greater than unity. At the same time, movement analyses must predict displacements within the allowable range as determined by Pipeline Design.

Under these conditions the pipeline may be stressed to near its design limits during the design ground motion event. Depending on the actual severity of the seismic event, the pipeline should remain in service although it may be necessary to effect repairs on pipeline segments which become over stressed. This position is in accord with Newmark (1980) who points out that after a "Design Contingency Earthquake", major repairs may be necessary. In undertaking analyses for slope stability purposes, see Table 1, we have adopted Newmark's (1980) contingency ground motion values which have a low probablity of occurrence during pipeline lifetime.

4.3.3 Monitoring Option

For some slopes, geotechnical factors are such that additional measures may be required to ensure safe operating conditions over the pipeline life. In such cases, while experience and judgement indicate that conditions are or can be made safe, the cost implications of service interruptions may require early warning of impending instabilities. For such slopes it may be judged necessary to implement a monitoring program. Such monitoring could involve slope indicators and stress/deformation monitoring on the pipeline as well as the conventional reports of right-of-way inspection. If unsafe conditions as defined by soil/pipe interaction studies are identified then remedial action would be initiated.

4.4 Allowable Slope Movement

Maximum allowable ground movements will be established based on pipe stress analyses, and determined by the design structural engineers. Permanent slope displacements will be estimated for the design contingency earthquake loading conditions utilizing procedures developed by Newmark and described in Subsection 6.4. Should the permanent displacements exceed the specified tolerable limits, mitigative measures will be designed so that these estimated displacements for the final slope configuration will be below the allowable maximum.

4.5 Earthquake Parameters

Ground motion accelerations to be used in stability analyses have been developed by Newmark and are summarized in Table 1 (Newmark, 1980). New information and earthquake occurrences obtained from regional seismographs is presently being included in the overall data base and seismic design criteria.

For the present, however, an acceleration of 0.35g is recommended by Newmark for Construction Sections III and IV in sediments, and 0.28g in soft soils where dynamic slope stability is likely to be a problem.

5.0 DESIGN INPUT PARAMETERS

5.1 Slope Geometry

Cross sections and/or profiles have been drawn for all slopes reported in the slope catalogue, Appendix A. The purpose of the cross section drawings is twofold:

1. To permit an appreciation of the slope conditions together with the implications of its geometry to the constructed pipeline configuration;

TABLE I

Effective Horizontal Ground Motions
Design Contingency Earthquake (After Newmark, 1980)

Zone	Mag.	Accel. %g Rock*	Accel. %g Sediment**	Accel. %g Soft Soil***	Veloc. cm/sec All	Displ. (cm) Rock*	Displ. (cm) Sediment**	Displ. (cm) Soft Soil***
A. Soil Response								
a. MP[1] 0-200	7.0	40	35	28	42	28	32	40
b. MP 200-250	6.5	32	27	22	32	20	24	28
c. MP 250-500	5.6	20	15	12	18	10	14	17
B. Structural Reponse								
a. MP 0-200	7.0	30	25	22	30	18	22	25
b. MP 200-250	6.5	22	18	16	20	11	14	16
c. MP 250-500	5.6	15	12	10	14	8	10	12

* Values to be used for rock, thick permafrost, or less than 10 metres of sediment over rock or permafrost.

** Values to be used for unfrozen sediment depths greater than 30 metres, or for cases of thin permafrost underlain by unfrozen sediments. For intermediate values, use linear interpolation between sediment depths of 10 and 30 m. These values also to be used for soft soil stratum depths of 3 m or less.

*** Values to be used when seismic shear wave velocity is less than 700 ft/sec for 10 m or more of soft soil under structure foundation, or in top 10 m if no structure is involved, Interpolate between sediment and soft soil values for depths between 3 m and 10 m.

(1) Mile Post.

2. The cross sections will be utilized as part of the input to slope stability analyses, where analysis is considered necessary.

For a specific slope, the profile at the steepest location has been selected. On slopes where length of pipe affected is relatively long, more than one cross section has been drawn.

5.2 Governing Engineering Properties of Soils

For slope stability analyses, the governing engineering property of a soil is its shear strength and the strength reducing factors, primarily pore pressures. The strength and strength reducing factors were obtained through triaxial consolidated-undrained tests with pore pressure measurements, direct shear tests and thaw consolidation tests. From these test samples, densities and water contents were obtained both prior to and after testing.

5.2.1 Shear Strength

In general, the mobilized shearing resistance along a failure plane can be written as:

$$\tau = c + \sigma \tan \phi$$

where τ is the shear strength

σ is the normal stress

c and ϕ are the cohesion intercept and friction angle obtained from the appropriate soil tests.

Triaxial and direct shear tests have been conducted to establish lower bound shear strength parameters for soils encountered along the route, based on soil type, dry density prior to testing and water contents.

1. Triaxial consolidated-undrained tests with pore pressure measurements provide both undrained and drained shear strength parameters.

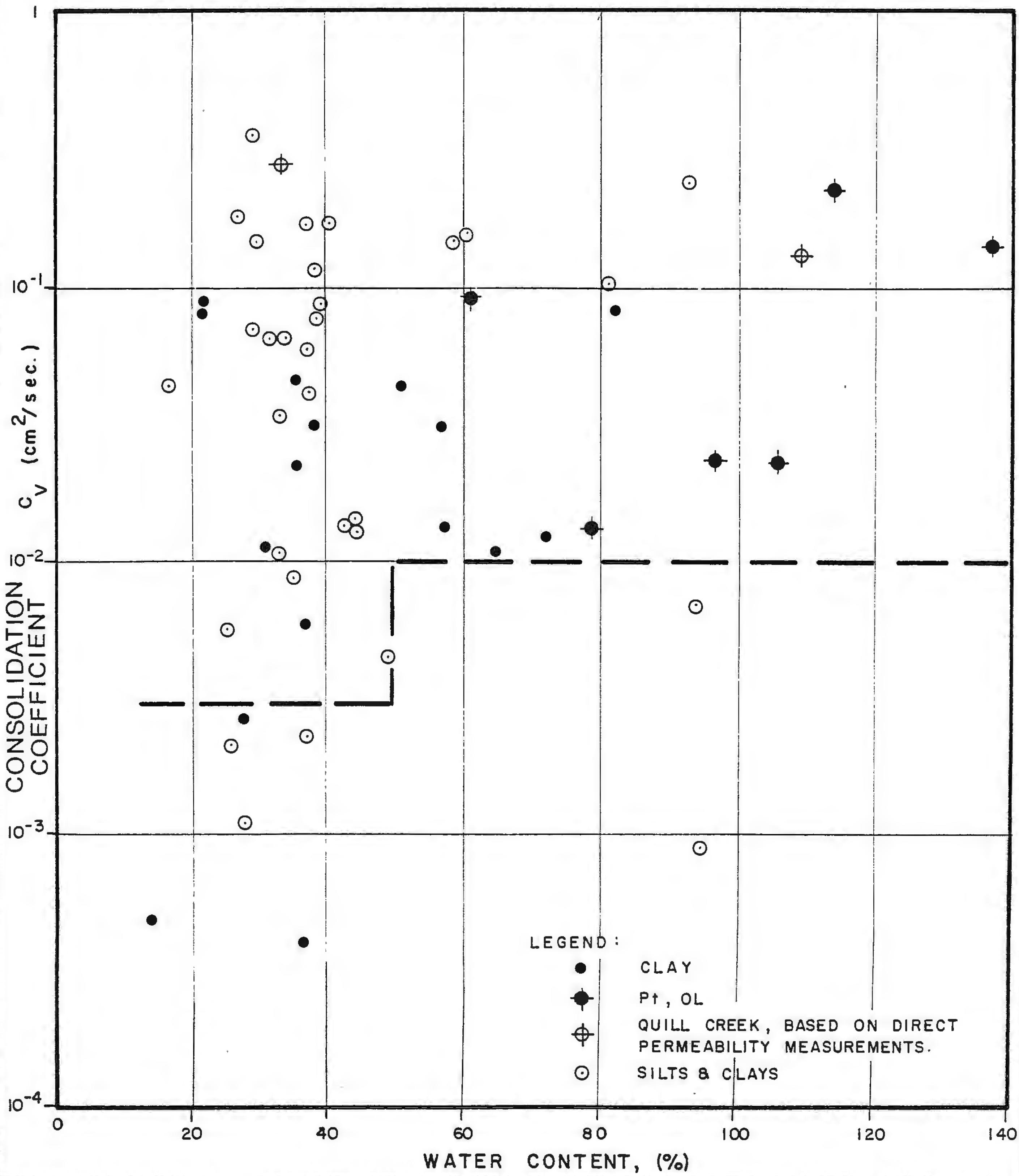
   The consolidated-undrained shear strength (ϕ_u, c_u) is applicable if shearing of the soil occurs without dissipation of excess pore pressures developed during shearing, such as in the case of earthquakes or any other rapidly applied load. Results of analyses utilizing these parameters usually represent a short term condition.

   The consolidated-drained shear strength parameters (ϕ', c') are applicable to long term conditions. The rate of application of shear stress and the rate of soil consolidation are such that excess pore pressures developed during shearing are largely dissipated. Full drainage before shearing is implied.

2. Direct shear tests provide only drained strength parameters. These tests are most applicable to coarser grained soils (coarse silt to sands) which drain quickly during shear. When tests on fine-grained soils are carried out, the rate of shearing must be sufficiently slow to allow dissipation of excess pore pressures.

5.2.2 Thaw-Consolidation Tests

Thaw Consolidation tests provide the coefficient of consolidation which is an essential parameter in determining the excess pore pressures resulting from thaw. Since c_v enters the computations under the square root, small variations in this parameter do not cause large changes in the predicted pore pressures for slope stability calculations (See Section 6.1.3) However, the measurement of this property remains a critical variable in thaw slope stability computations. The coefficient of consolidation versus initial (frozen) water contents for soils in the South Yukon are presented as Figure 3. In addition, more tests are presently underway. However, based on the existing data, it can be observed:



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COEFFICIENT OF CONSOLIDATION VERSUS WATER CONTENT FOR SOILS IN THE SOUTH YUKON

K 5501- E

FIG. 3

1. For water contents greater than 50 percent the minimum c_v is 10^{-2} cm^2/sec.

2. For water contents less than 50 percent the lower bound value of c_v is 3×10^{-3} cm^2/sec.

3. A c_v value of 10^{-2} cm^2/sec will be conservative for the majority of soils encountered along the route.

5.2.3 Rate of Thaw

An assessment of the rate of thawing is required to estimate excess pore pressures resulting from thawing. For a uniform, homogeneous layer, subjected to a step increase in surface temperature, thaw proceeds as defined by:

$$d = \beta\sqrt{t}$$

where d is the depth of thaw, and

t is the time.

and β is a constant predicted from thermal analysis, or based on experience

The above states that there is a linear relationship between depth of thaw and the square root of time. This determination of has been given by Nixon and McRoberts (1973), where simple solutions are presented and compared to more rigorous solutions. In addition, the limitations of the available solutions are also outlined.

Along the Yukon route, depth of thaw has been measured by installing thermistor strings at several location. For construction sections III and IV, the constant β is approximately 0.043 cm/ sec.$^{1/2}$ (2.41 m/yr$^{1/2}$). This has been found to apply equally well to active layer thaw depths in a single season, or to the long term deepening of the permafrost table that has been observed in the South Yukon, Alaska and the Mackenzie Valley.

There are several reasons for departures from the linear relationship given for the depth of thaw. The most significant of these is the presence of a surficial layer of peat which retards progression of the thaw interface. Values of β measured at several locations through interpretion of thermistor readings and predicted by geothermal studies will be utilized on a site specific basis, as required.

5.2.4 Thawing Slopes

Drained strength parameters will be utilized for free-draining materials under all types of loading conditions and for cohesive materials under static loading conditions. Relevant pore pressure parameters will also be input into the stability analyses. These include the elevation of the static groundwater table, and any excess pore pressure component as defined later in Section 6. For cohesive materials subjected to earthquake loads, the consolidated undrained shearing resistance will be utilized.

5.2.5 Unfrozen Slopes

Input parameters to the stability analyses of these slopes will be similar to those for thawing slopes, described in the preceeding subsection, except no excess pore pressures generated by the thawing of icy, fine-grained permafrost soils need be incorporated.

5.3 Groundwater Conditions

Groundwater conditions will be evaluated utilizing field drilling observations, piezometer readings and general topography of the area. In many cases where no site-specific information on groundwater is available, the water table will be assumed based on the geometry of the slope in relation to surrounding water bodies, springs, or other evidence of groundwater elevation.

5.4 Earthquake Loading

In current practice earthquake loads are considered as a steady force acting at the centre of gravity of the soil mass under consideration. Earthquake forces are converted into an equivalent horizontal static force by multiplying the soil mass by the prescribed acceleration. For Sections III and IV, an acceleration of 0.35 g has been recommended by Newmark (1980). Because the dynamic force (earthquake loading) is converted to a static force, this type of slope stability analysis is referred to as pseudostatic.

6.0 ANALYTICAL METHODS

Slope stability analyses (Skempton and Hutchinson, 1969; Morgenstern, Blight, Janbu, Resendiz, 1979) are based on limit equilibrium methods. These procedures are not concerned with the prediction of deformation and instead employ force equilibrium methods. The general procedure for slope stability analyses is as follows:

1. Postulate a slip mechanism

2. Using statics, estimate the shear strength required to maintain the slip mechanism in a state of limiting equilibrium

3. Compare the above-mentioned shear strength with the available shearing resistance through a factor of safety.

Deformations are controlled through the judicious selection of a minimum acceptable factor of safety. The error associated with the method of analysis is in the order of 10 percent difference, for the better available techniques (Lambe and Whitman, 1969). This difference is usually small when compared to that arising from an error in the selection of strength parameters. Based on this and other considerations, minimum acceptable safety factors have been established and discussed in Subsection 4.2.

6.1 Infinite Slope Stability Analysis

The infinite slope method of analysis is conducted for slopes where the thickness of the potentially unstable material is small compared to the length of the sloping soil mass. The analytical method is derived in closed (exact) form, and is completely accurate. This fact allows checks to be made between more complex computer analyses, and the infinite slope method.

The analysis consists of cutting a free-body element of soil from the slope and assumes:

1. The slope is very wide in the direction normal to the cross section, and only the stresses that act in the plane of the cross section are considered

2. The interslice stresses are equal and balance each other.

If the latter assumption was not true, the stresses on the vertical faces would change depending on location of the slice along the slope and such a situation would be inconsistent with observations that thin veneer of entire slopes have moved as a single mass.

6.1.1 Static Analysis

The equilibrium of a free body element of soil in terms of forces can be determined for a dry or submerged slope with no seepage and the factor of safety is given by:

$$FS = \frac{\tan \phi'}{\tan \alpha}$$

where: FS = Factor of Safety

ϕ' = Friction Angle

α = Slope Angle

Considering forces on a free body element of soil, one can correctly account for the effect of water by reducing the effective stress along the failure plane and therefore the effective shearing resistance. Seepage parallel to a slope reduces its stability and for the water table at the ground surface, its factor of safety is determined by:

$$FS = \frac{\gamma'}{\gamma} \frac{\tan \phi'}{\tan \alpha}$$

where γ' = buoyant unit weight

γ = total unit weight

The ratio of effective to total unit weight is usually about 0.5 for hydrostatic conditions.

6.1.2 Pseudostatic Analysis

Earthquake forces are inertia forces equal to the total mass of the soil multiplied by the acceleration. Since failures induced by earthquake loadings are undrained, the undrained envelope for soil shearing resistance may be used. This envelope exhibits a significant cohesive intercept, together with a lower friction angle, in contrast to the more usual effective strength envelope where the cohesive intercept is often equal or close to zero. As mentioned previously, coarser grained soils will drain during an earthquake event, and the use of the more conventional effective strength envelope may be appropriate. In these cases, the cohesive intercept will be essentially zero.

The following equations provide the safety factor for the described condition:

1. Dry Slope : Cohesion = 0

$$FS = \left[\frac{1 - a \tan \alpha}{a + \tan \alpha}\right] \tan \phi' \qquad (1)$$

where a = earthquake acceleration (fraction of gravitational acceleration).

α = slope angle (degrees)

ϕ' = soil friction angle (degrees)

2. Submerged Slope: Cohesion = 0

$$FS = \left[\frac{\frac{\gamma'}{\gamma} - a \tan \alpha}{a + \frac{\gamma'}{\gamma} \tan \alpha}\right] \tan \phi' \qquad (2)$$

where $\frac{\gamma'}{\gamma}$ = ratio of effective to total unit weights

3. Seepage Parallel to Slope: Water Table at Surface

$$FS = \left[\frac{c'}{\gamma H} \; \frac{\operatorname{Sec} \alpha \operatorname{Cosec} \alpha}{1 + a \cot \alpha}\right] + \left[\frac{\frac{\gamma'}{\gamma} - a \tan \alpha}{a + \tan \alpha}\right] \tan \phi' \qquad (3)$$

where c' = cohesion intercept (kPa)

h = depth of failure surface (m)

γ = total unit weight (kN/m^3)

The cases where the water table is below the ground surface may be handled by replacing the γ'/γ term in equation (3) by $\gamma'/\gamma + \gamma_w x/\gamma d$, where x and d are the depths to the groundwater table and the potential failure surface respectively.

It should be noted that when the earthquake acceleration is equal to zero, the equations presented for the static loading cases are recovered. For static loading, the relationships for dry and submerged slopes are identical. However, when the earthquake loads are applied in the analysis, the ratio of γ'/γ enters into the formulation. This is due to the fact that the dynamic load is applied to both the water and soil phases, whereas only the soil phase provides the shearing resistance.

For the condition where seepage is parallel to the slope and the cohesion is equal to zero, factors of safety are presented on Figure 4 for various slope angles, various friction angles, earthquake accelerations, and effective to total unit weight ratios. Figures 5 and 6 provide the acceleration at which a slope with parallel seepage is in limiting equilibrium, for frictional and cohesive soils, respectively.

6.1.3 Excess Pore Pressures

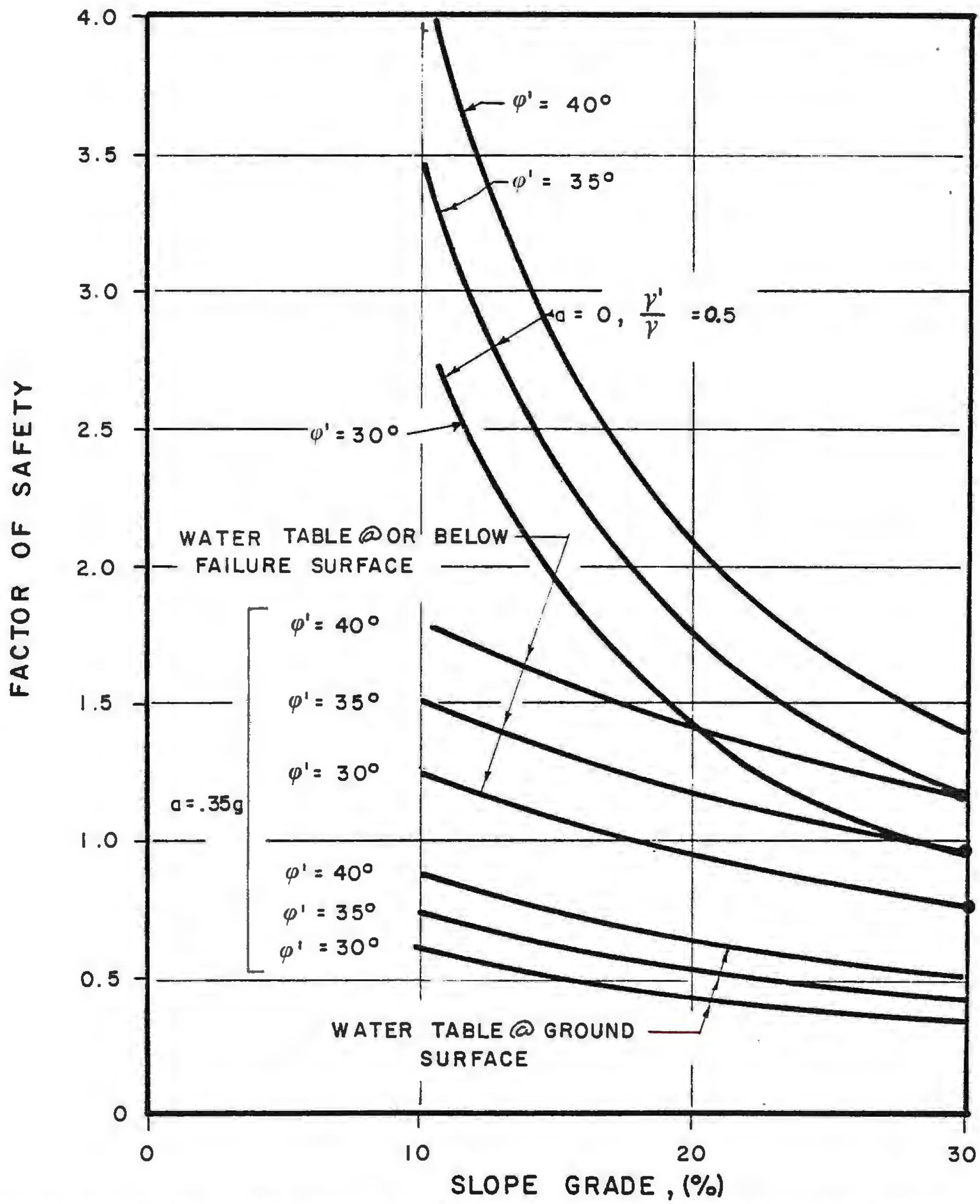
Excess porewater pressures can be set up in thawing soils as a consequence of thaw-consolidation. Morgenstern and Nixon (1971) have solved the one-dimensional thaw-consolidation problem and the excess pore pressures have been found to be:

$$u = \frac{\gamma' d}{1 + 1/(2R^2)}$$

$$FS = \left[\frac{\gamma'/\gamma - a\tan\alpha}{a + \tan\alpha}\right]\tan\varphi' \quad \text{WATER TABLE @ GROUND SURFACE}$$

$$FS = \left[\frac{1 - a\tan\alpha}{a + \tan\alpha}\right]\tan\varphi' \quad \text{WATER TABLE @ OR BELOW POTENTIAL FAILURE SURFACE}$$

WHERE: α = SLOPE ANGLE
φ' = FRICTION ANGLE
a = EARTHQUAKE ACCELERATION
γ', γ = EFFECTIVE, TOTAL UNIT WEIGHTS



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INFINITE SLOPE ANALYSIS
SEEPAGE PARALLEL TO
SLOPE: FRICTIONAL SOILS

K 5501-E

FIG. 4

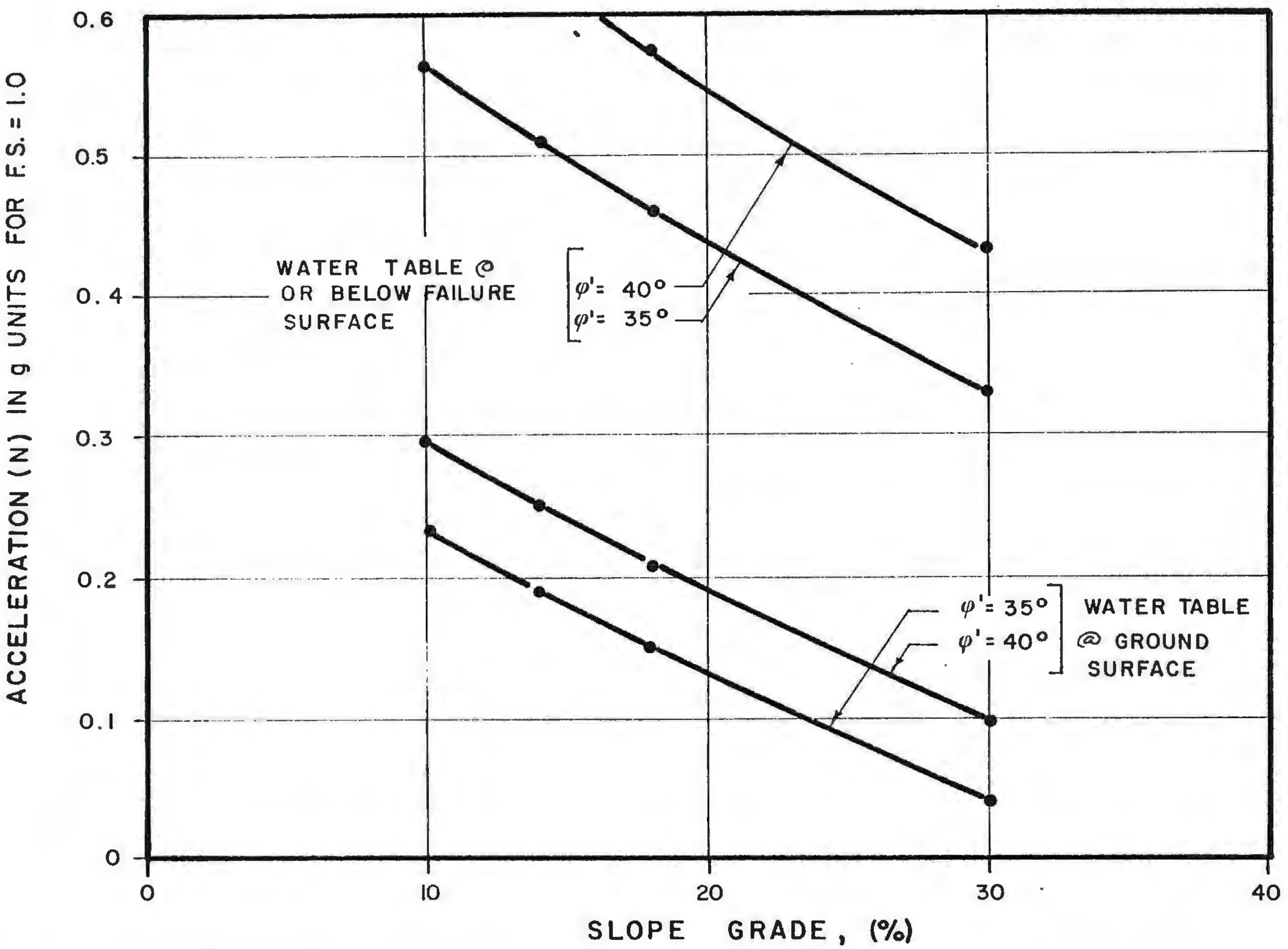
$$N = \frac{\tan\varphi' - \tan\alpha}{1 + \tan\alpha \tan\varphi'}$$ – WATER TABLE @ OR BELOW FAILURE SURFACE

$$N = \frac{\gamma'/\gamma \tan\varphi' - \tan\alpha}{1 + \tan\alpha \tan\varphi'}$$ – WATER TABLE @ GROUND SURFACE

WHERE: α = SLOPE ANGLE

φ' = FRICTIONAL ANGLE

N = ACCELERATION FOR F.S. = 1



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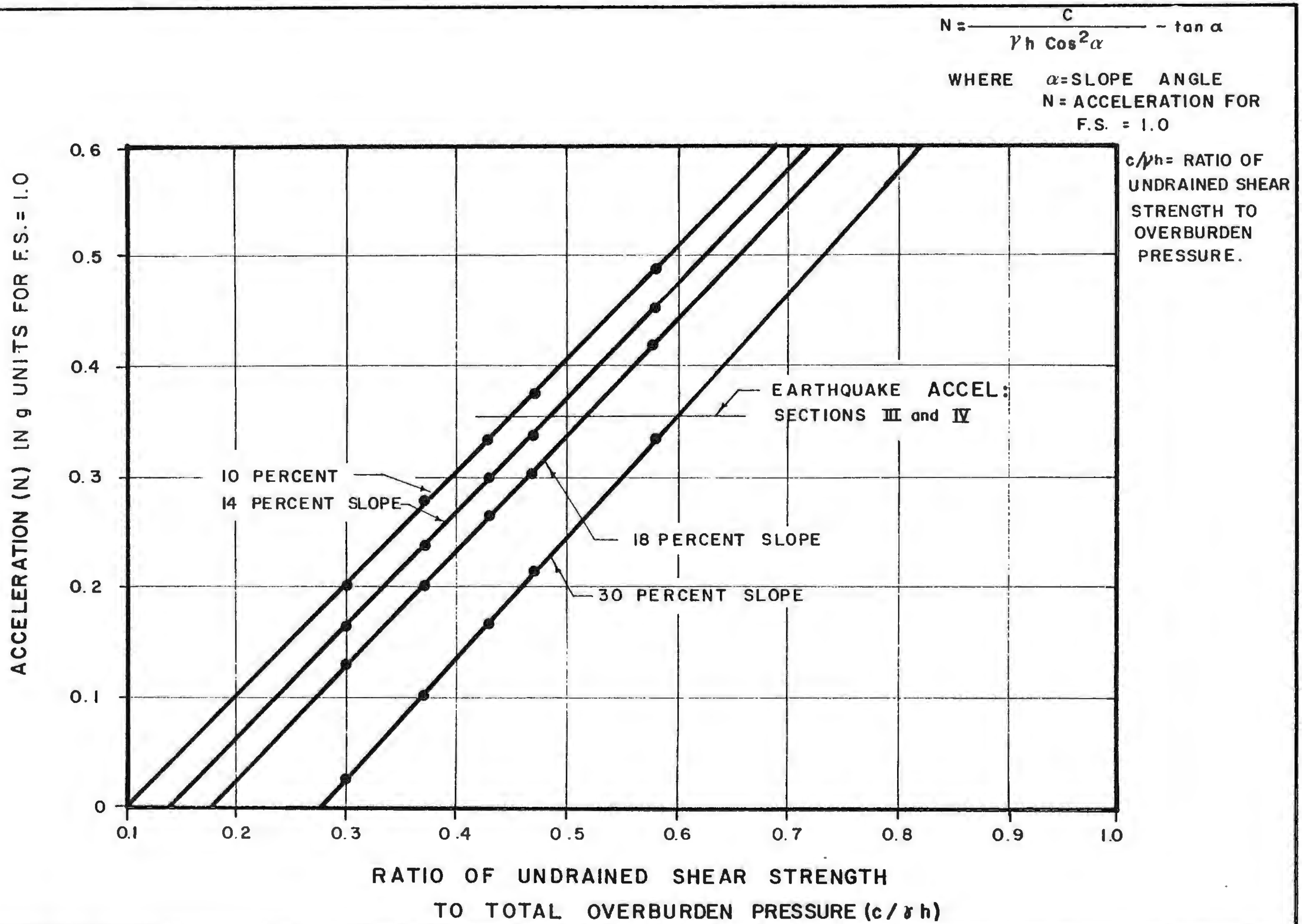
SLOPE GRADE VERSUS ACCELERATION FOR F.S. = 1.0, FOR PURELY FRICTIONAL SOILS – SEEPAGE PARALLEL TO SLOPE

K 5501-E

FIG. 5



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SLOPE GRADE VERSUS ACCELERATION FOR F.S.= 1.0, FOR COHESIVE SOILS SEEPAGE PARALLEL TO SLOPE

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FIG. 6

where u = excess pore pressures

γ' = effective unit weight

d = depth to the thaw front (frozen ground)

The thaw-consolidation ratio, R, (Morgenstern and Nixon, 1971) is defined by:

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

where c_v = coefficient of consolidation

β = a constant (units are m/yr $^{1/2}$)

Taking into account excess pore pressures generated by thawing (McRoberts and Morgenstern, 1974), the factor of safety equation for an infinite slope can be written as:

$$FS = \frac{\gamma'}{\gamma}\left[1 - \frac{1}{1 + 1/\left|2R^2\right|}\right]\frac{\tan\phi'}{\tan\alpha} \qquad (4)$$

where $\frac{\gamma'}{\gamma}$ = ratio of effective to total unit weight

ϕ' = soil friction angle

α = slope angle

Figure 7 presents the factor of safety versus slope grade for various friction angles and for excess pore pressure ratios of 0.08, 0.24 and 0. The latter represents hydrostatic conditions, with the excess pore pressures equal to zero. The first two excess pore pressure ratios correspond to c_v values of 1×10^{-2} and 3×10^{-3} cm^2/sec respectively. The ratio of submerged density to total density was assumed to be 0.5 in all cases and the constant indicative of the rate of thawing was estimated to be 0.043 $cm/\sqrt{sec}$.

6.2 Thaw Bulb Stability Analysis

Select granular material will be utilized to backfill the pipe ditch. Three potential types of failure are of concern and are addressed in the following.

6.2.1 Flooding

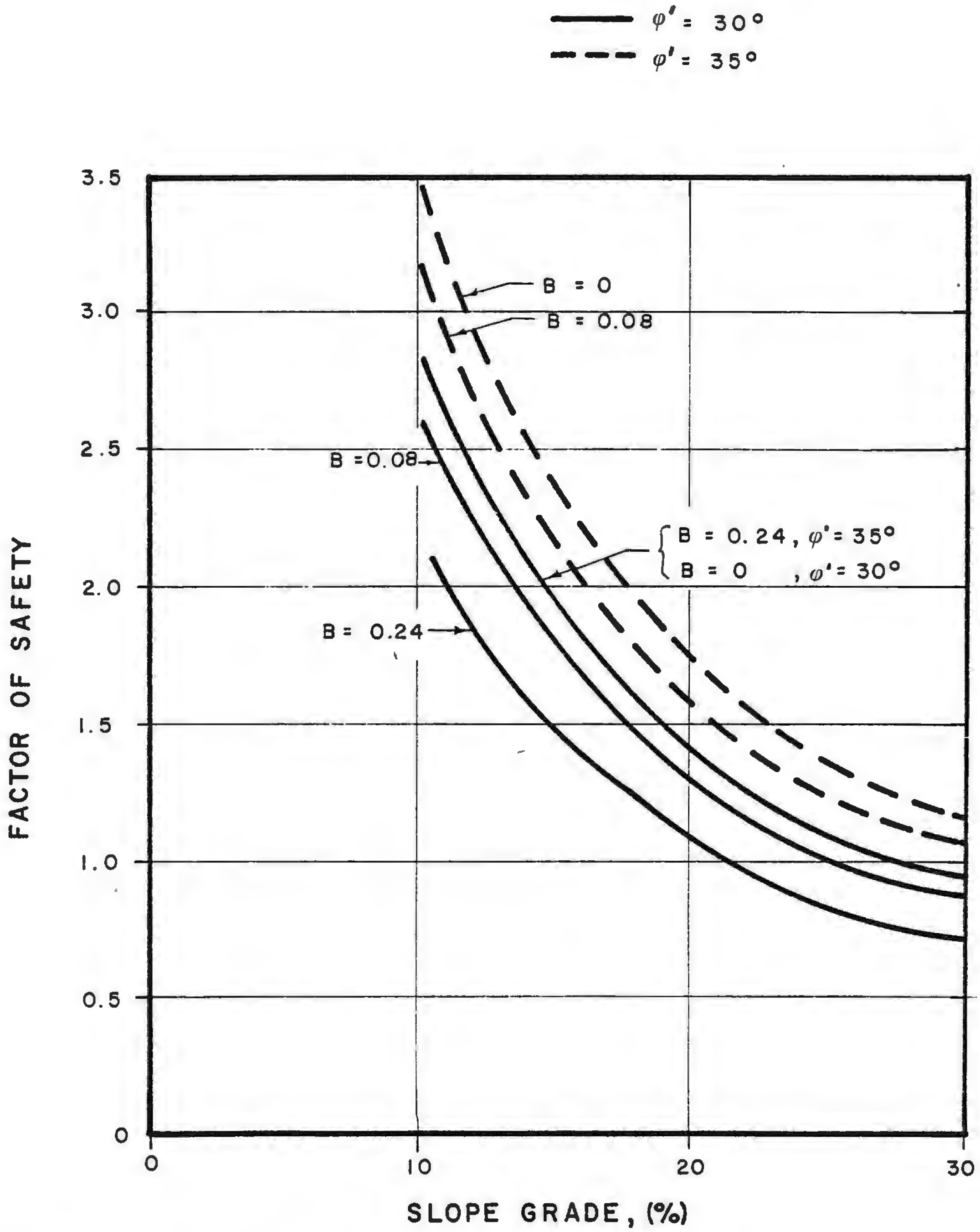
Flooding of the ditch backfill on a slope may cause a reduction in the strength of the backfill. Potential failure of the backfill in a downslope direction may be checked, incorporating the reduced shear strength due to flooding.

The thaw bulb stability analysis requires consideration of the two-dimensional effects. The sidewall shearing resistance can be accounted for by employing a ratio involving the coefficient of lateral earth

$$FS = \frac{\gamma'}{\gamma}\left[1 - \frac{1}{1+\frac{1}{2R^2}}\right]\frac{\tan\varphi'}{\tan\alpha}$$

WHERE: γ', γ = EFFECTIVE, TOTAL UNIT WEIGHTS
R = THAW - CONSOLIDATION RATIO
φ' = FRICTION ANGLE
α = SLOPE ANGLE

$$B = \left[\frac{1}{1+\frac{1}{2R^2}}\right]$$



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INFINITE SLOPE ANALYSIS WITH EXCESS PORE PRESSURES - FRICTIONAL SOILS

K 5501 - E

FIG. 7

pressure at rest (K_o), the ditch depth (Z) and the ditch width (S), i.e.:

$$\frac{K_o Z}{S}$$

For a ditch of infinite length, the factor of safety is given by:

$$FS = \frac{\gamma'}{\gamma} \left[1 + \frac{K_o Z}{S} \right] \frac{\tan \phi'}{\tan a}$$

where $\frac{\gamma'}{\gamma}$ = ratio of effective to total unit weight

ϕ' = soil friction angle

a = slope angle

When the ratio of $K_o Z/S$ is vanishingly small (i.e. shallow, wide potential surface of sliding), the infinite slope stability equations are recovered. In addition since pipe operation will result in thaw of the surrounding soils, infinite slope stability equations are more appropriate for study of long term stability, as they result in a lower (more conservative) factor of safety.

6.2.2 Thawing

Thawing of the native backfill may result in a reduced safety factor if excess pore water pressures are generated as a result of the thaw consolidation process. The stability of the backfill is primarily of concern in the inactive period subsequent to construction and before operation. As there will be free-draining bedding and padding adjacent to the pipe, this will tend to stabilize the backfill by preventing the build-up of excess pore pressures. In the early stages of thaw, the ratio accounting for two-dimensional effect $K_o Z/S$ would also be small. Therefore the appropriate one-dimensional infinite slope analysis would be employed in evaluating the stability of the ditch backfill.

6.2.3 Freeze-Back

During seasonal freeze-back of the ditch backfill, the possibility exists that higher pore pressures can be created in an unfrozen zone, upslope of the pipe. No additional adverse pore pressure conditions will be considered in the analysis for ditch backfill stability, as the granular backfill that will be utilized possesses a relatively high permeability. Therefore, completely saturated conditions on a slope close to the pipe are unlikely, as the backfill will generally act as a drainage path for water in a re-freezing active layer.

6.3 Rock Slope Analyses

6.3.1 General

Areas where rock will be encountered are identified on the Geotechnical Atlas. Rock cuts in Construction Sections III and IV, based on the cross section drawings, are not anticipated to exceed 20 m in height.

Slope stability in igneous and metamorphic rock types similar to those found in the Yukon, is governed mainly by the properties, attitudes and locations of rock mass discontinuities. The term discontinuities includes all fractures including joints, faults, shears and bedding surfaces. On this basis, relatively simple analyses suffice to develop recommended angles for rock slope excavation. The analytical methods utilized in the course of this work are discussed in greater detail in Subsection 6.3.3.

6.3.2 Methodology

In order to develop recommendations for rock slope excavations, the following tasks may be undertaken, depending on the magnitude of the specific slope in question.

In the office, locations of proposed rock cuts will be compiled from the Geotechnical Atlas. Blanket recommendations will be formulated for cut slopes less than 5 m in height, and the remaining major cut slopes requiring detailed investigation will be listed. The available bedrock geology information will be assembled. It is anticipated that the main source of this information will be reports of the Geological Survey of Canada. Target areas for examination of bedrock exposures will be established using data from the Geotechnical Atlas, anticipated locations of bedrock cuts and locations of bedrock outcrops along the pipeline right-of-way as determined by previously undertaken airphoto interpretation.

In the field, examination of available bedrock outcrops will be completed to determine discontinuity orientations and bedrock conditions. This procedure will only be carried out for the major cut slopes defined earlier at the office compilation stage. Wherever possible, joint orientations will be determined using line traverse methods. Discontinuity roughness will be established in the field as an aid to estimating the angle of internal friction, ϕ. A limited number of rock samples of various bedrock types will be collected in order to allow future laboratory shear testing.

During the subsequent analysis and engineering phase, discontinuity data gathered in the field will be plotted for various geological domains on contoured lower hemisphere Wulff nets (stereographic projections). The use of stereographic projection provides a simple technique for the analysis of the 3-dimensional data collected during the field program. A comprehensive model to determine the stability of bedrock wedges formed by the intersection of two or more discontinuities will be programmed on our in-house computer. It is anticipated that the method used will be that outlined in Hoek and Bray (1977). On the basis of kinematic and detailed instability calculations, recommendations will be developed for bedrock cut slopes. As part of these recommendations, preliminary guidelines for blasting and drainage associated with rock slopes will also be considered as required.

6.3.3 Discussion

As discussed previously, it is expected that the stability of cut slopes along the pipeline alignment will be governed by the properties of bedrock discontinuities. Possible modes of instability for slope cut by discontinuities include the following:

a) Wedge failures involving the translation of a block of rock bounded by intersecting discontinuities.

b) Translational failures along discontinuities where the slope may be considered to be infinitely wide.

c) Toppling failure involving rotational movement of rock blocks forward into the excavation.

d) Buckling of rock strata bounded by discontinuities dipping parallel to the slope.

Our initial analyses will be undertaken using kinematic methods in order to determine when a failure is possible along the discontinuity sets established during field work. In order for failure to be kinematically possible, the discontinuity set bounding the failure must daylight on the slope, in the case of translational failure. Alternatively, in the case of toppling failure, a condition describing the relation between discontinuity and angle of friction must be satisfied.

Kinematic analyses will be undertaken on lower hemisphere stereographic projections using methods widely used within the field of rock mechanics. Potential wedge failures will be analyzed using methods outlined in Hoek and Bray (1977). These methods permit the use of up to 5 separate planes bounding a rock block potentially subject to failure. In addition, anchor forces, if used, and earthquake accelerations can also be modelled.

Toppling will be modelled using kinematic methods similar to those described by Goodman and Bray (1976). We have available more comprehensive solutions which describe the forces on each rock block, however, it is anticipated that these solutions will not be required for this work. Potential buckling failures, if they occur, will be analyzed using methods developed by Cavers (1980).

The office work, field work and preliminary investigations will be undertaken prior to finalizing a decision on whether any drilling of bedrock will be required. At the present time, the necessity for drilling is only considered a remote possibility.

6.4 Slope Stability Analysis

All slopes where infinite or thaw-bulb methods of stability analysis are not applicable will be studied utilizing the Hardy Associates (1978) Ltd. Janbu2 computer program. This program is described in detail in Section 7. Both static and earthquake loading conditions will be considered. These slopes include:

1. Man-made slopes created by cutting into cross sectional slopes along the right-of-way. In permafrost areas, these slopes might either be protected by a gravel buttress or blanket, and the postulated failure surface can be either circular or irregular.

2. Stability of a fill placed on natural ground, especially the above ground mode where the foundation soils are ice-rich, must be investigated. The failure surface will most likely be at or near the thaw front and irregular.

3. Thawing slopes will be encountered in areas where the subsoil beneath the pipe elevation is thaw stable; thawing of the slope will result from pipe operation. For these slopes, both circular and irregular failure surfaces will be investigated.

4. For unfrozen slopes, both circular and irregular failure surfaces will be studied, depending on the positioning of the weaker layers within the slope.

Spencer (1969, 1978) has found that when earthquake loads are utilized, circular failure surfaces are more critical than logarithmic spiral surfaces in homogeneous soil conditions. However, homogeneous soil profiles are seldom found in practice. Moreover, the critical failure surface for a pseudostatic analysis is not necessarily the same as that for a static analysis. Therefore, both circular and irregular failure surfaces will be studied.

6.5 Slope Displacement Computations

When a slope does not maintain a safety factor greater than unity following a design seismic event, maximum displacement will be estimated utilizing a procedure described by Newmark (1965) and summarized in the following.

The displacement of rigid-plastic mass moving as a single body with resistance mobilized along the sliding surface is given by:

$$d = \frac{V^2}{2gN}\left[1 - \frac{N}{A}\right] \tag{5}$$

where: d = displacement of the rigid plastic mass

V = maximum ground velocity

g = acceleration due to gravity

N = measure of dynamic resistance to sliding

A = maximum acceleration

Newmark then normalized several recorded earthquakes to a maximum acceleration and velocity, and compared the normalized displacements to those estimated by the above equation. Subsequently, he concluded that better results would be obtained by multiplying the above equation by A/N. This quantity could be indicative of the effective number of pulses in the earthquakes. The equation takes the form:

$$D = \frac{V^2}{2gN}\left[1 - \frac{N}{A}\right]\frac{A}{N} \qquad \text{- for } \frac{N}{A} \geqslant 0.15$$

where: D = maximum slope displacement under earthquake loading.

N = acceleration at which the soil mass would be in a state of limiting equilibrium (Factor of Safety = 1.0) along a given slip surface according to a pseudostatic analysis.

For very low values of N/A, an upper bound for the maximum displacement can be computed from:

$$D = \frac{6V^2}{2gN} \qquad \text{- for } \frac{N}{A} < 0.15$$

The above equations provide realistic estimates of the maximum permanent slope displacement after the excitation (earthquake) has stopped. The predicted ground motions will be compared with the established maximum allowable slope movement (see subsection 4.3) to determine if ground movements are excessive, and whether remedial measures are necessary.

Displacements estimated using the above equations and an earthquake acceleration of 0.35 g, are plotted versus the earthquake acceleration resulting in limiting equilibrium of the slope on Figure 8. This latter quantity, i.e. the earthquake acceleration that just causes limiting equilibrium of the slope, can be obtained from Figures 5 and 6 for a planar slope failure.

7.0 JANBU2 COMPUTER PROGRAM

7.1 General

The Janbu2 computer program calculates the stability of a soil slope using Janbu's Simplified Method of Analysis (Janbu, 1954; Morgenstern and Sangrey, 1978). It has two notable features in that the program searches for the slip surface with the minimum factor of safety, and that it accepts non-circular slip surfaces. The input parameters for the program consist of a two-dimensional slope geometry with details of the soil layers and their respective strengths, groundwater conditions, external loads on the slope and the location of the initial trial slip surface.

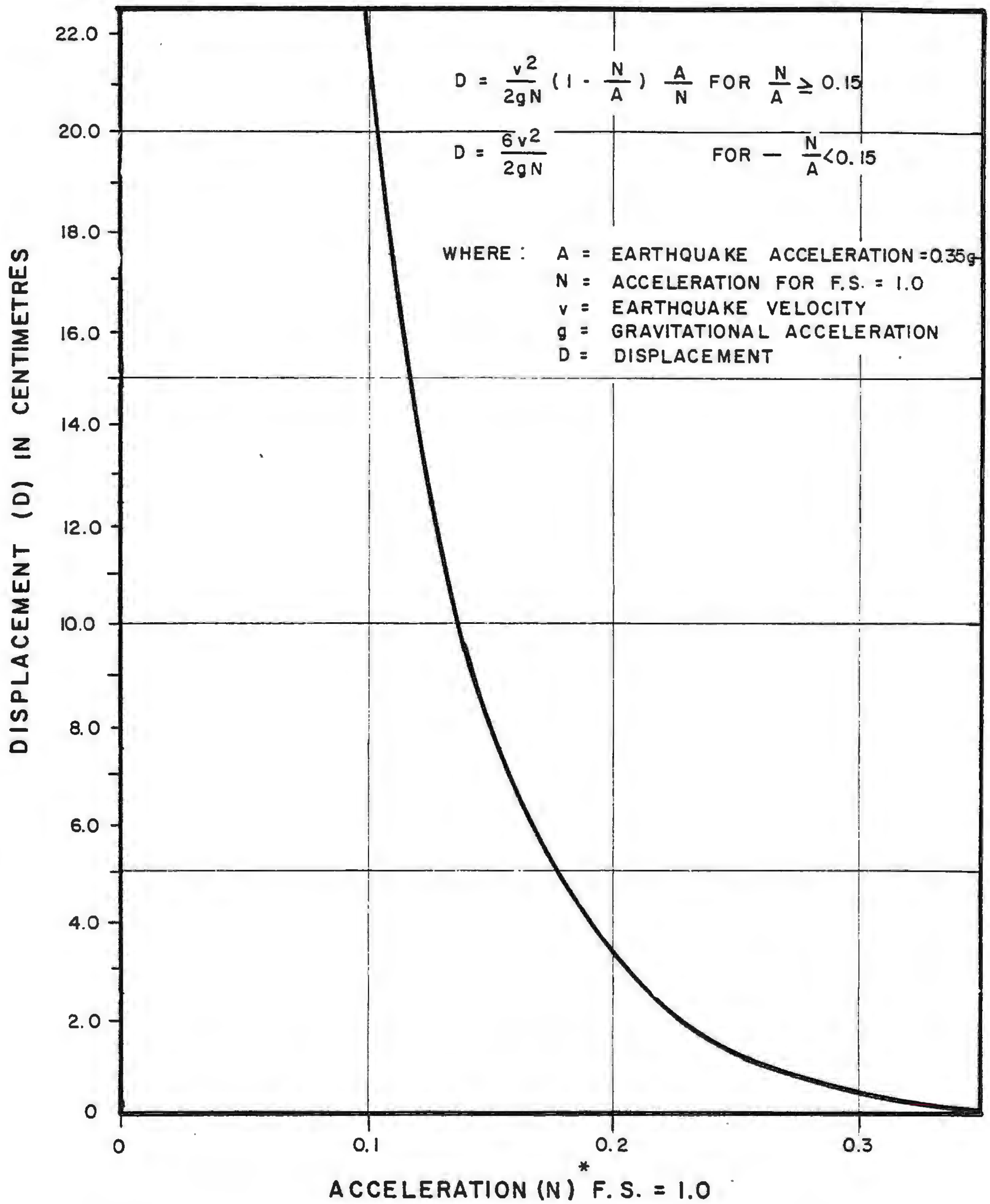
7.2 Method of Computation

Janbu's Simplifed Method of Analysis is one of the "Methods of Slices" commonly used in slope analysis. The following assumptions are made:

1. The interslice shear forces are assumed equal to zero

2. The normal force at the base of each slice is derived by summing vertical forces for the slice

3. Horizontal force equilibrium for the whole slope is used to derive the factor of safety. In this respect, moment equilibrium for the whole slope is not satisfied. Moment equilibrium methods sometimes encounter difficulties. This occurs when the denominator for the expression for the normal takes on a negative or zero value. This situation can result when the slice inclination angle is negative, or when the ratio of the tangent of the soil angle of internal friction divided by the factor of safety is large. Also if the factor of safety is less than unity and the pore pressures are very large, then the numerator may become negative



*OBTAINED FROM FIGURES 5 & 6.

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NEWMARK'S ESTIMATE OF SLOPE DISPLACEMENT

- EARTHQUAKE ACCELERATION = 0.35g
- VELOCITY 42 cm / sec.

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FIG. 8

4. A correction factor (f_o) is applied to the factor of safety to account for the neglected interslice shear forces, and this correction factor is related to the shear strength parameters and the slope geometry.

The slope is divided up into vertical slices above the assumed failure surface, and, using the assumptions invoked by Janbu's Simplified Method, the forces driving the slope towards failure and those resisting such failure are calculated. Janbu's formula for derivation of the safety factor, FS, is given by:

$$FS = \frac{\Sigma \text{ RESISTING FORCES}}{\Sigma \text{ DRIVING FORCES}} = f_o \left[\frac{\Sigma \frac{(c + (P - u) \quad \tan\phi') \Delta X}{n\,\alpha}}{\Delta W \; (\tan \alpha + Q)} \right]$$

where:

f_o = Correction factor as explained in 7.2.4, above

c = Cohesion of the soil in the slice in question

ϕ' = Angle of internal friction of the slice

P = Average weight of the slice per unit area

u = Porewater pressure at the base of the slice

ΔX = Width of the slice

ΔW = Weight of the slice per unit area

α = Angle that the base of the slice makes with the horizontal

Q = Earthquake acceleration

$$n\alpha = \cos^2\alpha \left[1 + \frac{\tan\alpha \ \tan\phi}{FS_1}\right]$$

FS_1 is the factor of safety assumed for the start of the iteration. The iteration for the computation of the safety factor, FS, is only stopped when the difference between FS and FS_1 is less than 0.01.

7.3 Computation Procedure

The initial slip surface is analysed and its factor of safety found. The first stage of optimization takes the initial plane and, keeping the starting and ending points fixed, sequentially moves its ordinates up or down at each of the sections by the specified amount until the slip surface with the minimum factor of safety is found. The slip surface is established as the potential failure surface. Subsequently, one of the end points is adjusted to start at a new section line and the process of adjusting the intermediate ordinates of the slip surface is repeated. For each set of end points, a slip plane with the minimum factor of safety is determined then compared to the current potential failure surface. If its safety factor is lower, it becomes the new potential failure surface. The program continues through the following pattern of adjusting the end points:

1. The upslope end point is adjusted in both directions;

2. The downslope end point is adjusted in both directions; and,

3. Various combinations of both end points are tried.

At the end of the sequence, described above, the slip surface with the minimum factor of safety is written out by the program, and this is the safety factor recorded for the slope.

7.4 Verification of the Program

Based on the results presented in the following subsections (7.4.1 and 7.4.2), it can be concluded that the Hardy Associates (1978) Ltd. Janbu2 computer program results compare very well to both commercially available computer program results and closed-form computations. This conclusion can be drawn based on the good agreement obtained for the wide range of loading and groundwater conditions considered in the comparitive studies carried out. With respect to closed-form computations, (i.e. infinite slope stability analyses), it can be observed that Janbu2 factor of safety is slightly higher for cohesive soils. This can be attributed to the fact that only a free body element of soil is considered for the closed form solution and therefore slices at the end of the failure surface, which have a smaller downslope gravity component are not considered. However, for frictional materials, this effect is negligible since the overburden weight of the end slices is very small. The profiles utilized for vertification of the computer program are included in Appendix B.

7.4.1 Static Loading Conditions

Verification of the program is summarized in Table II, together with the material properties input to the analyses. The cross sections, referenced by figure numbers, are included as Appendix B. Static loads are those resulting from slope geometry, groundwater conditions and surcharge loads, if applicable, and will be permanently applied to the slope under consideration.

7.4.2 Earthquake Loading Conditions

Earthquake loads are inertia forces calculated utilizing the peak earthquake acceleration multiplied by the total mass. These forces are applied as a constant horizontal force. Verification of the program is summarized in Table III together with the material properties input to the analyses. The cross sections are referenced by figure numbers and are presented in Appendix B.

TABLE II

VERIFICATION OF JANBU2 COMPUTER PROGRAM - STATIC LOADING CONDITION

FIGURE NO.	γ_T (Mg/m^3)	c (kPa)	ϕ (degrees)	ACCEL. (g)	GROUNDWATER CONDITIONS	JANBU2 FACTOR OF SAFETY	COMPARISON FACTOR OF SAFETY	COMPARISON SOURCE/METHOD
1	2.0	0	33	0	Dry	3.13	3.13	CSC/Spencer
2	2.0	0	33	0	Submerged	3.14	3.12	CSC/Spencer
3	2.0	0	28	0	Ground Surface	1.51	1.51	Closed Form/Infinite Slope Analysis
4	2.0	0	30	0	Ground Surface	2.96	2.9	Closed form/Infinite Scope Analysis
	2.0	0	35	0	Ground Surface	3.59	3.5	
	2.0	0	40	0	Ground Surface	4.30	4.2	
5	2.0	0	30	0	Ground Surface	0.91	0.97	Closed form/Infinite Slope Analysis
	2.0	0	35	0	Ground Surface	1.11	1.18	
	2.0	0	40	0	Ground Surface	1.32	1.40	
6	1.9	29(1)	(1)20	0	As shown	1.31	1.30	Can.Geotech.Journal/ Simplified Janbu
	1.6	0(2)	(2)10	0		$\frac{FS}{fo} = 1.22$	1.33	Rigorous Janbu
							1.25	Morgenstern/Price
							1.25	Spencer
7	1.9		(1)34	0	As shown	1.14		
	1.9		(2)26	0		$\frac{FS}{fo} = 1.09$	$\frac{FS}{fo} = 1.17$	Closed Form/Simplified Janubu
	1.9		(3)26.5	0				

TABLE II con't

VERIFICATION OF JANBU2 COMPUTER PROGRAM - STATIC LOADING CONDITION

FIGURE NO.	γ'_T (Mg/m^3)	c (kPa)	ϕ (degrees)	ACCEL. (g)	GROUNDWATER CONDITIONS	JANBU2 FACTOR OF SAFETY	COMPARISON FACTOR OF SAFETY	COMPARISON SOURCE/METHOD
8	1.9		(1)34	0	As Shown	1.50		
	1.9		(2)26					
	1.9		(3)26.5			$\frac{FS}{fo} = 1.44$	$\frac{FS}{fo} = 1.38$	Closed form/Simplified Janbu
9	1.9	0	(1)34.0	0	As Shown	$\frac{FS}{fo} = 1.56$	$\frac{FS}{fo} = 1.48$	Closed form/Simplified Janbu
	1.9	0	(2)26.0					
	1.9	0	(3)26.5					
10	1.9	0	(1)34.	0	As Shown	1.44	1.46	CSC/Fredlund
	1.9	0	(2)30			$\frac{FS}{fo} = 1.37$	$\frac{FS}{fo} = 1.35$	
	1.9	0	(3)26					
	1.9	0	(4)24					
	1.9	0	(5)26.5					

TABLE III

VERIFICATION OF JANBU 2 COMPUTER PROGRAM - EARTHQUAKE LOADING CONDITION

FIGURE NO.	γ_T (Mg/m^3)	c (kPa)	ϕ (degrees)	ACCEL. (g)	GROUNDWATER CONDITIONS	JANBU 2 FACTOR OF SAFETY	COMPARISON	
							FACTOR OF SAFETY	SOURCE/METHOD
1	2.0	0	33	0.10	Dry	2.08	2.08	CSC/Spencer
1	2.0	0	33	0.28	Dry	1.27	1.27	CSC/Spencer
1	2.0	0	33	0.35	Dry	1.10	1.10	CSC/Spencer
3	2.0	0	28	0.28	Ground Surface	0.527	0.524	Closed Form/Infinite Slope Analysis
3	2.0	6.9	28	0.28	Ground Surface	1.082	1.064	Closed Form/Infinite Slope Analysis
4	2.0	0	30	0.35	Ground Surface	0.88	0.87	Closed Form/Infinite Slope Analysis
4	2.0	0	35	0.35	Ground Surface	0.73	0.72	Closed Form/Infinite Slope Analysis
4	2.0	0	40	0.35	Ground Surface	0.60	0.60	Closed Form/Infinite Slope Analysis
4	2.0	4.8	30	0.35	Ground Surface	0.99	0.96	Closed Form/Infinite Slope Analysis
4	2.0	24	30	0.35	Ground Surface	2.55	2.39	Closed Form/Infinite Slope Analysis
4	2.0	0	30	0.35	0.6 m Below Ground Surface	0.87	0.85	Closed Form/Infinite Slope Analysis

TABLE III (Cont'd)

VERIFICATION OF JANBU 2 COMPUTER PROGRAM - EARTHQUAKE LOADING CONDITION

FIGURE NO.	γ_T (Mg/m^3)	c (kPa)	ϕ (degrees)	ACCEL. (g)	GROUNDWATER CONDITIONS	JANBU2 FACTOR OF SAFETY	COMPARISON FACTOR OF SAFETY	COMPARISON SOURCE/METHOD
4	2.0	0	35	0.35	0.6 m Below Ground Surface	1.05	1.03	Closed Form/Infinite Slope Analysis
4	2.0	0	40	0.35	0.6 m below Ground Surface	1.26	1.24	Closed Form/Infinite Slope Analysis
5	2.0	0	30	0.35	Ground Surface	0.33	0.35	Closed Form/Infinite Slope Analysis
5	2.0	0	35	0.35	Ground Surface	0.40	0.43	Closed Form/Infinite Slope Analysis
5	2.0	0	40	0.35	Ground Surface	0.48	0.51	Closed Form/Infinite Slope Analysis
5	2.0	4.8	30	0.35	Ground Surface	0.62	0.62	Closed Form/Infinite Slope Analysis
5	2.0	24	30	0.35	Ground Surface	1.75	1.68	Closed Form/Infinite Slope Analysis
5	2.0	0	30	0.35	0.6 m Below Ground Surface	0.53	0.53	Closed Form/Infinite Slope Analysis
5	2.0	0	35	0.35	0.6 m Below Ground Surface	0.64	0.64	Closed Form/Infinite Slope Analysis
5	2.0	0	40	0.35	0.6 m Below Ground Surface	0.76	0.77	Closed Form/Infinite Slope Analysis

TABLE III (Cont'd)

VERIFICATION OF JANBU2 COMPUTER PROGRAM - EARTHQUAKE LOADING CONDITION

FIGURE NO.	γT (Mg/m^3)	c (kPa)	ϕ (degrees)	ACCEL. (g)	GROUNDWATER CONDITIONS	JANBU2 FACTOR OF SAFETY	COMPARISON FACTOR OF SAFETY	COMPARISON SOURCE/METHOD
11	1.8	12	11	0.45	Ground Surface	0.527 $\frac{FS}{fo} = 0.50$	0.49	Dames & Moore - Alyeska
12	1.6 (1) 2.0 (2) 2.0 (3) 2.2 (4)	0(1) 34(2) 0(3) 0(4)	(1)11 (2)20 (3)35 (4)37	0.20	As Shown	$\frac{FS}{fo} = 1.49$	$\frac{FS}{fo} = 1.45$	Closed Form/Simplified Janbu
12	1.6(1) 2.0(2) 2.0(3) 2.2(4)	12(1) 34(2) 0(3) 0(4)	(1)11 (2)20 (3)35 (4)37	0.30	As Shown	$\frac{FS}{fo} = 1.16$	$\frac{FS}{fo} = 1.12$	Closed Form/Simplified Janbu

8.0 MITIGATIVE MEASURES

Subsequent to the preliminary geotechnical assessment and field reconnaissance program, several re-routes of the pipeline were implemented in order to avoid potentially unstable slopes. The two major considerations in re-routing were the evaluation of the requirements for slope stabilization and the additional length of pipe involved. Stabilization techniques for frozen cut slopes and other mitigative measures (Lambe, 1962; Hutchinson, 1977) that may be considered are discussed in the following subsections.

8.1 Field Testing of Frozen Cut Slopes Stabilization Techniques

Area "E" of the Quill Creek test site, constructed by Foothills during the winter 1980 - 1981, provides full scale field tests of various types of cutslope stabilization techniques in ice-rich soils. These techniques are envisaged for the portions of the Yukon pipeline route traversing ice-rich slopes. The soil exposed in the slopes at the test site consist primarily of ice-rich silts and the geotechnical investigation data has been presented by Hardy Associates (1978) Ltd. (1981). Instrumentation aimed at monitoring the behavioural characteristics of both natural and stabilized slopes consists primarily of thermistors and slope indicators.

Four stabilization techniques for five different conditions have been employed and are described in the following:

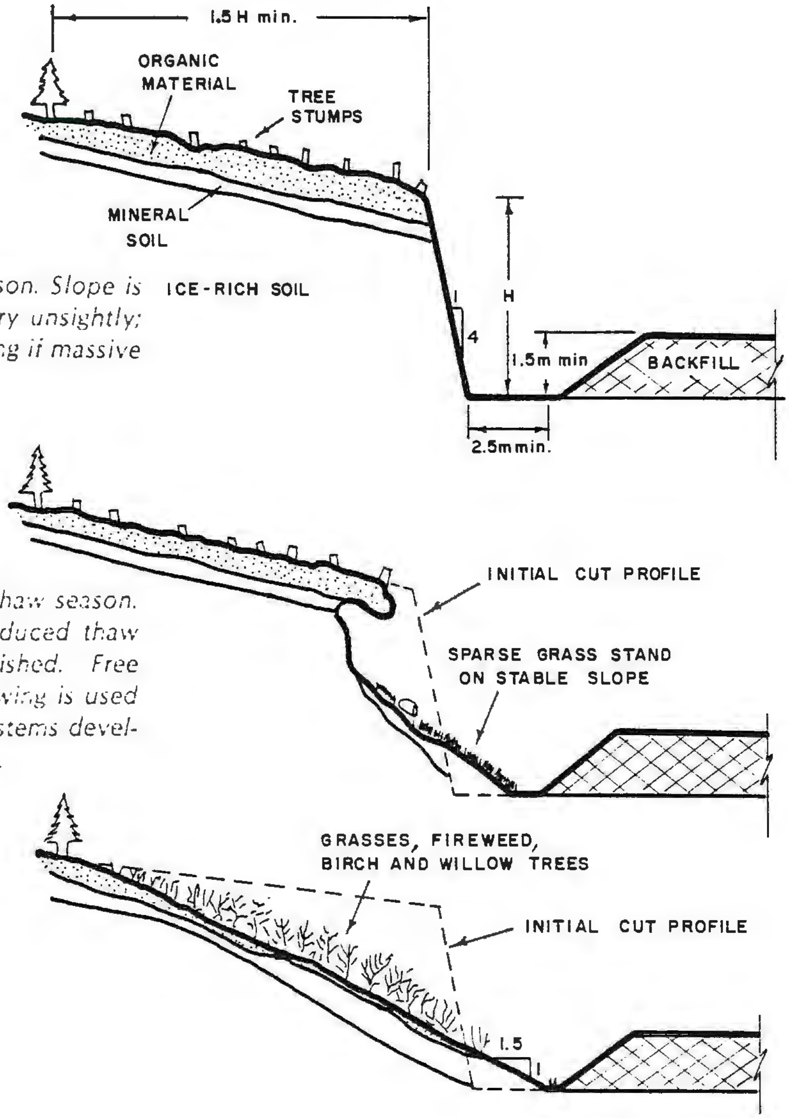
1. (a) Vertical Cut - approximately 6 m high. The trees were cut down for some distance behind the crest of the initial cut-slope. Stabilization of this cut in these soil types is anticipated to occur naturally over some time (Brown and Berg, 1980). The configuration and process is shown diagramatically in Figure 9. To date, this cut has performed as anticipated. The icy soils in the backslope have thawed and sloughed to the base of the slope. It is expected that the slope will ultimately stabilize at an angle of about 1.5 to 1.

   The natural stabilization technique is most applicable when ice distribution in the soil is in the form of lens or wedges, and the soil matrix is not a high plastic clay or silty clay. As a consequence, its use should be reviewed in the field, prior to application, in conjunction with site specific soil-ice conditions.

   (b) Vertical Cut - approximately 2 m high. The anticipated stabilization process is similar to that described in (a) above. In addition, a wire mesh net was nailed to the tree-stumps in the area immediately behind the crest.

AFTER BROWN AND BERG, 1980

a. Initial frozen cut profile.



b. End of first thaw season. Slope is mostly unstable and very unsightly; ditch will require cleaning if massive ice is present.

c. End of fifth or sixth thaw season. Slope stabilizes with reduced thaw and vegetation established. Free water from minimal thawing is used by plants whose root systems develop new organic material.

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IDEALIZED STABILIZATION OF ICE-RICH CUTS

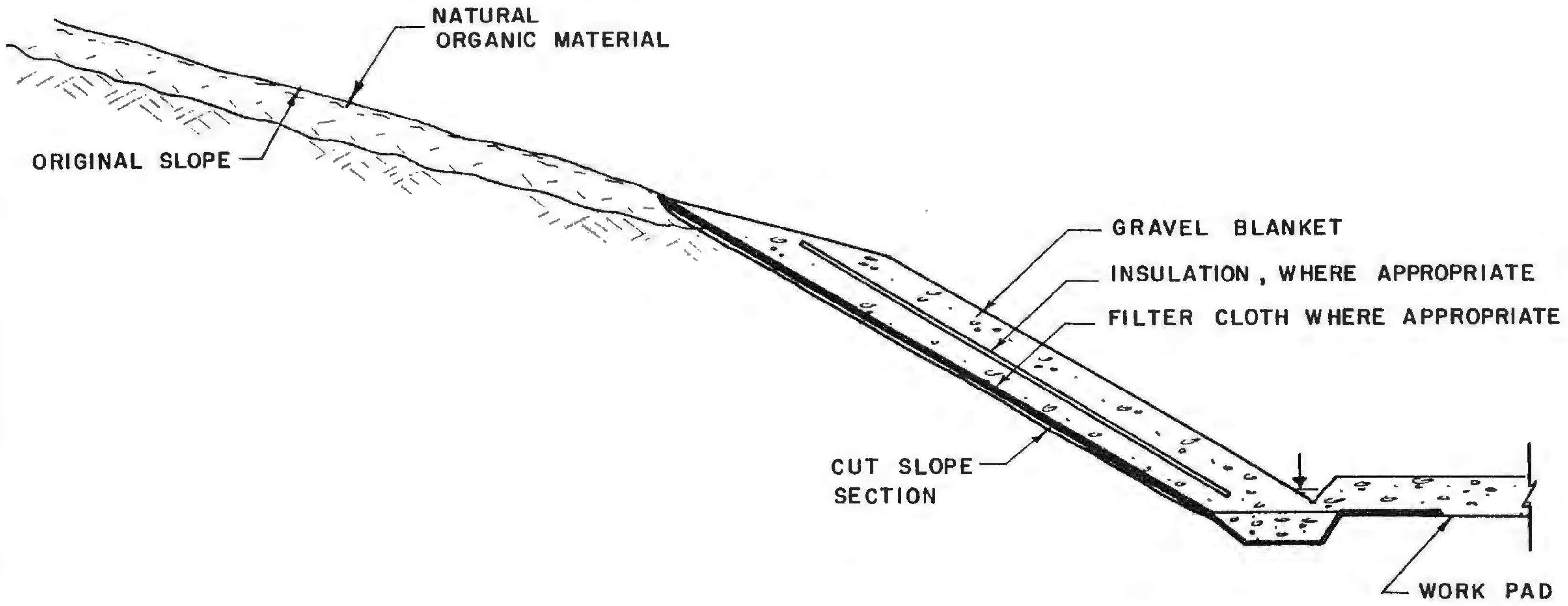
K 5501-E

FIG 9

2. <u>2:1 Cutslope - Revegetated.</u> In this section, the cut is approximately 8 m high and has been flattened to form a slope 2 horizontal to 1 vertical. Subsequently, the slope face has been revegetated.

3. <u>Gravel Blanket.</u> The stabilization technique is the same as that described in 2, above. However, instead of revegetating, a 1.8 m thick gravel blanket was placed on the slope face. The conceptual gravel blanket configuration for cutslope protection is shown on Figure 10. At the test site location, no filter cloth and insulation were considered necessary.

4. <u>Gravel Buttress.</u> At this location, the cut is approximately 4 m high and has been flattened to 2 vertical to 1 horizontal. Subsequently, a gravel buttress 1.5 and 4.5 m wide at the crest and base, respectively, was added. The conceptual gravel buttress configuration is presented diagrammatically on Figure 11.



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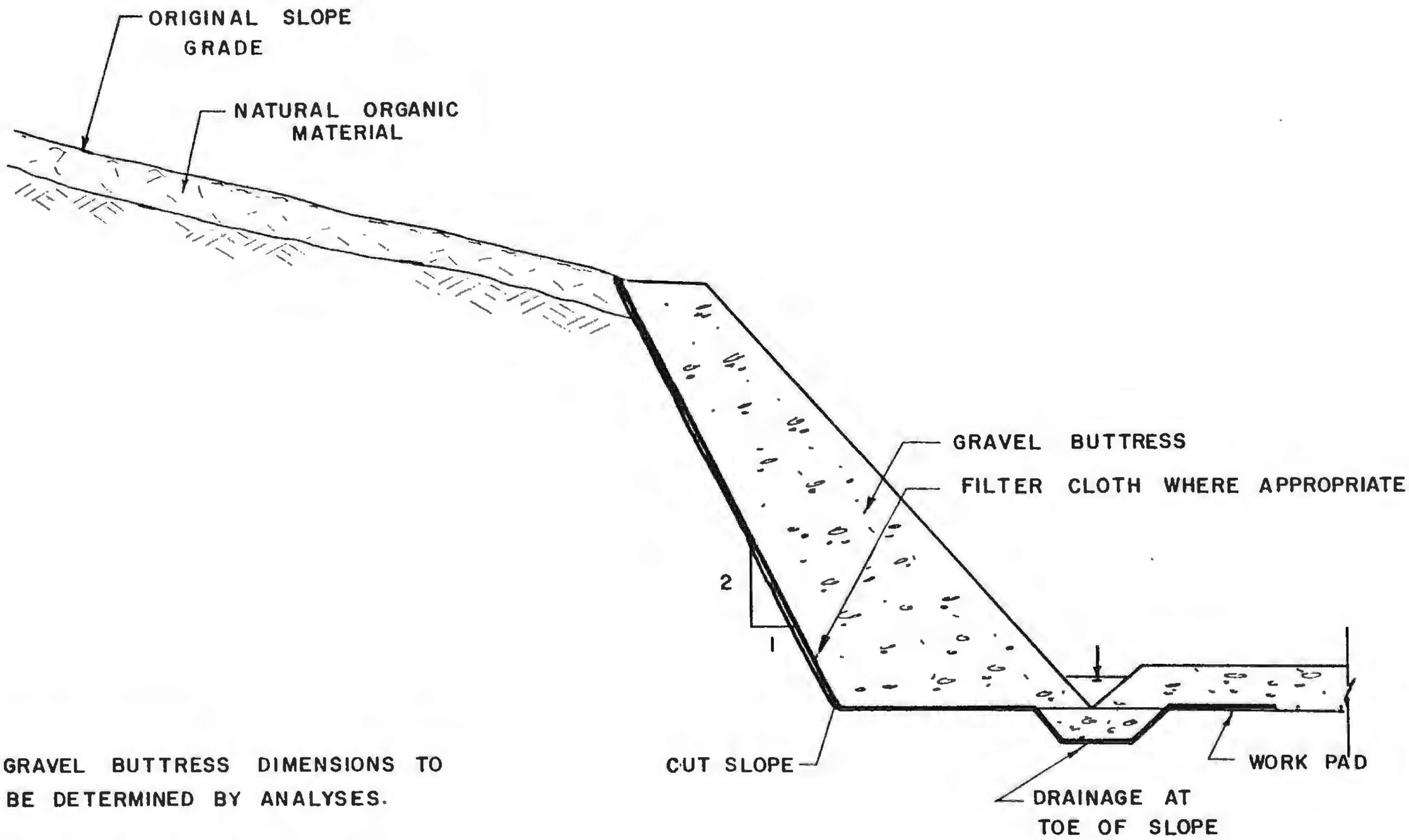
CONCEPTUAL GRAVEL BLANKET CONFIGURATION FOR CUT SLOPE PROTECTION

K 5501- E

FIG. 10



NOTE: GRAVEL BUTTRESS DIMENSIONS TO BE DETERMINED BY ANALYSES.

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CONCEPTUAL GRAVEL BUTTRESS CONFIGURATION

K 5501 - E

FIG. 11

8.2 Frozen or Thawing Slopes

8.2.1 Prevent or Limit Thaw

In longitudinal areas where ice-rich subsoils are encountered below shallow depths, thawing beneath the pipe will be limited using an above-grade construction mode. Therefore, slope instability beneath or adjacent to the pipe will not be a concern.

On natural slopes on the pipeline R.O.W. where clearing and construction activity has taken place, some limited thawing may occur in the long term. The stability of these longitudinal slopes will account for the limited deepening of the permafrost table that may occur over long periods (see Section 5.2.3 of Part I).

Thawing will be prevented or limited, as appropriate, on cut slopes through construction of proven and adequately designed stabilization techniques, as described in Subsection 8.1.

8.2.2 Regrading

Unstable slopes could be flattened to increase their static and seismic safety factors. Regrading would be applicable to short slopes of relatively small height, and at locations where permafrost degradation would not result in any adverse effects on the pipeline and environment.

8.2.3 Surcharging

Any surcharge added to the toe of a slope will result in increased stability. Engineered surcharge in the form of gravel blankets or regrading in conjunction with gravel backfilling may be used as means of slope stabilization.

8.2.4 Restraining Structures

Buttresses constructed of free-draining granular materials are anticipated to be the primary type of restraining structure for permafrost slopes along the route, where working space is restricted. A conceptual configuration for the gravel buttress has been presented on Figure 11. Synthetic fabrics placed beneath a gravel blanket may be used to form a filter layer between the free-draining granular material and the underlying finer soils.

8.2.5 Groundwater Control and Drainage

This type of slope stabilization method would form an integrated part of Drainage and Erosion Control. Water from thawing cutslopes will not be allowed to pond on the pipeline right-of-way. On longitudinal slopes, water resulting from thaw or surface run-off would be diverted from the pipe ditch.

8.3 Unfrozen Slopes

8.3.1 Excavation and Filling

Loose or soft materials on unstable slopes can be removed and replaced either wholly by free-draining materials or, by the recompacted loose material.

Excavation, at the crest of the slope, will to some extent unload the slope and increase its stability. Filling to load the slope will generally be accomplished by surcharge berms. It is important to achieve the correct positioning of the stabilizing structures to achieve proper drainage.

8.3.2 Drainage

Proper drainage will ensure that surface water is directed away from the pipeline right-of-way. It is not anticipated that other slope drainage procedures will be required. They may be utilized, however, for specific situations and would consist of either trench drains or horizontal drains. The trench drains would be set sufficiently deep to intercept groundwater flowing towards the slope. Conventional horizontal drains may be used either above or in conjunction with vertical drains.

8.3.3 Restraining Structures

Restraining structures may play an important role in slope stabilization where space is restricted. These structures would consist primarily of gravel buttresses and would be used to provide added shearing resistance to cross slopes. The buttress materials would be free-draining in order to ensure that there would be no build-up of excess pore pressures.

Other types of restraining structures are not envisaged as the majority of soil slopes close to the pipeline are not excessively high.

8.3.4 Miscellaneous Methods

These methods consist of treating the soil to increase its strength. The miscellaneous methods of soil improvements can be classified as either mechanical (compaction, addition or removal of soil particles), chemical (injection of chemicals), or electrical (electro-osmosis).

The only method envisaged at present is compaction, if required.

8.3.5 Erosion Control

Erosion control is fundamental to the prevention of slope failures since the close link between mass movement and erosion is well known. Erosion involves the removal of surface stabilizing soil and vegetation layers, leaving the slope more susceptible to instability.

Where the toe of a slope is located near a lake or a river, revetments may be used to prevent erosion at critical locations.

Surface erosion on slopes will be prevented by the proper design of drains or diversion structures for surface water. In addition, surface rehabilitation or revegetation may be implemented where required.

Respectfully submitted,

HARDY ASSOCIATES (1978) LTD.

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PART II

SEISMIC LIQUEFACTION STUDIES

1.0 INTRODUCTION

1.1 Definition

Seismic liquefaction describes the behavior of saturated cohesionless soils subjected to earthquake-induced shaking. These soils may lose some of their shear strength, and may acquire characteristics of a viscous liquid mass with flow capabilities.

A loose cohesionless soil deposit will tend to compact and decrease in volume when subjected to seismic motion. If the soil is saturated, and the boundary conditions of the deposit do not permit rapid drainage of excess water into adjoining strata, the pore water pressure will increase. When this increase equals the confining pressure, initial liquefaction is defined as having occurred. When the duration and intensity of shaking are sufficiently large, the pore water pressure increase may be sufficient to produce an almost complete loss of strength. This may result in a collapse of the intergranular structure, or failure by liquefaction. The soil would then undergo large deformations and displacements with little resistance, and would behave like a viscous liquid.

For damaging liquefaction to occur, the reduction in strength of the soil must propagate over a significant area and depth within the profile. Portions of the profile with greater resistance to liquefaction would prevent this propagation, thereby limiting the extent of damage. Without the development of initial liquefaction, failure by liquefaction is impossible. However, the converse is not true. Dense soils cannot strain extensively without dilation, and soils with some of their strength derived from cohesive properties will not develop large strains during an earthquake, even if initial liquefaction is achieved. The term initial liquefaction is an effective research parameter which requires careful interpretation for practical applications.

Factors which affect the onset of liquefaction and the severity of the potential damage include:

1) intensity and duration of shaking;
2) soil properties;
3) in situ stresses;
4) groundwater conditions; and
5) topography.

Field investigations and laboratory tests indicate that soil strength would be re-established after termination of the ground motion and dissipation of the excess pore water pressures.

1.2 <u>Effect</u>

When seismic liquefaction occurs on level ground, there is a tendency for objects of greater or lower density than the liquefied soil, to sink or float, respectively. This tendency is resisted by the viscosity of the fluidized soil. An unweighted buried pipeline would tend to float upward, and a heavily weighted buried pipeline; would sink downwards in the liquefied soil mass. This would subject the pipeline to stresses introduced by bending. A buried pipe weighted so as to equalize the densities of a gas-filled line and liquefied soil, would be subjected to relatively small movements in a flat terrain.

When liquefaction occurs in a soil mass which is not confined by adjoining stable strata, lateral sliding or motion in the unconfined direction may occur. In slopes, liquefied soils will tend to move downslope and could initiate slides and movements. Ground movements of limited displacement, flows involving extensive movement of soils, and slope and embankment failures could result from seismic liquefaction. Descriptions of documented modes of liquefaction failures from past events are available in the technical literature.

The proposed pipeline traverses areas where soils with low cohesion, high moisture content, and a shallow ground water table may be subjected to earthquake ground motions. Therefore, the potential for slope movement, or liquefaction-triggered landslides, must be evaluated. The pipeline system must be designed to accommodate or prevent such occurrences to minimize or avoid environmental damage and pipe deformation.

1.3 Purpose

The purpose of the liquefaction studies is to identify areas having a high potential for seismic liquefaction, and to provide input to the design process so that either the area can be avoided or appropriate design measures can be made to eliminate the potential of damage to the pipeline system. The procedure followed to achieve this specific objective includes:

1) the assessment of soil, groundwater, and topographic conditions along the alignment;
2) the expected seasonal or imposed variations of these conditions;
3) the attenuation of earthquake ground motions; and
4) the use of these data in evaluating liquefaction potential.

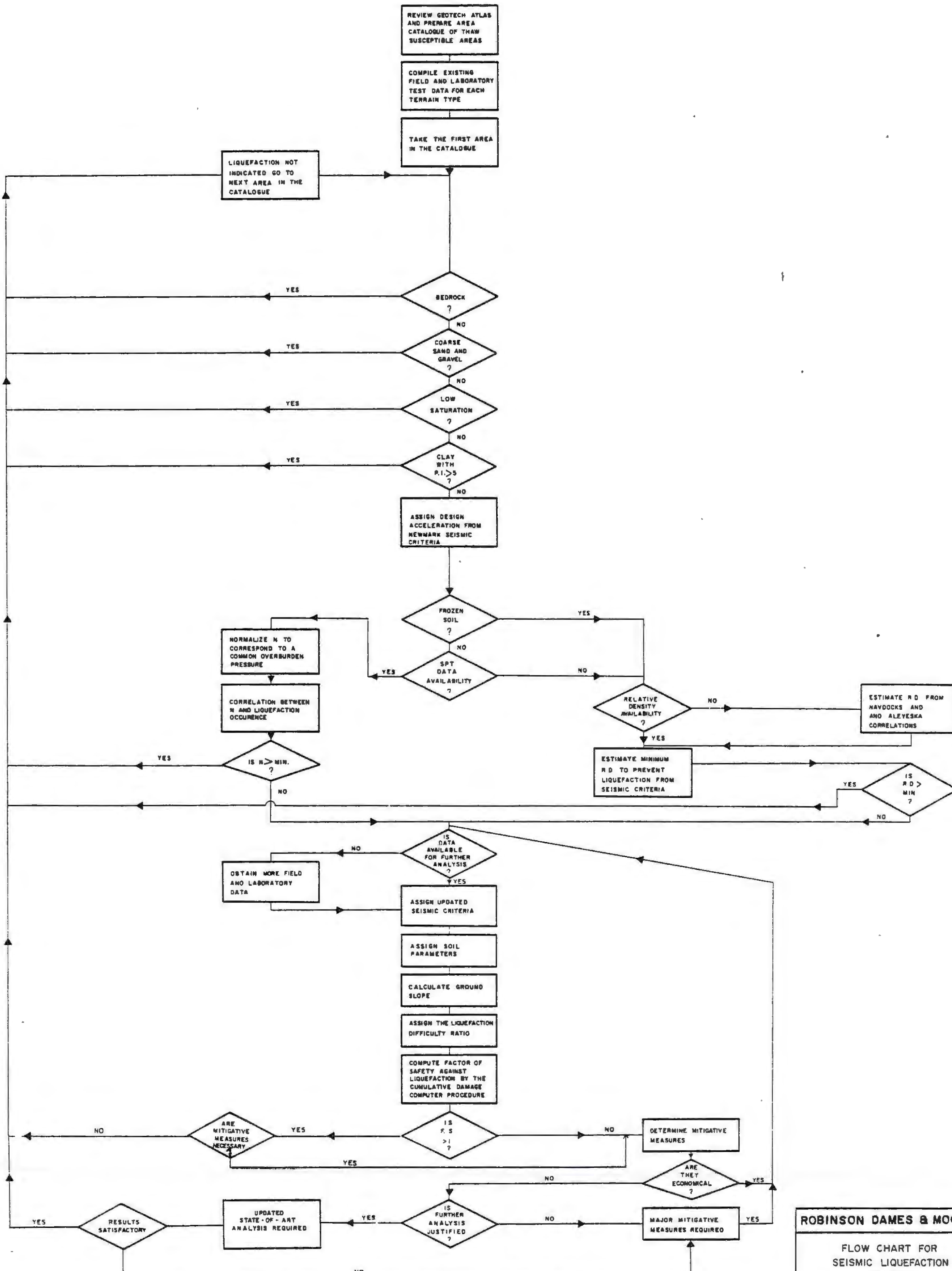
An assessment of liquefaction potential along the alignment is essential in finalizing the pipeline route, and is a basic consideration in the selection of construction modes. It is also necessary to identify locations requiring additional study and/or field exploration before construction, and to locate any areas where realignment or rerouting may not effectively avoid potential problem areas.

1.4 Scope

The scope of the liquefaction studies includes:

1) establishment of input data pertinent to, and necessary for, the evaluation and solution of liquefaction problems;
2) estimation of the potential effects of liquefaction on the pipeline and environment;
3) definition of the design criteria in terms of technical parameters which must be satisifed;
4) development of new, or adoption of existing, procedures to assess liquefaction potential; and
5) application of these results to pipeline design, and use of remedial measures to prevent, or to accommodate, potential liquefaction problems.

The procedure proposed to perform this scope of work is summarized on the flow chart, Figure 12, and described in Section 5.0.



ROBINSON DAMES & MOORE

FLOW CHART FOR SEISMIC LIQUEFACTION STUDIES

CG05501E

FIG 12

2.0 DESIGN CRITERIA

2.1 Liquefaction Potential

"Non-liquefaction" conditons will exist where any of the following criteria are satisfied:

1) frozen soils which would remain frozen;
2) bedrock;
3) coarse sand, gravel, cobbles and boulders;
4) soils that are not saturated; and
5) soils with a plasticity index of at least 5.

Conditions other than those defined above will be analyzed to determine whether the soils are of "low" or "high" liquefaction potential. This determination would be made by methods discussed in the "Analytical Procedures" chapter. Areas with "low" liquefaction potential will be considered safe for construction. Areas of "high" liquefaction potential will require further evaluation and possible design of mitigative measures.

2.2 Topography

The occurrence of liquefaction in major flat areas will not produce difficulties in pipeline integrity and operation, and will not abrogate the environmental concerns. Terrain sloping at two percent or less will be classified as flat. This assumption is based on numerous case histories of liquefaction slides, documented by Casagrande (1970) and others where liquefied mass stabilized at slope grades of two percent or greater. Similar slope grades have been observed at tailing disposal sites. A review of the alignment will be made to ensure that the boundary conditions of flat areas efficiently prevent large ground movements. During liquefaction of these areas, the pipeline will tend to float or sink. Remedial measures will require burial below the liquefiable level, or weighting to neutralize buoyancy.

Slope categories can be divided into:

1) longitudinal, nearly parallel with the pipeline;
2) cross-slope, at an angle to the pipeline.

If liquefaction occurs on sloping ground, the soil mass may undergo movements of sufficient magnitude to result in unacceptable deformation of affected structures. Therefore, remedial measures are required, either to prevent liquefaction, or to provide design sufficient to withstand the imposed forces.

2.3 Ground Types

Ground types can be considered in three categories with respect to the pipeline:

1) thawed, where existing ground is not frozen;
2) frozen, where existing ground is, and will remain, frozen;
3) thawing, where existing ground is frozen, but could thaw out during construction and operation.

Permanently frozen areas do not require analysis, because the shear strength of frozen soil is sufficiently large to resist both gravitational and earthquake forces. Thawing areas are defined as presently frozen areas which are, or will be, thawing, because of natural or man-made causes. Environmental changes, such as clearing, covering or removing the organic mat, or grading, will generally initiate thawing unless protective measures are taken. All frozen areas will be analyzed as thawing areas, unless the design mode is such that they will remain frozen throughout the lifetime of the pipeline.

2.4 <u>Soil Types</u>

Coarse grained soils, such as coarse sands, gravels, cobbles and boulders, which are in close proximity to drainage areas, are generally not susceptible to liquefaction. Because of their high permeability, these soils can be rapidly drained of excess water, thereby preventing the buildup of pore pressure.

Fine grained soils, such as clays and fine silts, are generally less susceptible to liquefaction. These materials have sufficient cohesive strength to resist grain movement, which could result in the development of excess pore pressures. This condition is considered to be satisfied when the plasticity index is greater than five.

Medium grained soils, such as fine to medium sands, and coarse silts, are considered to be more susceptible to liquefaction. They cannot drain at a rate sufficient to prevent the buildup of pore pressures and do not have sufficient cohesive strength to resist grain movement.

2.5 Saturation

Saturation of the soil, or the presence of a high groundwater table, is a requirement for liquefaction to occur. For soils of relatively low permeability, such as fine to medium sands and coarse silts, poor drainage would result in the buildup of pore pressures during a seismic event. Fine grained soils which are presently frozen and rich in ice content would become saturated when thawed, and could liquefy.

2.6 Seismicity

Seismic zones and acceleration parameters would be obtained from Design Contingency Earthquake criteria developed by Newmark (1980), and updated by Robinson Dames & Moore (1981). The design contingency earthquake is the most severe earthquake which could occur near the project. The pipeline system is to be designed so as to minimize damage to the system and to ensure monitoring, communication, control, and orderly shut-down, if necessary, after such an event. The Design Operating Earthquake is the seismic event through which the pipeline should be able to operate during, and continue operation after, its occurrence.

The alignment was divided into three zones on the basis of a seismic hazard classification defined by the Richter Magnitude of the design earthquake. The seismic activity was found to decrease from west to east. Within each zone, the design seismic ground motions would depend on the soil conditions. The conditions can be divided into three types, as tabulated:

1) rock, frozen soil, or limited depth to rock;
2) unfrozen sediments; and
3) soft and loose soils.

Acceleration values were considered to decrease for progressively softer soils. Conversely, displacements were considered to increase for progressively softer soils. Maximum ground velocities were considered to be independent of soil type.

In order to differentiate between seismically induced soil movements and structural deformations, the intensity of motions used for design was considered to have two levels, as shown on Table IV. The higher level is applicable to soil and buried pipeline motions, and the lower level is appropriate for the response of structures and above-ground pipeline elements. Buried pipe would be considered to move with the ground, and to have nearly the same curvature and longitudinal strain as the soil. These requirements impose both compressive and tensile forces in the pipe, as well as lateral bending. This assumption is valid only if the material surrounding the pipe remains intact, and does not liquefy, heave, or settle. Under liquefaction conditions, the pipe may no longer be supported directly by the soil, and further larger deformations must be considered. Liquefaction of the foundation material under the embankment sections of the pipeline could result in movement of the embankment.

2.7 Allowable Movement

The allowable permanent slope movement will depend on the amount of movement that can be tolerated by the pipeline without causing overstressing and rupture, the distance over which the movement would occur, and the direction of movement. The allowable movement of the pipeline will be determined by the design structural engineers.

TABLE IV

Effective Horizontal Ground Motions
Design Contingency Earthquake (After Newmark, 1980)

Zone	Mag.	Accel. %g Rock*	Sediment**	Soft Soil***	Veloc. cm/sec All	Displ. (cm) Rock*	Sediment**	Soft Soil***
A. Soil Response								
a. MP[1] 0-200	7.0	40	35	28	42	28	32	40
b. MP 200-250	6.5	32	27	22	32	20	24	28
c. MP 250-500	5.6	20	15	12	18	10	14	17
B. Structural Response								
a. MP 0-200	7.0	30	25	22	30	18	22	25
b. MP 200-250	6.5	22	18	16	20	11	14	16
c. MP 250-500	5.6	15	12	10	14	8	10	12

* Values to be used for rock, thick permafrost, or less than 10 metres of sediment over rock or permafrost.

** Values to be used for unfrozen sediment depths greater than 30 metres, or for cases of thin permafrost underlain by unfrozen sediments. For intermediate values, use linear interpolation between sediment depths of 10 and 30 m. These values also to be used for soft soil stratum depths of 3 m or less.

***Values to be used when seismic shear wave velocity is less than 700 ft/sec for 10 m or more of soft soil under structure foundation, or in top 10 m if no structure is involved. Interpolate between sediment and soft soil values for depths between 3 m and 10 m.

(1) Mile Post.

3.0 STATE-OF-THE-ART

3.1 Analysis Procedures

The methods presently being used for evaluating the liquefaction of a soil deposit may be classified into three categories:

1) uniform cyclic procedure;
2) cumulative damage procedure; and
3) empirical correlation procedures.

The uniform cyclic procedure is a deterministic method and the cumulative damage procedure is a probabilistic method. Both require laboratory data on the cyclic stresses required to develop either liquefaction, or significant cyclic strains on representative soil samples. These data are usually in the form of the ratio of the applied cyclic shear stress to the effective overburden pressure versus the number of stress cycles required to develop either liquefaction or significant cyclic strains. A comparison of the cyclic stresses induced in the field with those required in the laboratory then permits an evaluation of the factor of safety against liquefaction.

Empirical procedures involve the determination of in-situ soil charateristics as a means of comparing the liquefaction potential of a proposed site with that of other sites where liquefaction is known to have occurred in previous earthquakes.

Almost all published seismic liquefaction evaluation procedures have been developed in the last two decades, following liquefaction damage caused by the 1964 Niigata and Alaska earthquakes. The most recent literature on state-of-the-art were prepared by Valera and Donovan (1977) and Seed (1979). Both include comparisons of the different procedures with examples.

3.2 Uniform Cyclic Procedure

3.2.1 Development

The uniform cyclic procedure was developed by Seed and Idriss (1971). It requires that the analysis be performed deterministically within the time domain, which necessitates the choice of a specific earthquake represented by an acceleration time history. The irregular time history adopted for design purposes is converted into an equivalent uniform cycle shear stress history by using averaging techniques and assuming the shear stress is proportional to the total mass in a unit area column above the depth of interest. Liquefaction is evaluated by comparison with results of laboratory tests corrected to reflect field conditions relating cyclic uniform shear strength to the development of significant strains. This comparison is generally made by using double amplitude axial strain of 20 percent in laboratory cyclic triaxial tests.

Data presented by Seed and Idriss on liquefaction case histories are limited to recorded maximum ground accelerations less than 0.35g and soils with relative densities of less than 70 percent. From pre-earthquake blow-count data, Ross, Seed and Migliaccio (1969) concluded that failure of several bridge foundations during the 1964 Alaskan earthquake were probably attributed to liquefaction. Whitman (1971) estimated that the blow-count data might indicate a relative density of up to 80 percent, but discounted its value in his data set. The uncertainties involved in this interpretation, and the exceeded instances of liquefaction occurrence, illustrate the need to properly evaluate liquefaction potential, as it might affect the gas pipeline.

3.2.2 Limitations

Application of the uniform cyclic procedure is limited to the corresponding data base of recorded liquefaction case histories on level ground and would not readily accommodate the wide range of soils, terrain, and seismic conditions pertinent to this project. In a liquefaction study for the Alyeska project, Dames & Moore estimated the liquefaction potential of soils in seismic and relative density ranges, including those not covered by the published procedures. The study was based on both laboratory tests performed during the project, and case histories. Results correlated very closely with case history data presented by Seed and Peacock (1970) for up to the 70 percent relative density limit. For higher densities, the data showed a non-linearity with an increase in resistance to liquefaction, indicating that direct extrapolation, based on published data, would be unreasonably conservative.

Other difficulties with the deterministic approach arise from the inability to account for the random nature of earthquake ground motions, and the problems associated with considering all earthquakes as having similar duration and response characteristics. The advantages of supplementing the knowledge of high-density soil behavior under high seismic accelerations, and the development of a more versatile analytical methodology capable of accounting for more parameters influencing soil response, became apparent. Such parameters include the soil profile, the epicentral distance, and the focal mechanism, as discussed by Husid, Medina and Rios (1969).

3.3 Cumulative Damage Procedure

3.3.1 Development

The cumulative damage procedure was developed by Donovan (1971). A schematic representation of the procedure is shown on Figure 13. It is a probabilistic approach, and is based on the concept that the effect of cyclic loading on a soil is analagous to fatigue effects in structural materials. The procedure considers both the random nature of earthquake motions, and the parameters of soil response. It makes

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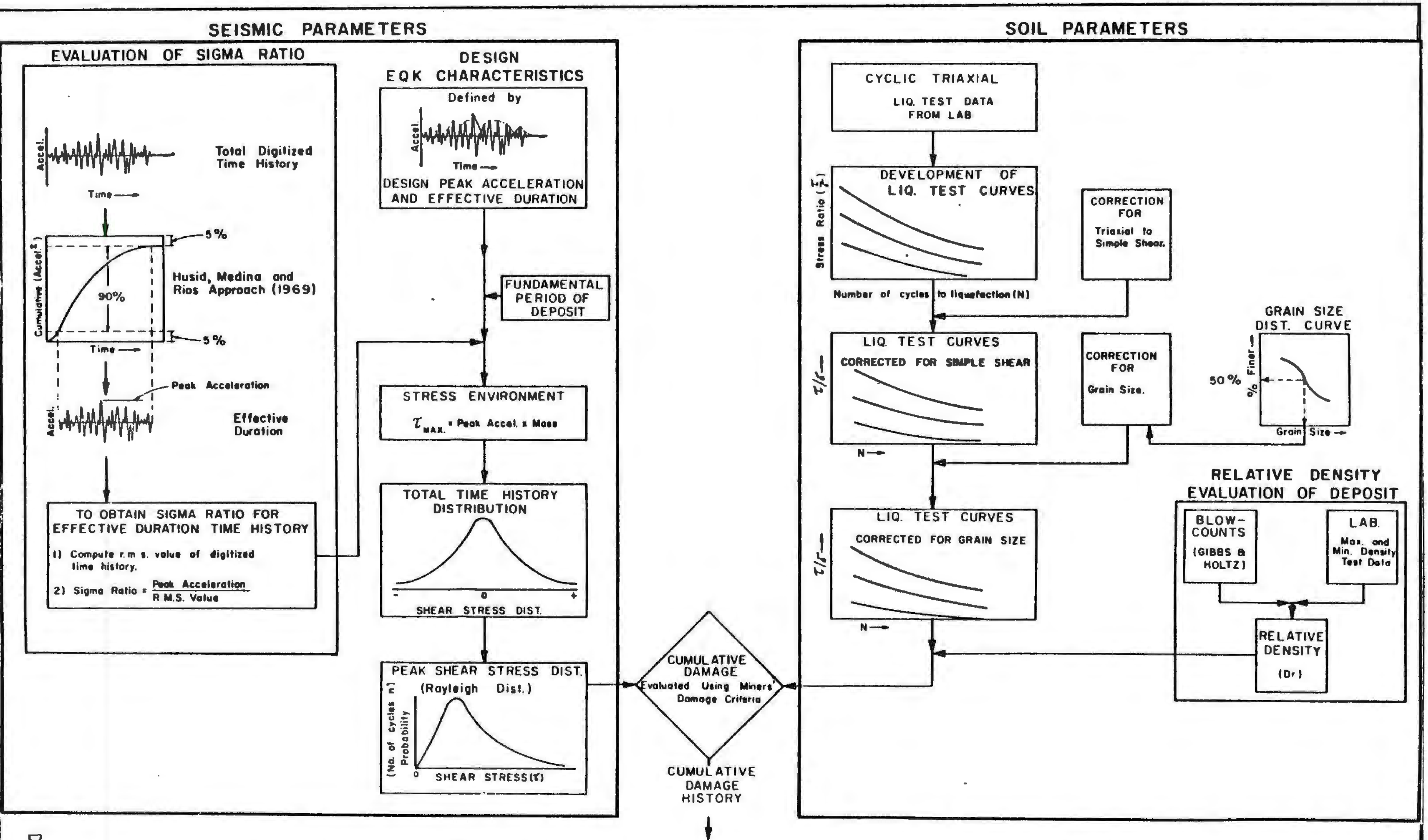


SCHEMATIC REPRESENTATION OF DONOVAN LIQUEFACTION EVALUATION PROCEDURE

ROBINSON
DAMES & MOORE

use of Miner's Damage equation in integral form, which enables it to be versatile and adaptable to digital computers for rapid parametric studies. This is an advantage for a large magnitude project, such as a pipeline where data will be limited, and parametric studies would be necessary to achieve a high degree of reliability.

Laboratory cyclic loading tests of soil samples in either triaxial compression, or simple shear, are used to determine the cyclic deviator, or shear stress, and the number of corresponding cycles to cause liquefaction failure. The phenomenon, the tests, and the resulting relationships bear many similarities with corresponding aspects of metal fatigue. A stochastic approach, similar to that used in the analysis of structures subjected to random loadings by wind gusts or wave action, was applied to the evaluation of liquefaction potential. The procedure incorporated the results of the Alyeska liquefaction studies, which provided liquefaction test data for a wide range of soil types and densities. The procedure was initially verified by its use on the case history data published by Seed and Peacock (1970). Although the character of earthquake motion is generally non-stationary, Donovan (1971) showed that the portion of the strong motion which causes liquefaction of susceptible soils can be considered as a random stationary process.

3.3.2 Parameters Required

The statistical parameters incorporated into the procedure are obtained from real earthquake records. The effective duration of shaking is estimated from the method proposed by Husid, Medina, and Rios (1969), which uses the central portion of an earthquake time history. The digitized acceleration values of the total time history are squared and cumulatively added with increasing time. The duration is then assumed to be that portion of the record between 5 percent and 95 percent of the total summation. The distribution of accelerations or shear stresses in a time history extending through the effective duration approximates a normal or Gaussian distribution.

Liquefaction is believed to be controlled by the maximum value of the shear stress reached at each cycle of loading. The distribution of peak shear stresses obtained by response to the ground motion can be approximated by a Rayleigh distribution. If the standard deviation (root-mean-square value) of the time history is known, the Gaussian and Rayleigh distributions can be obtained. Because earthquakes are usually specified in terms of a peak acceleration value, the most probable distribution of stresses must be obtained. This can be achieved by using the ratio between the maximum peak value, and the root-mean-square value which we have called the sigma ratio.

The number of expected cycles of shaking is obtained by dividing the design duration by the fundamental period of the soil site. The fundamental period is dependent on the soil profile. The probability of any stress level being obtained is the product of the Rayleigh distribution and the expected number of cycles. The distribution of shear stresses can then be incorporated with the liquefaction characteristics of the soil to obtain, by the use of Miner's rule, the cumulative damage and the probability of liquefaction occurring.

3.3.3 Analytical Procedure

At any depth in the soil profile, the shear stress time history, $\tau_h(t)$, can be closely represented by the equation:

$$\tau_h(t) = xa(t)$$

where h = depth below ground surface

t = time

$a(t)$ = surface acceleration at time t

x = soil and water mass in a unit area column above h.

The probability density function of the stress history envelope using the Rayleigh distribution is:

$$p(\tau_h) = \frac{\tau}{x^2 \sigma_{a(t)}^{\;2}} \; \mathrm{EXP}\left[\frac{\tau^{\;2}}{-2x^2 \sigma_{a(t)}^{\;2}}\right]$$

where: $\sigma_{a(t)}{}^2$ = standard deviation of $a(t)^2$

$a(t)$ = surface acceleration at time t

The most probable number of stress cycles, N, which will occur during the earthquake, is assumed to be given by:

$$N = \frac{W_o T}{2\pi}$$

where T = fundamental period of the soil

W_o = fundamental frequency of vibration

Miner's Linear Damage criteria relates damage by the relationship:

$$\text{Damage} = D = \frac{m1}{M_1} + \frac{m2}{M_2} + \ldots\ldots \frac{m_n}{M_n}$$

where: m_1 = number of cycles of loading occurring at stress level 1

M_1 = number of cyles required to produce failure at stress level 1

Because the probability density function is continuous, the damage expression must be represented in integral form

$$D = \int_0^{\infty} \frac{m(\tau)}{M(\tau)} \, d\tau$$

where $m(\tau) = \frac{\omega T}{2\pi} p(\tau)$

and $M(\tau)$ is the liquefaction criteria relating the dimensionless stress ratio and the number of cycles to cause liquefaction, as shown on Figure 14.

The damage expression therefore becomes:

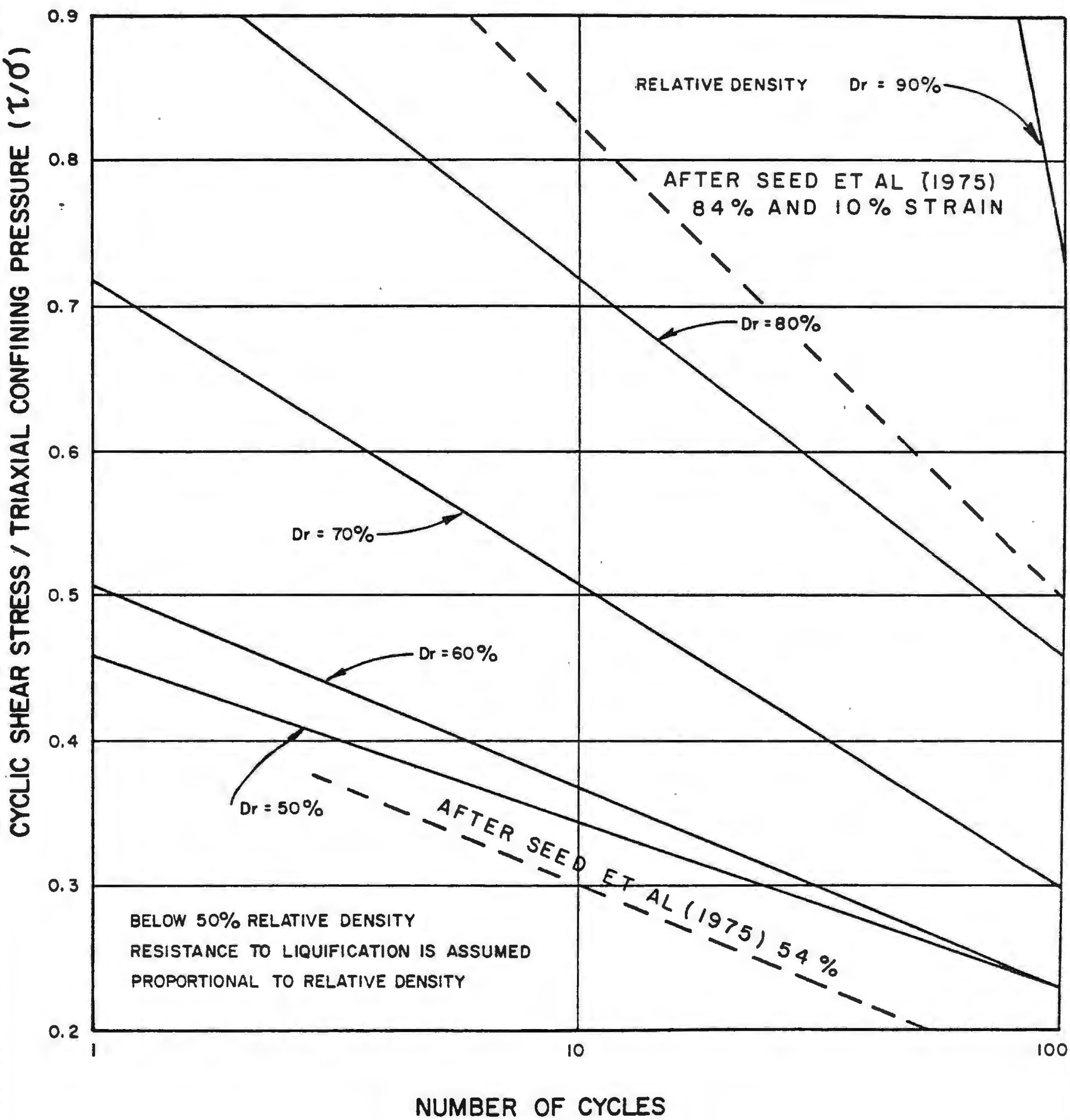
$$D = \frac{\omega T}{2\pi} \int_0^{\infty} \frac{\tau}{M(\tau) x \sigma_a^2} \text{EXP} \left(- \frac{\tau^2}{2x^2 \sigma_a^2}\right) d\tau$$

The integration of this expression can be readily accomplished by a digital computer. The interpretation is that when $D \geq 1.0$, liquefaction will occur.

3.4 Empirical Methods

3.4.1 General

Empirical methods involve the comparison of site conditions with field conditions where liquefaction did or did not occur during previous earthquakes. Seed and Peacock (1970) compared the relative density of sand deposits with field values of the cyclic stress ratio , where is the average cyclic shear stress induced by the earthquake,



AN IDEALIZED RELATIONSHIP BETWEEN STRESS RATIO AND THE NUMBER OF CYCLES REQUIRED TO PRODUCE 20% AXIAL STRAIN

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DAMES & MOORE

FIGURE 14

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and is the effective vertical stress at the depth of induced stress. The induced stresses were computed using the estimated ground surface accelerations. Although relative density is suitable for laboratory studies of liquefaction behavior of a uniform soil, computation of this parameter for natural soil deposits is difficult and may result in significant errors. Because relative density was usually determined from standard penetration tests, recent studies have used the test blow counts directly to evaluate liquefaction potential.

3.4.2 Standard Penetration Test

Japanese investigators, such as Ohsaki (1966), Kishida (1966), and Koizumi (1966), were the first to apply standard penetration test data directly to evaluate liquefaction potential. The first correlation involving an extensive number of sites was published by Seed and Peacock, and has formed the basis of subsequent correlations by Castro (1975), Christian and Swiger (1975), and Seed, Arango and Chan (1975).

The most recent correlation between field liquefaction behavior of sands under level ground, and corrected penetration resistance based on field data and large scale laboratory test data, was published by Seed (1976). For each sand deposit, the cyclic stress ratio associated with its liquefaction potential was plotted against the corrected average penetration resistance of the deposit. The corrected resistance corresponded to the measured penetration resistance corrected to an effective overburden pressure of 98.1 kilopascals, as explained by Gibbs and Holts (1957). The corrected resistance values plotted correspond to the average values of the measured standard penetration resistance within the depth of interest. All data presented corresponded to critical depths of 8 metres or less, with the exception of one where liquefaction was observed at a depth of 17 metres during the 1971 San Fernando earthquake.

The People's Republic of China (1974) have developed a correlation between the liquefaction potential of a saturated sand deposit, and its standard penetration resistance. The average critical value of the standard penetration resistance, N_{crit}, separating liquefiable from non-liquefiable conditions, is computed by the following expression:

$$N_{crit} = \bar{N} (1 + 0.125 (d_s - 3) - 0.05 (d_w - 2)$$

where d_s = depth to sand layer in metres;

d_w = depth of water table in metres; and

$\bar{N}$ = 6 for Design Intensity 7

10 for Design Intensity 8

16 for Design Intensity 9

The Chinese Design Intensity levels are approximately equal to Modified Mercalli Intensities VII, VIII and IX, respectively. The equation is only applicable at depths of 15 metres or less.

3.4.3 Blasting Test

Florin and Ivanov (1961) described the use of a standard blasting test in the U.S.S.R. to evaluate liquefaction potential. For deposits of 8 to 11 metres in thickness, a 5-kilogram charge of ammonite is exploded at a depth of 4.5 metres. The resulting settlements at the ground surface are determined within 5 metres of the explosion. When the average settlement in this zone is less than 8 to 10 centimetres, and the ratio of settlements from successive shots is less than about 0.6, it is considered not necessary to consider liquefaction in the design.

4.0 PARAMETER ASSESSMENT

4.1 General

The evaluation of the liquefaction potential of soils involves the interaction of two sets of parameters:

1) seismic motion; and
2) soil properties as they relate to resistance to liquefaction.

An assessment of liquefaction parameters would be undertaken by consultants with acknowledged expertise in geology, soil mechanics, and earthquake and arctic engineering. The approach would involve reference to many sources of information, including alignment sheets, engineering, geological and seismicity reports, topographic maps, aerial photographs, ground-level photographs, and field trips. The main source of information would be the Geotechnical Atlas, which has been prepared by Foothills Pipe Lines (South Yukon) Ltd. The input would also include the knowledge of regional geology, topography, and soil movement occurrences.

4.2 Topography

Topography would be defined from the Geotechnical Atlas, which includes topographic maps prepared by photogrammetric methods. These maps have contour intervals of 3 metres and horizontal scales of 1 : 10000. They show all existing natural and man-made features, terrain types, and drilled boring locations. Slopes would be calculated for areas which are classified, during the preliminary assessment of the route, as being significant areas. Slopes would be scaled off from the Atlas maps.

4.3 Soil Properties

4.3.1 General

For significant areas of the alignment, the following parameters pertaining to the soil, its physical characteristics, and its state of stress under static conditions would need to be determined:

1) initial stresses;
2) relative density;
3) fundamental period, and
4) liquefaction difficulty ratio.

Field and laboratory data would be obtained from reports prepared by the following consultants:

a) Klohn Leonoff Consultants Ltd. (1976, 1977, 1979);
b) Hardy Associates (1978) Ltd. (1978 - (2), 1979, 1980, 1981)
c) EBA Engineering Consultants Ltd. (1980).

4.3.2 Initial Stress

The initial stress conditions primarily affecting the capacity of soil to resist liquefaction are total overburden pressure, effective overburden pressure, and initial shear stress, if any. The initial soil stress conditions would be estimated by using data related to the geometry of the terrain, groundwater conditions, and soil densities.

4.3.3 Relative Density

Relative density is the principal characteristic of a cohesionless soil which affects its resistance to liquefaction. The tendency of soils to compact under seismic or vibratory loading can be correlated with their relative density. The relative density of frozen and thawed soils at each area under consideration would be estimated by comparing available dry density data with maximum and minimum density test curves produced during the Alyeska project and published by Donovan and Singh (1978). These curves are shown on Figure 15. If density data are not available, dry densities would be estimated from available mositure content data and a zero air voids curve based on a specific gravity of 2.7.

Relative density can also be estimated from standard penetration resistance data obtained on presently thawed ground during field drilling programs. The relationship developed by Gibbs and Holtz (1957), and modified by Seed and Idriss (1971), between standard penetration resistance and relative density, would be used to supplement the laboratory data. These are shown on Figure 16.

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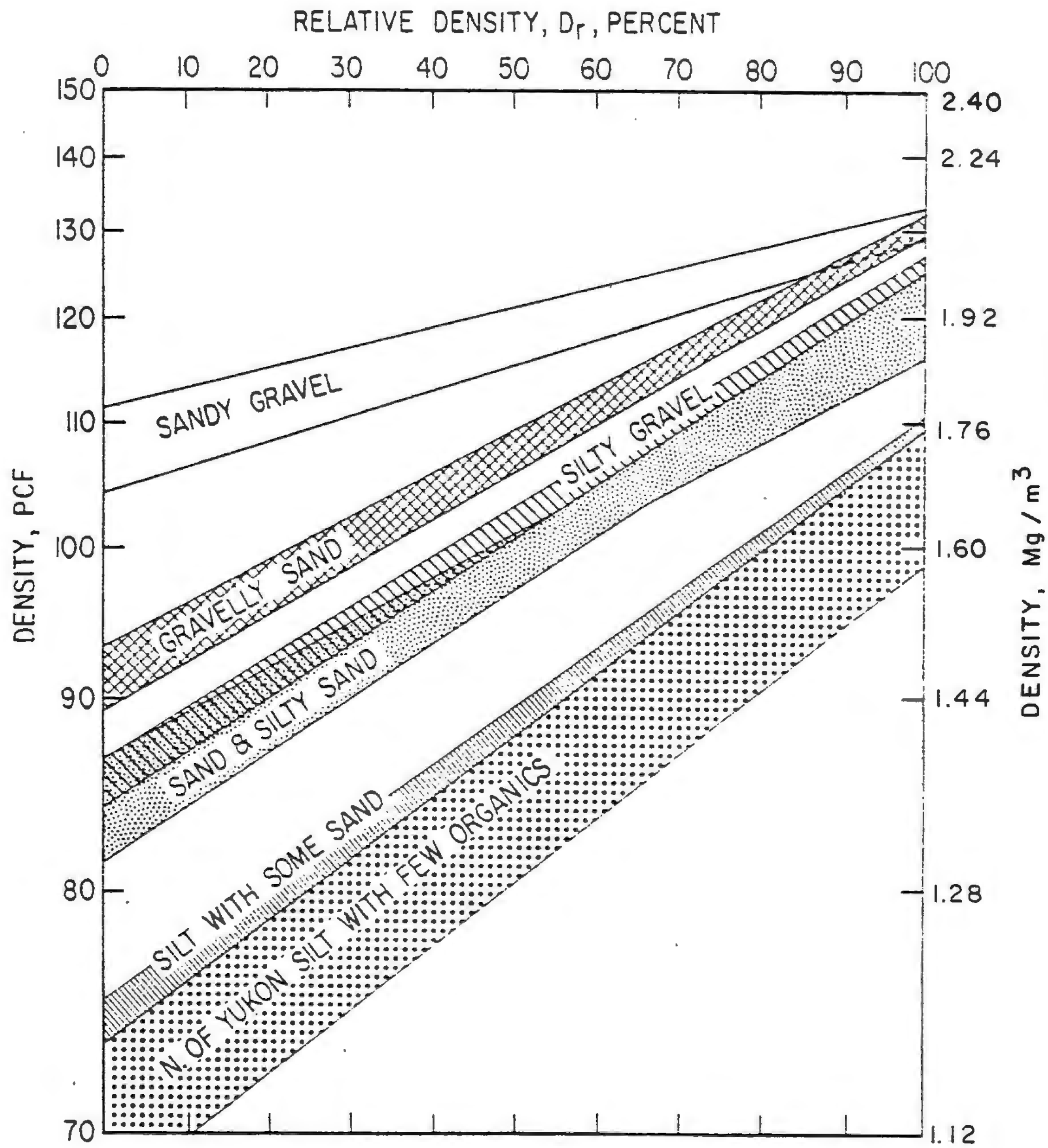

RELATIONSHIP BETWEEN DRY DENSITY AND RELATIVE DENSITY FOR THE ALYESKA PROJECT

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FIGURE 15

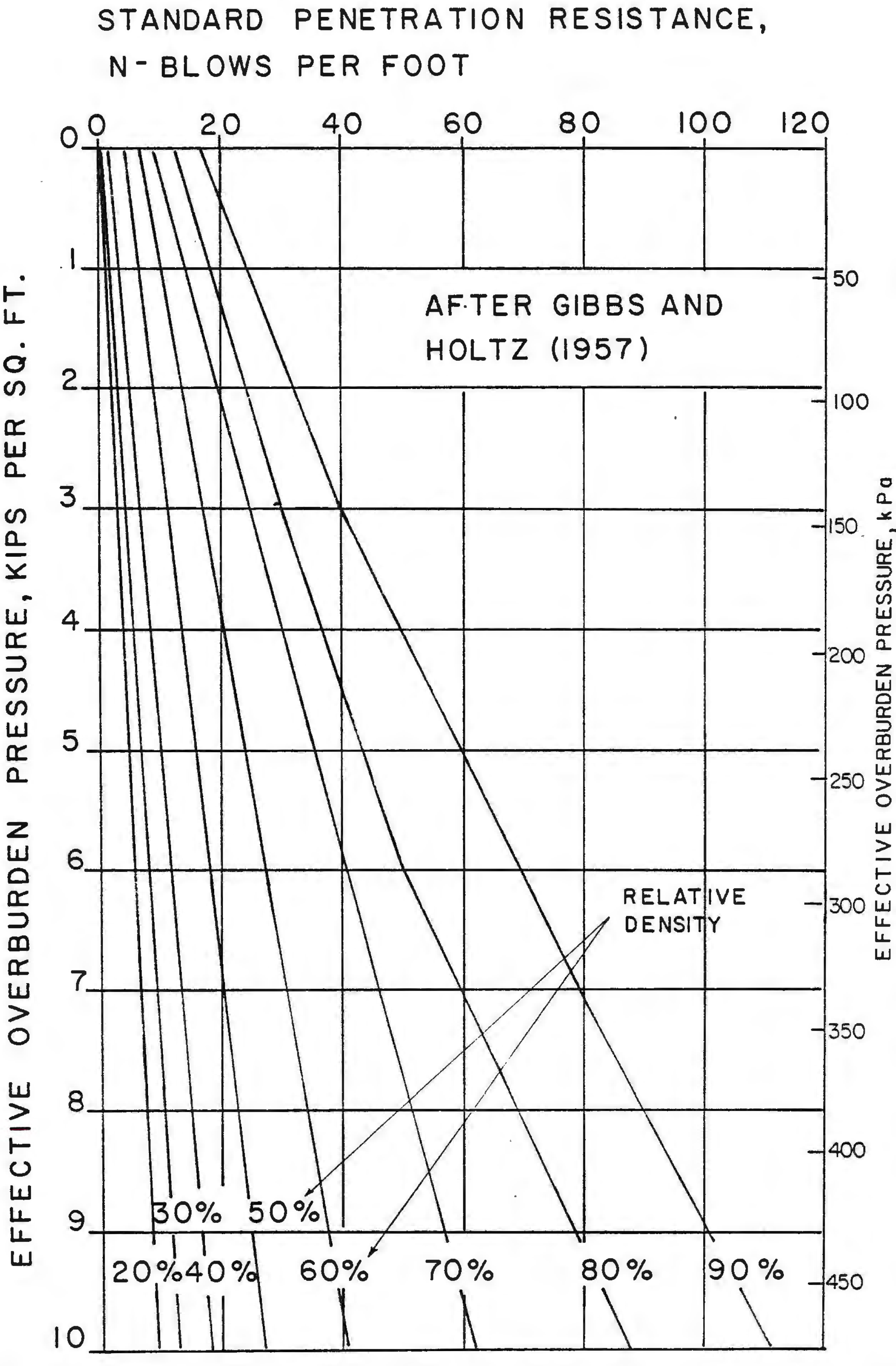

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FIGURE 16

4.3.4 Fundamental Period

The fundamental period of the total soil profile, and the design duration of shaking, determine the number of cycles of earthquake shaking to which the soil would be subjected. The period depends upon depth and physical characteristics of the soil deposit and is evaluated by site response analyses. Donovan (1972) presented an anticipated range of period values for some generic soil profile types. Valera and Donovan (1977) presented values for dense sand and soft clays subjected to the same accelerations as those proposed for this project by Newmark (1980). Detailed site response analyses performed for the Alyeska project were partly documented by Donovan and Singh (1978). On the basis of these references, the maximum period for this project would be limited to 1.0 second for thick and/or weak deposits. The minimum value would be limited to 0.5 second for shallow and/or stiff soil deposits. Because these limits also account for the random energy distribution within earthquakes, they would provide a conservative basis for estimating the number of cycles of shaking.

4.3.5 Liquefaction Difficulty Ratio

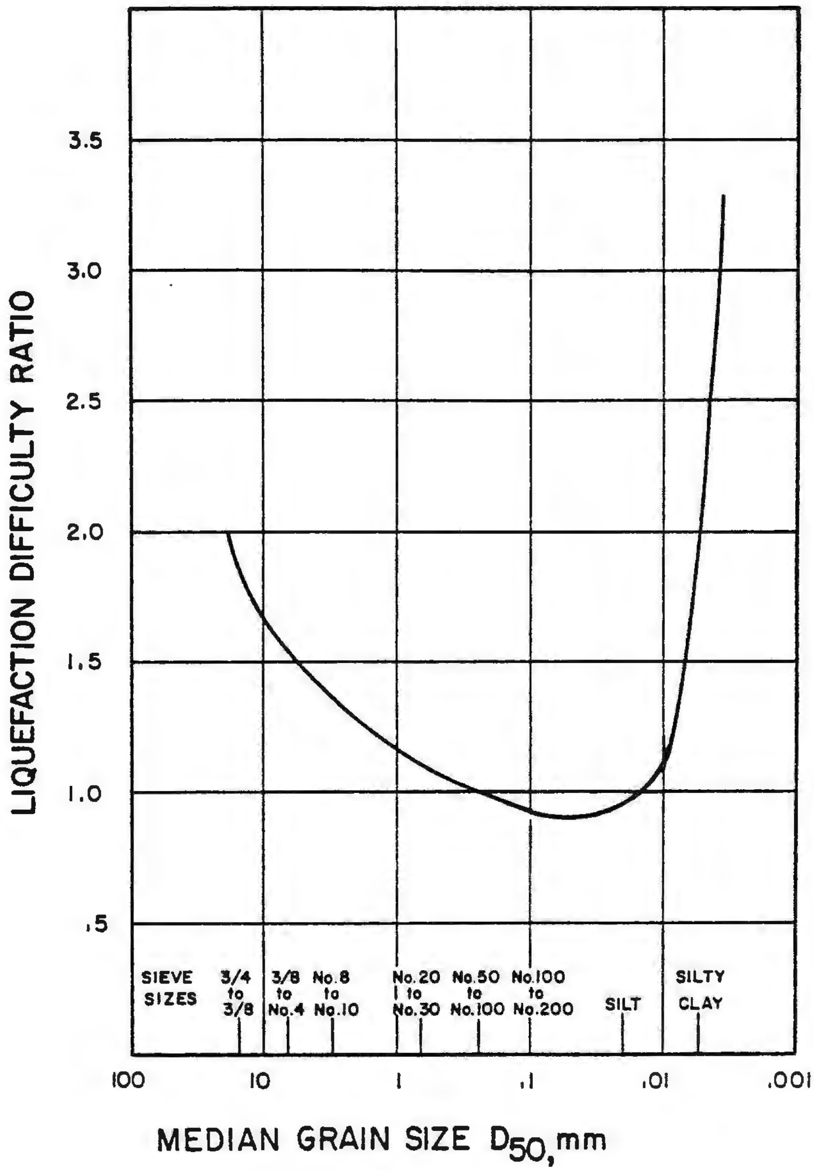
The liquefaction difficulty ratio expresses the soil susceptibility to liquefaction as a function of the grain size distribution. From the work of Lee and Fitton (1969) a medium fine sand with a median grain size, D_{50}=0.2 mm, has been considered as a standard and assigned a liquefaction difficulty ratio of unity. Coarser soils are more difficult to liquefy, and have ratios greater than unity. Finer granular soils are easier to liquefy, and have ratios less than unity. D_{50} is the grain size at which 50 percent of the particles are finer than the median grain size. The relationship between difficulty ratio and mean grain size D_{50} for average soil types, was presented by Donovan and Singh (1978) and is shown on Figure 17. This relationship may be modified in cases where the soils consist of a widely graded mixture of particle sizes.

4.4 Seismic Parameters

4.4.1 Characteristics

Seismic characteristics governing the evaluation of liquefaction potential of soils would be evaluated as follows:

1) design acceleration;
2) duration of shaking;
3) sigma ratio; and
4) number of cycles.



EFFECT OF GRAIN SIZE ON
LIQUEFACTION POTENTIAL
(AFTER LEE & FITTON, 1969)

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FIGURE 17

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4.4.2 Design Acceleration

The maximum ground acceleration used in the analysis directly expresses the severity of seismic shaking and would be used to determine the magnitude of imposed stresses. The design accelerations appropriately attenuated for the limitations of the soil profile properties are taken from the Design Contingency Earthquake criteria developed by Newmark (1980) and presented as Table IV.

4.4.3 Duration of Shaking

The duration directly controls the number of seismic stress cycles to which the soil will be subjected during an earthquake. Methods of evaluating durations have been studied by Housner (1965), Husid, Medina and Rios (1969), Donovan (1972), and Bolt (1973). Donovan computed the effective duration of shaking by using the approach of Husid et al, where the time period, containing 90 percent of the total cumulative acceleration squared, was considered. For earthquakes of Richter Magnitude 5 to 8, the following empirical relationship was developed:

$$D = 4 + 11 \ (M-5)$$

Where D = effective duration, or duration of strong shaking in seconds, and

M = Richter Magnitude of the earthquake.

4.4.4 Sigma Ratio

The sigma ratio is directly related to the distribution of cyclic shear stress peaks in the statistical function representing the earthquake motion. It is obtained by dividing the peak stress by the root-mean-square stress. Sigma ratios have been computed by Valera and Donovan (1977) for various recorded earthquakes. Using total and effective durations, values were found to range from 5 to 12, and 3 to 7, respectively. For a given peak acceleration, a smaller sigma ratio value is indicative of a more severe earthquake. Valera and Donovan recommended that a conservative value of 4 be used in general liquefaction analyses. However, Donovan and Singh (1978) reported that a value of 4.5 was used for the Alyeska project. This value was considered to be conservative for the stipulated requirement of a large earthquake occurring directly beneath the pipeline.

4.4.5 Number of Cycles

The probable number of cycles of shaking can be obtained by dividing the effective duration of the motion by the estimated fundamentatal period of the site. Donovan (1971) showed that this gives a number which is slightly less than would be obtained from more complex analyses, but is compensated for by the conservative assumption of a Rayleigh distribution. Situations where application of the number of cycles obtained in this way can be misleading, will occur in loose or soft soil profiles, with depths greater than 30 metres, or with fundamental periods greater than 1 second. In this situation, the effects of ground vibrations are much more complex than envisioned by the simple vertical shear wave propagation, and must be considered as a special case.

4.5 Saturation

Saturation of the soil, or the presence of a high groundwater table, is a requirement for liquefaction to occur. On the basis of relative density curves produced during the Alyeska project, and published by Donovan and Singh (1978), soils with moisture contents below the following values should be assumed to be either unsaturated or at a sufficiently high relative density so that liquefaction would not occur.

Richter Magnitude	Critical Moisture Content (% of dry weight)					
	Sandy Gravel	Gravelly Sand	Silty Gravel	Sand & Silty Sand	Sand Silt	Silt & Clay
7.0	13	17	21	22	29	30
6.5	16	22	26	28	36	38
5.6	17	25	29	30	39	41

5.0 ANALYTICAL PROCEDURE

5.1 Sequence

The procedure for evaluating the potential for seismic liquefaction along the alignment involves the following steps:

1) examination of the alignment on a Geotechnical Atlas to identify and catalogue each thawed or thaw susceptible area;

2) evaluation of the geometric characteristics of each area;

3) determination of the geological features and engineering properties of the soils;

4) estimation of the characteristics of the groundwater regime;

5) consideration of the proposed construction mode and construction area grading;

6) consideration of conditions likely to prevail during construction and operation;

7) checking of "non-liquefaction" criteria for each area;

8) identification of significant areas not satisfying "non-liquefaction" criteria;

9) assessment of parameters for use in liquefaction analysis;

10) identification of "low" and "high" liquefaction potential areas by empirical analyses;

11) assessment of parameters for use in cumulative damage analysis;

12) calculation of the factor of safety;

13) application of mitigative measures to achieve a factor of safety of at least 1.0; and

14) refinement of the analyses based on pertinent field observations and updated studies.

The first nine steps of the procedure are essentially a definition of the liquefaction potential parameters. The last five steps involve analytical and empirical methods for analysis. A flow chart describing the logic followed in the analytical procedure is shown as Figure 12.

5.2 Area Catalogue

An area catalogue would be prepared by terrain type for each section of the pipeline alignment. The primary reference sources would be the Geotechnical Atlas, and a listing of proposed design modes. The catalogue would include all terrain type segments along the alignment, that either exist in a presently unfrozen state, or exist in a frozen state, but would be subjected to thaw during construction and operation of the pipeline. The zones would be identified by the kilometre location, K.P., along the pipeline.

Significant data concerning the characteristics of each zone would be included in the catalogue. These would include the topography grade, area length, existing data base, estimated soil profile, estimated groundwater condition, existing degree of freezing, and the proposed design mode. The data base would include a listing of borings drilled within the terrain type. A preliminary assessment of the anticipated liquefaction potential would also be included.

If the criteria for "non-liquefaction" potential are satisfied for an area during the preliminary assessment of the area catalogue, the area would be eliminated from further considerations. Areas which remain would be identified as significant areas, and would require further analyses to evaluate liquefaction potential. These areas would be highlighted on the area catalogue. A sample of the area catalogue is presented in Appendix "C".

5.3 Empirical Analysis

Soil and seismic parameters would be estimated as described in Section 4.0 from available field density, moisture content, and blow count data, supplemented by the results of studies documented by Seed and

Lee (1966), Finn, Pickering and Bransby (1970 and 1971), Donovan and Singh (1978), and Newmark (1980). To account for the change in susceptibility to liquefaction for various soil types, a liquefaction difficulty ratio would be introduced to correct the basic family of curves presented by Donovan and Singh (1978).

5.4 Cumulative Damage Analysis

5.4.1 Analytical Sequence

The assessment of liquefaction potential by the cumulative damage approach would result in the following steps:

1) calculation of the most probable number of cycles of loading from the fundamental period of the site and the total duration of strong shaking;

2) calculation of the imposed peak soil shear stress by multiplying the maximum design surface acceleration by the total mass of soil materials in a unit area column above the depth being investigated;

3) calculation of the distribution of the cyclic shear stress peaks by using statistical procedures;

4) calculation of the cumulative damage by Miner's equation as a summation of the ratios of the number of cycles of exposure to a given stress level to the number of cycles required to produce failure at the same level; and

5) calculation of the factor of safety by iterative evaluation of the cyclic shear stress, to give a cumulative damage of 1.0, and dividing the stress values obtained by the respective earthquake induced stress values.

5.4.2 Results of Analysis

A cumulative damage of at least 1.0 indicates probable liquefaction failure, or the occurrence of large strains caused by significant strength loss. The comparable situation in a laboratory cyclic triaxial test is considered to have been reached when a double-amplitude axial strain of 20 percent is obtained. If cumulative damage is less than 1.0, liquefaction is not likely, and the liquefaction potential is considered to be "low". However, the sharp boundary between "high" and "low" potential defined by a cumulative damage of 1.0 is not warranted in practical seismic and soil mechanics problems. Therefore, an engineering review of all the factors affecting liquefaction would be made to assess further the potential for liquefaction in sensitive areas where the cumulative damage is close to 1.0.

5.4.3 Computer Program

The cumulative damage computer program (CUMLIQ) that would be used in this project is publically available through the United States National Information Service in Earthquake Engineering (NISEE). The program was developed by Dr. N.C. Donovan of the Dames & Moore San Francisco office during his involvement in the Alyeska Project. It was verified initially by its use on the case history data of Seed and Peacock (1970) as shown in Table V, and later by its extensive use on the Alyeska Project. A schematic representation of the liquefaction evaluation procedure is shown on Figure 13.

5.5 Liquefaction Potential

Following this engineering review, a final evaluation would be made in terms of "high" or "low" liquefaction potential for each area being studied. When "low" liquefaction potential is indicated by the analysis, the design would be considered safe with respect to seismic liquefaction.

TABLE V

VERIFICATION OF THE CUMULATIVE DAMAGE PROGRAM

CASES			LIQUEFACTION EVALUATION PARAMETERS*				RESULTS	
NO.	IDENTIFICATION		RELATIVE DENSITY	ACCEL. g	FUND. PERIOD, SEC. T_o	DURATION SEC.	ANALYTICAL	REPORTED
1	NIIGATA	(1806. M = 6.6)	0.53	0.12	0.80	20	NO	NO
2	NIIGATA	(1806. M = 6.6)	0.64	0.12	0.80	20	NO	NO
3	NIIGATA	(1887. M = 6.1)	0.53	0.12	0.80	12	NO	NO
4	NIIGATA	(1887. M = 6.1)	0.64	0.08	0.80	12	NO	NO
5	MINO OWARI - OGAKI	(1891. M = 8.4)	0.65	0.35	1.00	75	YES	YES
6	MINO OWARI - GINAN WEST	(1891. M = 8.4)	0.55	0.35	1.00	75	YES	YES
7	MINO OWARI - UNUMA	(1891. M = 8.4)	0.75	0.35	1.00	75	YES	YES
8	MINO OWARI - OGASE POND	(1891. M = 8.4)	0.72	0.35	1.00	75	YES	YES
9	EL CENTRO - BRAWLEY	(1940. M = 7.0)	0.50	0.275	0.50	30	YES	YES
10	EL CENTRO - ALL-AM	(1940. M = 7.0)	0.43	0.25	0.50	30	YES	YES
11	EL CENTRO - SOLFATARA C.	(1940. M = 7.0)	0.32	0.25	0.50	30	YES	YES
12	TOHNANKAI - KOMEI	(1944. M = 8.3)	0.40	0.08	1.00	70	NO	YES**
13	TOHNANKAI - MEIKO ST.	(1944. M = 8.3)	0.30	0.08	0.50	70	YES	YES
14	FUKUI - TAKAYA	(1948. M = 7.2)	0.72	0.30	0.60	30	NO	NO
15	FUKUI - TAKAYA	(1948. M = 7.2)	0.90***	0.30	1.00	30	NO	NO
16	FUKUI - SHONENJI TEMPLE	(1948. M = 7.2)	0.40	0.30	1.00	30	YES	YES
17	FUKUI - AGR. UNION	(1948. M = 7.2)	0.50	0.30	1.00	30	YES	YES
18	SAN FRANCISCO - LAKE MERCED	(1957. M = 5.5)	0.53	0.18	0.50	18	NO	YES**
19	CHILE - PUERTO MONTT	(1960. M = 8.4)	0.50	0.15	1.00	75	NO	YES**
20	CHILE - PUERTO MONTT	(1960. M = 8.4)	0.55	0.15	1.00	75	NO	YES**
21	CHILE - PUERTO MONTT	(1960. M = 8.4)	0.75	0.15	1.00	75	NO	NO
22	NIIGATA	(1964. M = 7.5)	0.53	0.16	0.80	40	YES	YES
23	NIIGATA	(1964. M = 7.5)	0.70	0.16	0.80	40	NO	YES**
24	NIIGATA	(1964. M = 7.5)	0.64	0.16	0.80	40	NO	NO
25	NIIGATA	(1964. M = 7.5)	0.53	0.16	0.80	40	NO	NO
26	ALASKA	(1964. M = 8.3)	0.50	0.15	2.00	180	YES	YES
27	ALASKA - SNOW RIVER	(1964. M = 8.3)	0.44	0.15	2.00	180	YES	YES
28	ALASKA - QUARTZ CREEK	(1964. M = 8.3)	1.00***	0.12	2.00	180	NO	NO
29	ALASKA - SCOTT GLACIER	(1964. M = 8.3)	0.65	0.16	2.00	180	YES	YES
30	ALASKA - VALDEZ	(1964. M = 8.3)	0.68	0.25	2.00	180	YES	YES
31	TOKACHIOKI - HACHINOHE	(1968. M = 7.8)	0.78	0.21	1.00	45	NO	NO
32	TOKACHIOKI - HACHINOHE	(1968. M = 7.8)	0.58	0.21	1.00	45	YES	YES
33	TOKACHIOKI - HACHINOHE	(1968. M = 7.8)	0.80	0.21	1.00	45	NO	NO
34	TOKACHIOKI - HAKODATE	(1968. M = 7.8)	0.55	0.18	1.00	45	YES	YES
35	SANTA BARBARA-SHEFFIED DAM	(1925. M = 6.3)	0.40	0.20	0.50	15	YES	YES
36	CARACAS-CARA BALLEDA	(1967. M = 6.3)	0.60	0.13	0.50	15	NO	YES**
37	SAN FERNANDO - JUVENILE HALL	(1971. M = 6.6)	0.30	0.40	0.40	15	YES	YES
38	SAN FERNANDO-JENSEN PLANT	(1971. M = 6.6)	0.58	0.35	0.40	15	YES	YES
39	CHILE-HUACHIPATO	(1960. M = 8.4)	1.00***	0.25	0.50	75	NO	NO
40	CHILE-HUACHIPATO	(1960. M = 8.4)	1.00***	0.25	0.50	75	NO	NO
41	KERN COUNTY-KERN STATION	(1952. M = 7.7)	0.58	0.25	0.25	30	YES	NO**

* ESTIMATED VALUE

** ANALYTICAL RESULT DIFFERS FROM REPORTED RESULT

*** ANALYSES PERFORMED ALTHOUGH POTENTIAL FOR LIQUEFACTION CONSIDERED UNLIKELY

6.0 MITIGATIVE MEASURES

6.1 Types

Several measures could be taken to prevent, minimize, accommodate, or avoid potential liquefaction problems. The measures, or combination of measures, could differ substantially from one area of potential liquefaction to another. The types of mitigation considered would depend on factors, such as topography, thaw condition, soil profile, proposed design mode, and groundwater condition of both the affected terrain and adjoining terrains.

The remedial measures available can be categorized into two types:

1) conventional methods, such as:
   a. rerouting,
   b. weighting, and
   c. deeper burial,

2) special methods, such as:
   a. drainage,
   b. grouting,

c. compaction,

d. soil replacement,

e. slope flattening,

f. retaining structures, and

g. freezing.

Conventional methods incorporate the remedy into the design of the pipeline system, with limited impact on the environment. Special methods involve changes to the nature of the problematic soils, with greater impact on the environment.

The final choice of mitigative measure would depend on the economics involved, an acceptable degree of risk, and the impact on the environment. A decision analysis framework applicable to liquefaction susceptible areas has been developed by Haldar (1980). It attempts to select the best solution by consideration of technical and economic aspects and considers the fact that liquefaction does not always lead to unacceptable consequences.

6.2 Conventional Method

6.2.1 Rerouting

Boundary conditions for flat areas would generally confine liquefiable soils. However, for sloped areas, or flat areas that are not confined, a review of adjoining terrains would be made to assess the confinement capabilities and liquefaction potential. If these terrains are not susceptible, or less susceptable to liquefaction, the pipeline could be rerouted to bypass the problem areas. A major consideration in this decision would be the additional length of pipe involved.

6.2.2 Weighting

In flat terrains, sections of the pipeline could be weighted to neutralize bouyancy and eliminate or reduce the tendency for the pipe to float in the liquefied soil. Weighting could not be considered in sloped areas, because the mass movement of soil in the downslope direction would dictate the failure mechanism.

6.2.3 Deeper Burial

In areas where liquefaction could occur in a veneer or blanket of soil which overlies a more competent stratum, the pipe could be buried below the liquefiable layer. Liquefaction and movement of the upper layer would not affect the integrity of the pipe. This approach would be feasible only for shallow liquefiable soils of depths no more than 3 or 5 metres. Beyond these depths, special methods of stabilization would probably be required.

6.3 Special Methods

6.3.1 Drainage

Because saturation and a high groundwater table are requirements for liquefaction to occur, drainage may be installed to stabilize wet areas. Drains would conduct water away from problem areas, and would lower the groundwater table to a point where liquefaction would no longer be a problem. Permanent drainage systems would be required for continuous dewatering of the affected soils. Regional topographic considerations would be an important criteria for this method of stabilization. Freezing from permafrost and partial thaw conditions must be considered. In permafrost areas where a thaw bulb develops, drainage may be impractical.

6.3.2 Grouting

If a relatively small increase in cohesive strength and a decrease in void ratio would sufficiently reduce the liquefaction potential, the feasibility of grouting the soil may be evaluated. The extent of the area requiring treatment must necessarily be reasonably limited. This method could be applied in parallel with efforts to confine the liquefaction susceptible soil layer of concern.

6.3.3 Compaction

If a relatively small increase in density would sufficiently reduce the liquefaction potential, the feasibility of compacting the soil may be evaluated. This may be achieved by blasting or vibro flotation methods. The extent of the area requiring treatment could be large and the impact on the environment would be significant. Careful controls would be required during these operations.

6.3.4 Soil Replacement

Where a large increase in density is necessary, and both areal extent and depth of the region in question are sufficiently limited, the method of soil replacement may be used. The loose soil would be excavated to the depth required. If moist and unfrozen, it would be placed back in lifts compacted to specified densities. If saturated or frozen, it would be wasted and replaced with imported fill, placed and compacted to specified densities.

The method could be used where the layer thickness of liquefiable soil is relatively small and is located at shallow depths. A narrow, buried berm of dense soil would be built to resist the lateral movements and forces imposed by the adjacent liquefiable soils. The length, for practical reasons, must be reasonably limited. This procedure would be the most practical in the case of localized sections of thawed ground.

6.3.5 Retaining Structures

Large soil movements could be prevented by confining the liquefied soil with reasonably limited retaining structures, such as sheet piling or retaining walls. The impact on the environment would be minimal, as only small areas could be realistically stabilized by this method.

6.3.6 Slope Flattening

Sloped areas which are susceptible to liquefaction could be flattened to less than two percent to minimize the effect of soil movement on the pipeline. The design alternatives for flat terrains would then be applicable. The environmental impact of performing slope flattening could be significant.

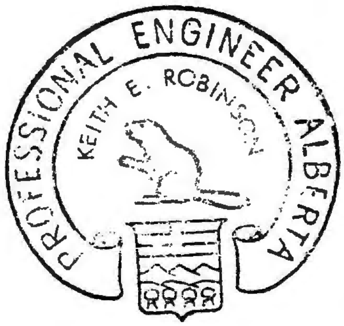
6.3.7 Contingency Provisions

If special design measures prove inadequate for the Design Contingency Earthquake, the soil liquefaction potential could be evaluated for the Design Operating Earthquake. If liquefaction potential is "low" under these conditions, then contingency provisions could be instituted under special conditions and stipulations. This criterion would be used only if all other possible solutions to the problem prove to be not feasible.

6.4 Construction Inspection

Continuous observation of all earthwork operations should be performed by experienced geotechnical and arctic engineering crews to verify or modify designs and assist in the implementation of restoration efforts. Observations would be made and documented, of soil characteristics, discontinuities, saturation, ice presence, and thermal conditions.

Respectfully Submitted,
ROBINSON DAMES & MOORE



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APPENDIX "A"

HARDY ASSOCIATES (1978) LTD.
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A1. DEFINITION OF SYMBOLS USED IN SLOPE CATALOGUE INFORMATION SHEET

BACKGROUND INFORMATION																		PRELIMINARY GEOTECHNICAL ASSESSMENT	
SLOPE NO.	A.S. NO.	K.P.	SLOPE CONDITIONS							EXISTING DATA BASE			F OR U	TERRAIN TYPE	SUBSURFACE STRATIGRAPHY	GROUNDWATER CONDITIONS	PROPOSED DESIGN MODE	DESIGN PROBLEMS	REQUIRED ANALYSIS
			ALONG ℄			CROSS ℄			ASPECT AZIM.										
			AVE (MAX) %	LENGTH (m)	HT (m)	AVE (MAX) %	LENGTH (m)	HT (m)		BOREHOLES	TESTS	INSTR.							
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

1. SLOPE NUMBERING SEQUENCE (V-I: section – slope number)
2. ALIGNMENT SHEET WHERE THE SLOPE IS LOCATED
3. KILOMETER POST AT START OF SLOPE OR PIPE AFFECTED
4. AVERAGE (MAXIMUM) SLOPES, IN PERCENT ALONG CENTRELINE OF PIPE. SLOPE IS POSITIVE WHEN SLOPE HEIGHT IS INCREASING CONCOMMITANTLY WITH CHAINAGE.
5. LENGTH OF SLOPE IN METRES MEASURED HORIZONTALLY
6. HEIGHT OF SLOPE IN METRES
7. AVERAGE (MAXIMUM) SLOPES, IN PERCENT, ACROSS CENTRELINE OF PIPE. SLOPE IS POSITIVE WHEN, LOOKING IN DIRECTION OF INCREASING CHAINAGE, THE SLOPE IS UPHILL TO THE RIGHT.
8. LENGTH OF PIPE THAT MAY BE AFFECTED BY THE SLOPE, IN METRES.
9. HEIGHT OF SLOPE IN METRES
10. AZIMUTH OF THE DOWNHILL DIRECTION OF THE SLOPE, IN DEGREES.
11. DESIGNATION OF AVAILABLE BOREHOLES PERTINENT TO THE SLOPE.
12. SUMMARY OF SPECIALIZED LABORATORY TESTING CONDUCTED ON SAMPLES OBTAINED AT THE BOREHOLE LOCATIONS.

    TS - THAW STRAIN   DS - DIRECT SHEAR   Qu - UNCONFINED COMPRESSION
    TXCU - TRIAXIAL CONSOLIDATED-UNDRAINED   TXUU - TRIAXIAL UNCONSOLIDATED-UNDRAINED
13. INSTRUMENTATION PLACED AT THE BOREHOLE LOCATIONS.

    T - THERMISTOR   S - STANDPIPE PIEZOMETER   PC - CASAGRANDE PIEZOMETER
    P - PNEUMATIC PIEZOMETER   EMR - ENERGY MINES RESOURCES THERMISTOR
    N - NO INSTRUMENTATION
14. SOIL CONDITION : FROZEN OR UNFROZEN
15. TERRAIN TYPE BASED ON AIRPHOTO INTERPRETATION. FOR DETAILS REFER TO GEOTECHNICAL ATLAS.
16. GENERALIZED SUMMARY OF APPLICABLE BOREHOLE LOGS.
17. GROUNDWATER CONDITIONS:

    (M) - MEASURED FROM PIEZOMETERS   (E) - ESTIMATED
    (A) - ESTIMATED AVERAGE BASED ON PIEZOMETER READINGS
18. PIPE DESIGN MODE
19. IDENTIFICATION OF POTENTIAL FAILURE MECHANISMS, PRIMARY AND SECONDARY (IF APPLICABLE), AND RECOGNITION OF THE ADVERSE EFFECTS ON THE PIPE.
20. METHOD OF ANALYSIS EMPLOYED TO MODEL THE IDENTIFIED POTENTIAL FAILURE MECHANISM.

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DEFINITION OF SYMBOLS USED IN SLOPE CATALOGUE INFORMATION SHEETS

No. K 5501A

A2. DESIGNATION OF PROPOSED PIPE DESIGN CONSTRUCTION MODES

1: Standard Burial

2: Saddle-weighted burial

3: Continuous weighted burial

4: Unweighted burial

5: Deep burial

6: Insulated gravel embankment

7: Concrete restrained above grade

8: Transition: burial to above grade

9: Special design for road crossings

A3. SLOPE CATALOGUE - SAMPLE

			BACKGROUND INFORMATION															PRELIMINARY GEOTECHNICAL ASSESMENT	
SLOPE NO	A.S. NO.	K.P.	SLOPE CONDITIONS							EXIST. DATA BASE			F or U	TERRAIN TYPE	SUBSURFACE STRATIGRAPHY	GROUNDWATER CONDITIONS	PROP. DESIGN MODE	DESIGN PROBLEMS	REQUIRED ANALYSIS
			ALONG ℄			CROSS ℄			ASP. AZIM.										
			AVE.(max) %	L (m)	HT (m)	AVE. (max) %	L (m)	HT (m)		BOREHOLES	TESTS	INSTR.							
III-13	19	128.38				-9(15)	200	9	218	80-11-091		T#12	F	$\frac{f.vE_{M.B}}{sE_{R.B}}$	0.0-6.0 Peat, organic silt and volcanic ash 6.0-6.2 Silt 6.2-7.3 Sand 7.3-10.0Clay		7		
										80-01-43		N	F		3 m organic cover, slightly coarser grained than above				
III-14	19	128.68				-15	150	3	217	80-11-091		T#12	F	$\frac{f.vE_{M.B}}{sE_{R.B}}$	0.0-6.0 Peat, organic silt and volcanic ash 6.0-6.2 Silt 6.2-7.3 Sand 7.3-10.0Clay		7		
										80-01-43		N	F		3 m organic cover, slightly coarser grained than above				
III-15	19	128.99	-19(30)	125	24				126	80-11-091		T#12	F	$\frac{f.vE_{M.B}}{sE_{R.B}}$	0.0-6.0 Peat, organic silt and volcanic ash 6.0-6.2 Silt 6.2-7.3 Sand 7.3-10.0Clay		7		
										80-01-43		N	F		3 m organic cover, slightly coarser grained than above				
III-16	19	129.51	-20	15	3				123	76-0-2 81-01-109		T	F	$\frac{f.vE_{M.B}}{sE_{R.B}}$	0.0-2.7 Peat & organic silt 2.7-4.1 Ice with soil inclusions 4.1-8.5 Silt 8.5-9.1 Bedrock 80-11-091: 6m Peat		7		
III-17	19	129.68	-24(25)	75	18				86	76-0-1		T	F	$\frac{f.vE_{M.B}}{sE_{R.B}}$	0.0-(4.1to5.0)Peat, organic silt (ice layer) (4.1to5.0)-(5.8to8.5)Silt (5.8to8.5)-9.1 Gravel, Sand/Bedrock		9(5)		
										76-0-2		T	F						
										78-DJ-1 78-DJ-2	TS DS, TS	T N	F F/U 10.8	$\frac{fC_B}{ED_{(E)}}$	0.0-0.5 Peat, organic silt, 5.0-6.5 Silt with peat inclusions and layers 6.5-(13.3to15.5)Clay, Sand and and gravel (13.3 Bedrock @ 78-DJ-1)		9(5)		

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SLOPE CATALOGUE INFORMATION SHEET

K 5501 A

APPENDIX "B"

HARDY ASSOCIATES (1978) LTD.
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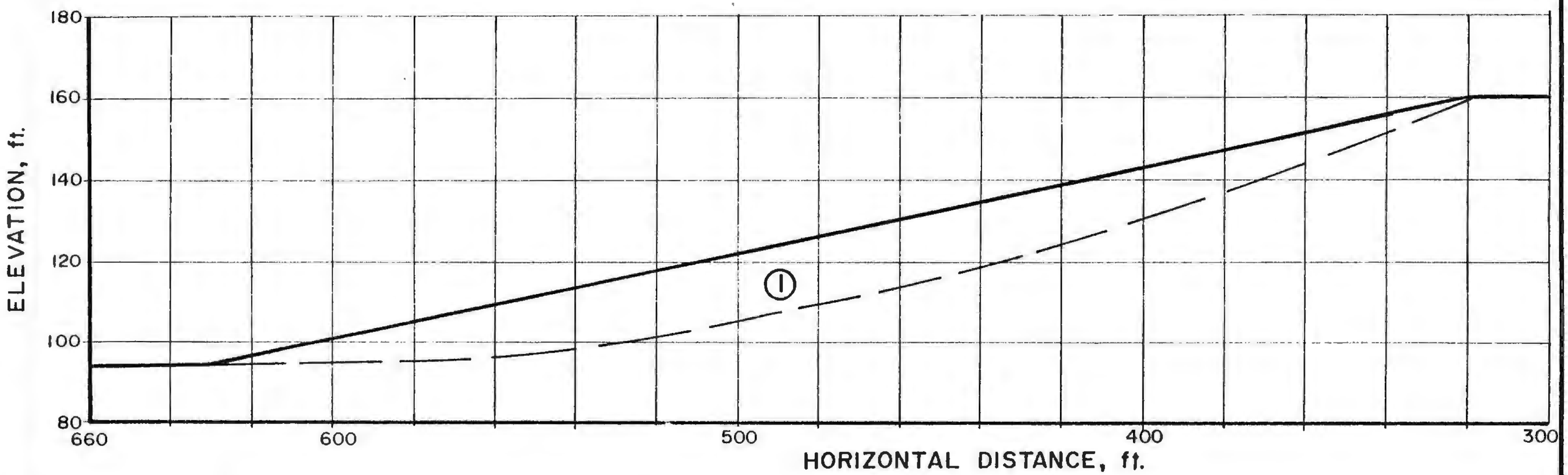
CROSS SECTION OF SLOPES

UTILIZED FOR VERIFICATION OF

JANBU2 COMPUTER PROGRAM



NOTE: No. 1 γ_T = 125 pcf ϕ' = 33°

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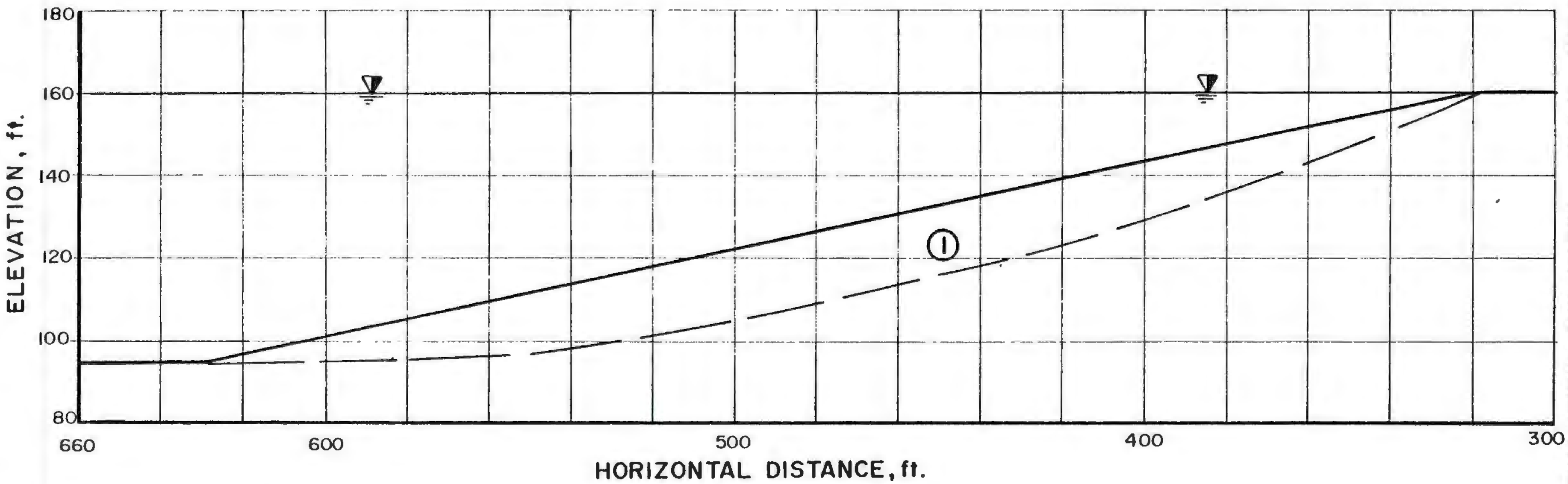
SLOPE CROSS SECTION

K5501E

Fig B1



NOTE No. 1 γ_T =125 pcf ϕ'= 33°

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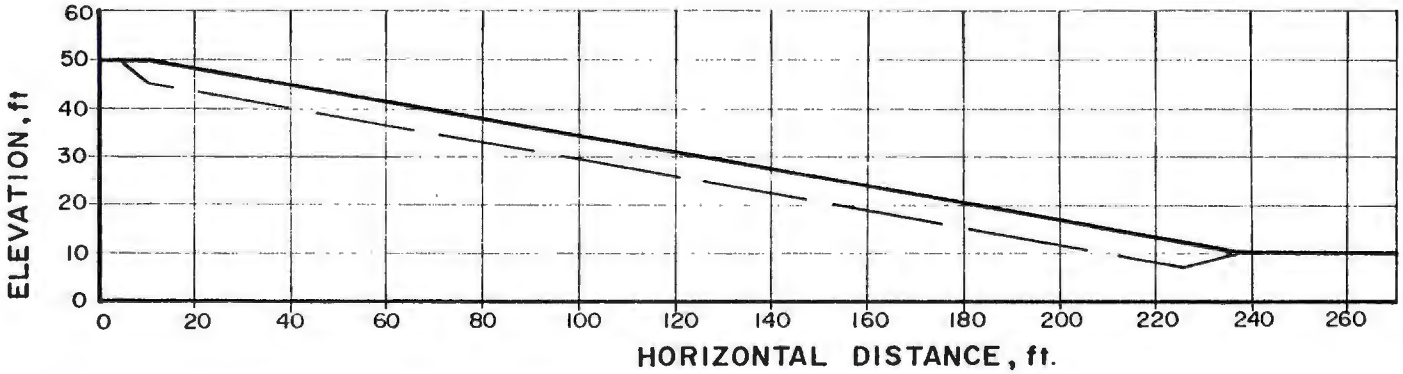
SLOPE CROSS SECTION

K5501 E

Fig. B2



NOTE: For material properties and groundwater conditions, refer to table.

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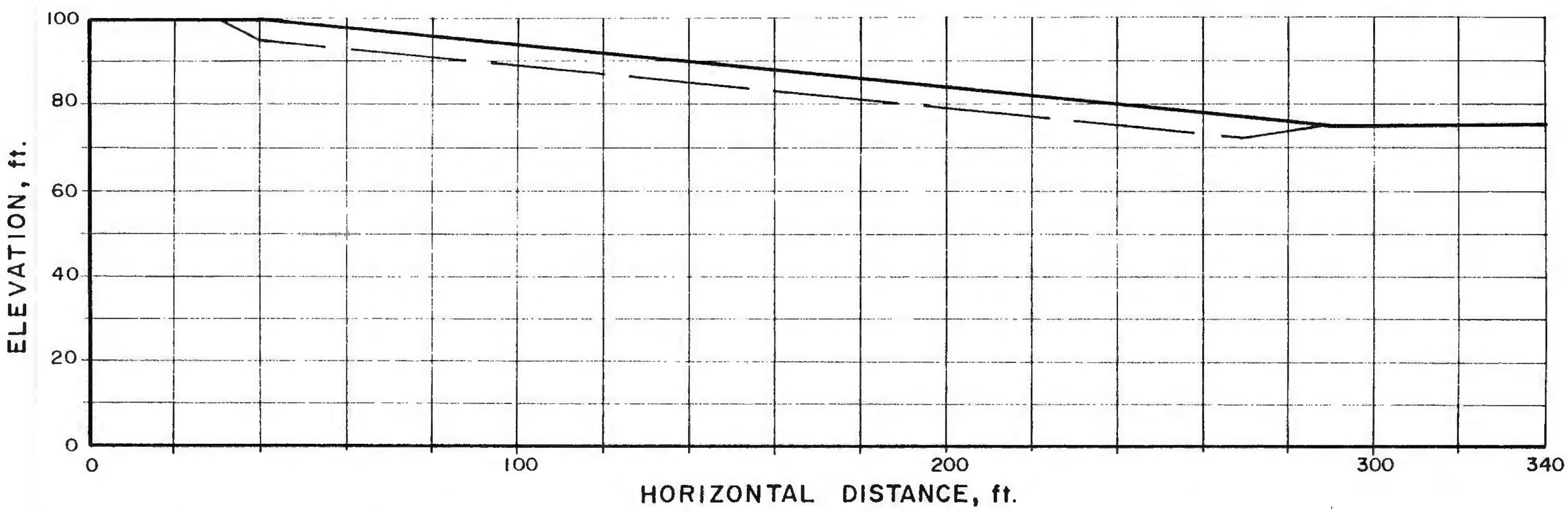
SLOPE CROSS SECTION

K 5501E

Fig. B3



NOTE For material properties and groundwater conditions refer to table.

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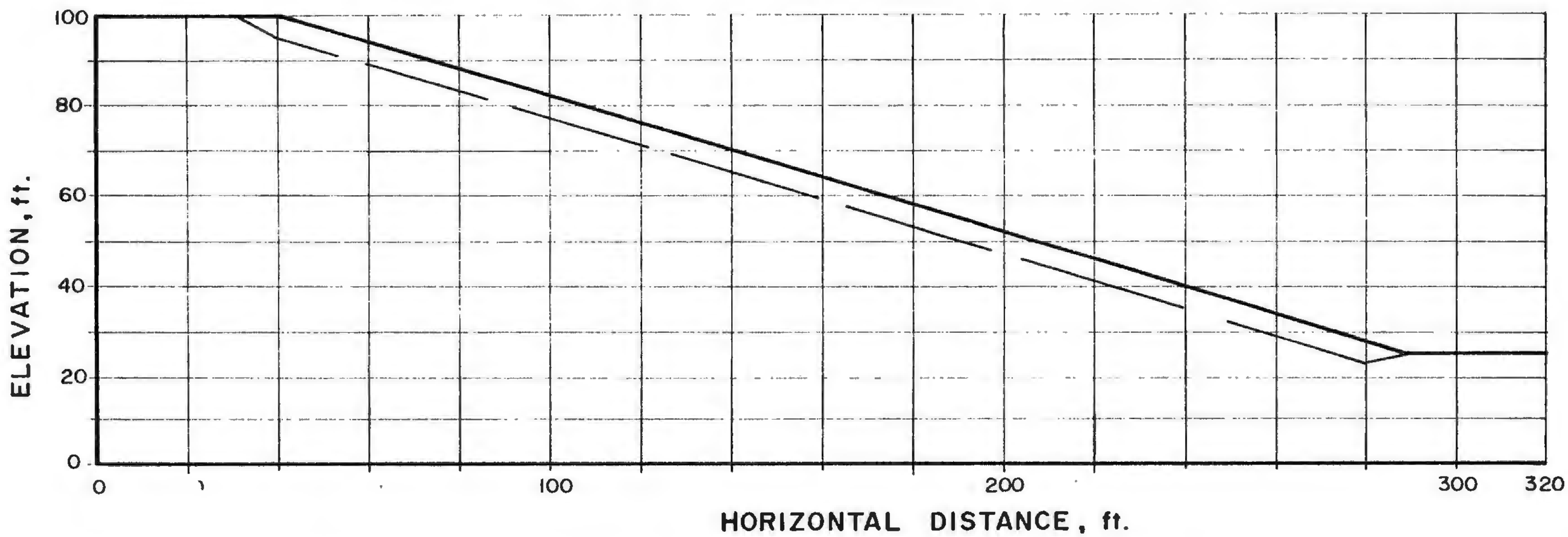
SLOPE CROSS SECTION

K5501 E

Fig B4



NOTE: For material properties and groundwater conditions, refer to table.

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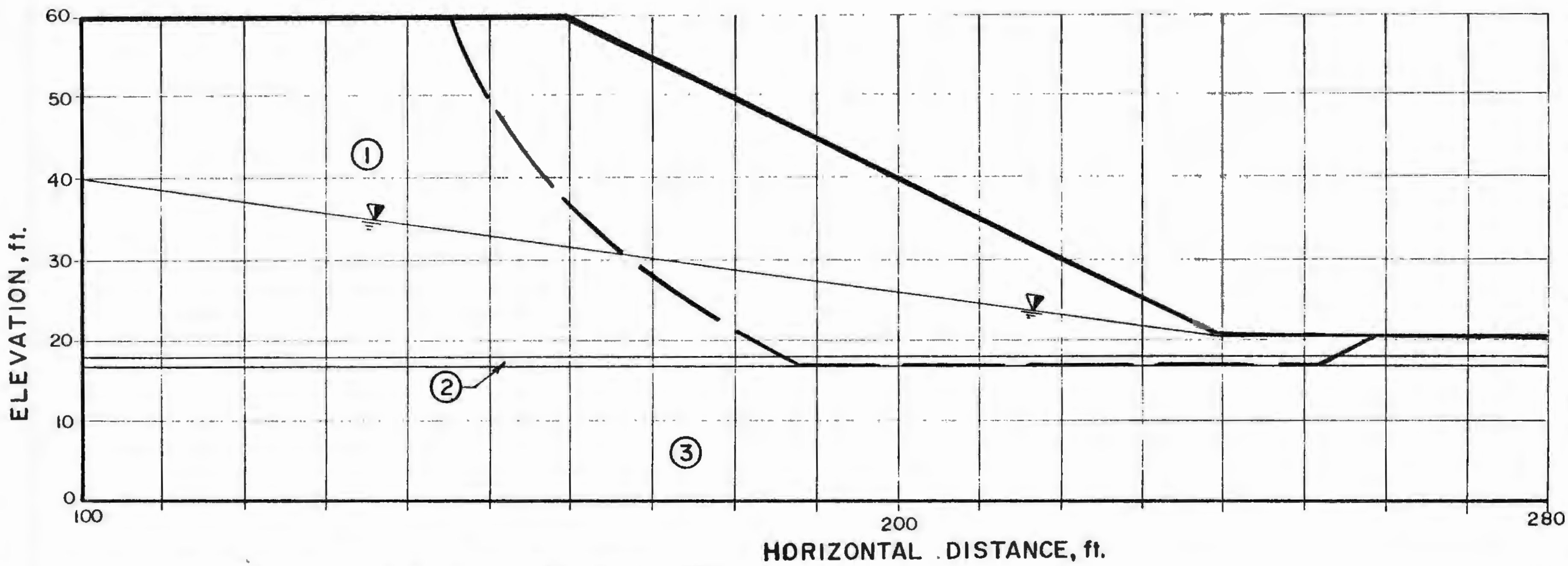
SLOPE CROSS SECTION

K5501E

Fig. B5



NOTE From Canadian Geotechnical Journal 1977, Vol. 14, p.436

No.1 γ_T = 120 pcf, ϕ = 20°, c = 600 psf

No.2 γ_T = 100 pcf, ϕ = 10°, c = 0

No.3 Bedrock

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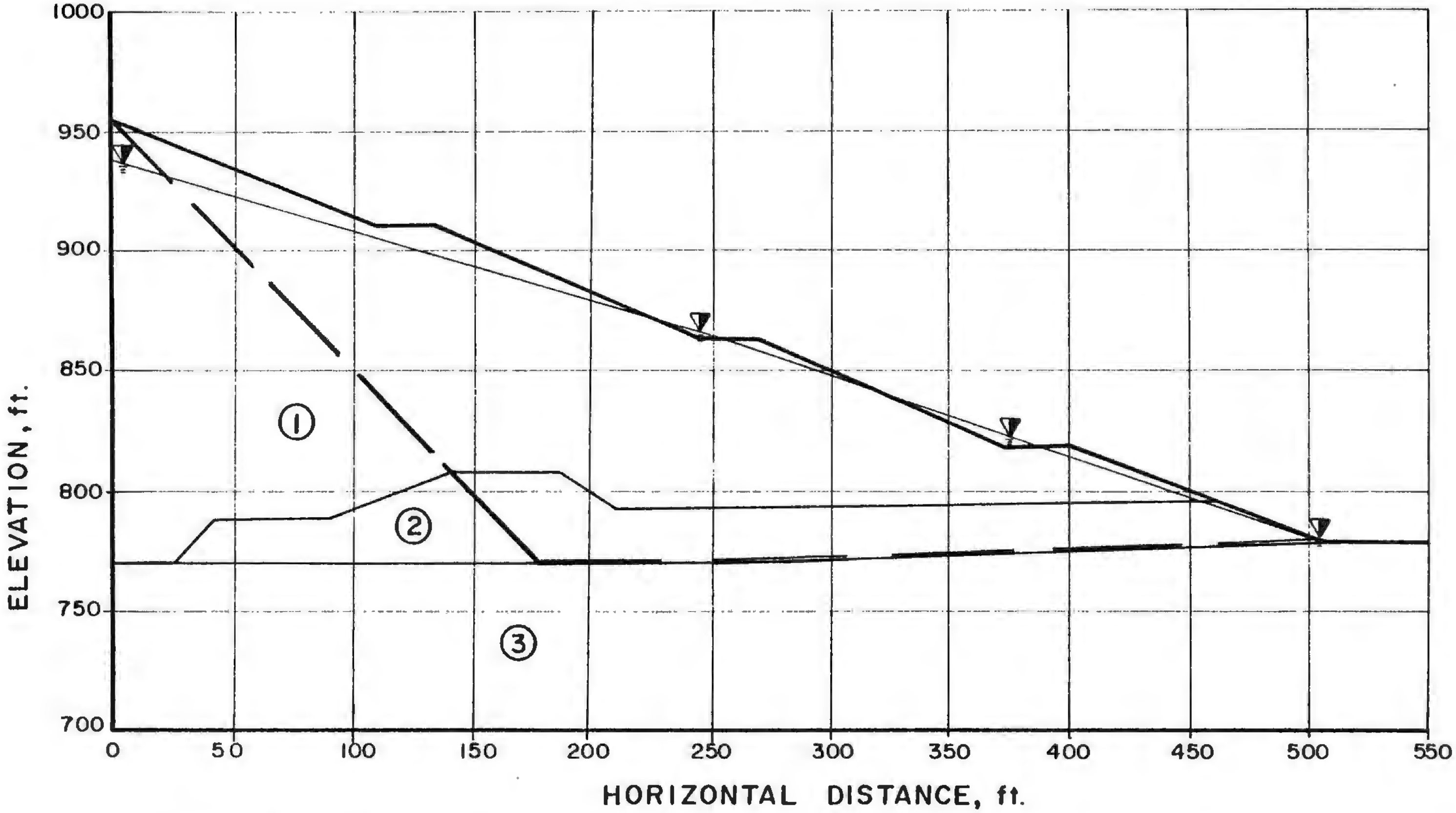
SLOPE CROSS SECTION

K 5501E

Fig. B6



NOTE No. 1 γ_T = 120, ϕ = 34°
No. 2 γ_T = 120, ϕ = 26°
No. 3 γ_T = 120, ϕ = 26.5°

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SLOPE CROSS SECTION

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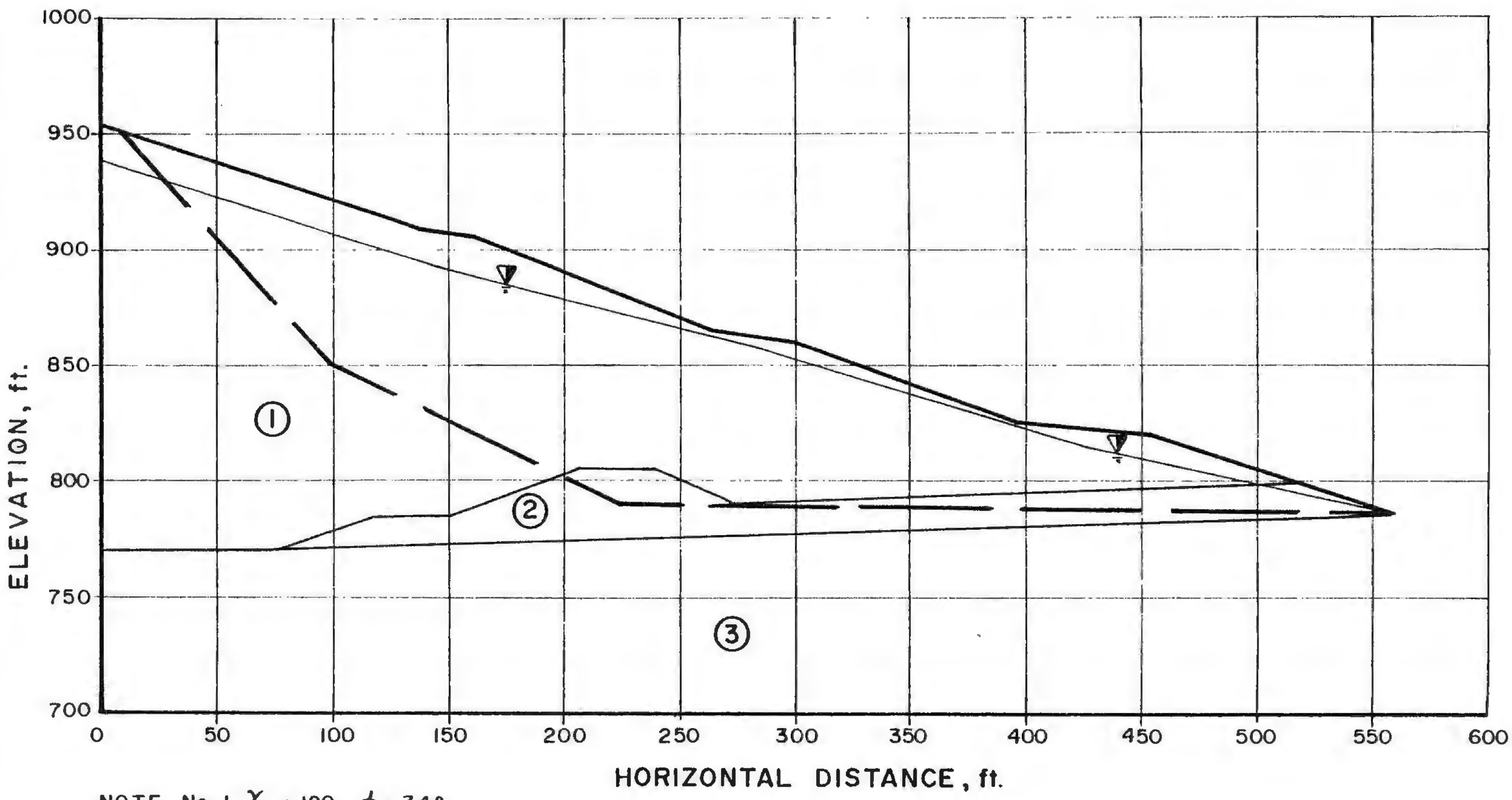
Fig. B7



NOTE No. 1 $\gamma_T = 120$, $\phi = 34°$
No. 2 $\gamma_T = 120$, $\phi = 26°$
No. 3 $\gamma_T = 120$, $\phi = 26.5°$

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SLOPE CROSS SECTION

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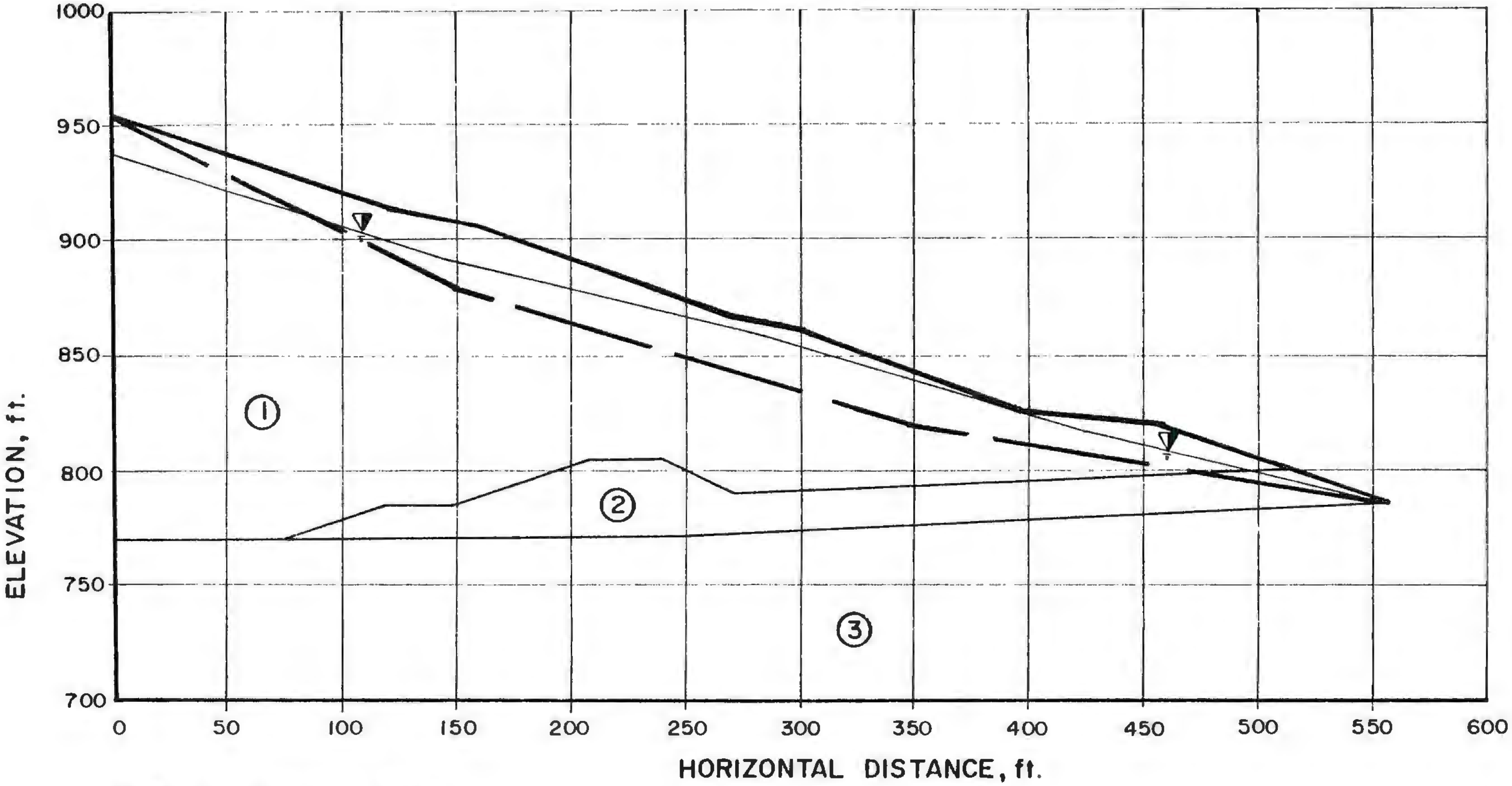
Fig. B8



NOTE No. 1 $\gamma_T = 120$, $\phi = 34°$

No. 2 $\gamma_T = 120$, $\phi = 26°$

No. 3 $\gamma_T = 120$, $\phi = 26.5°$

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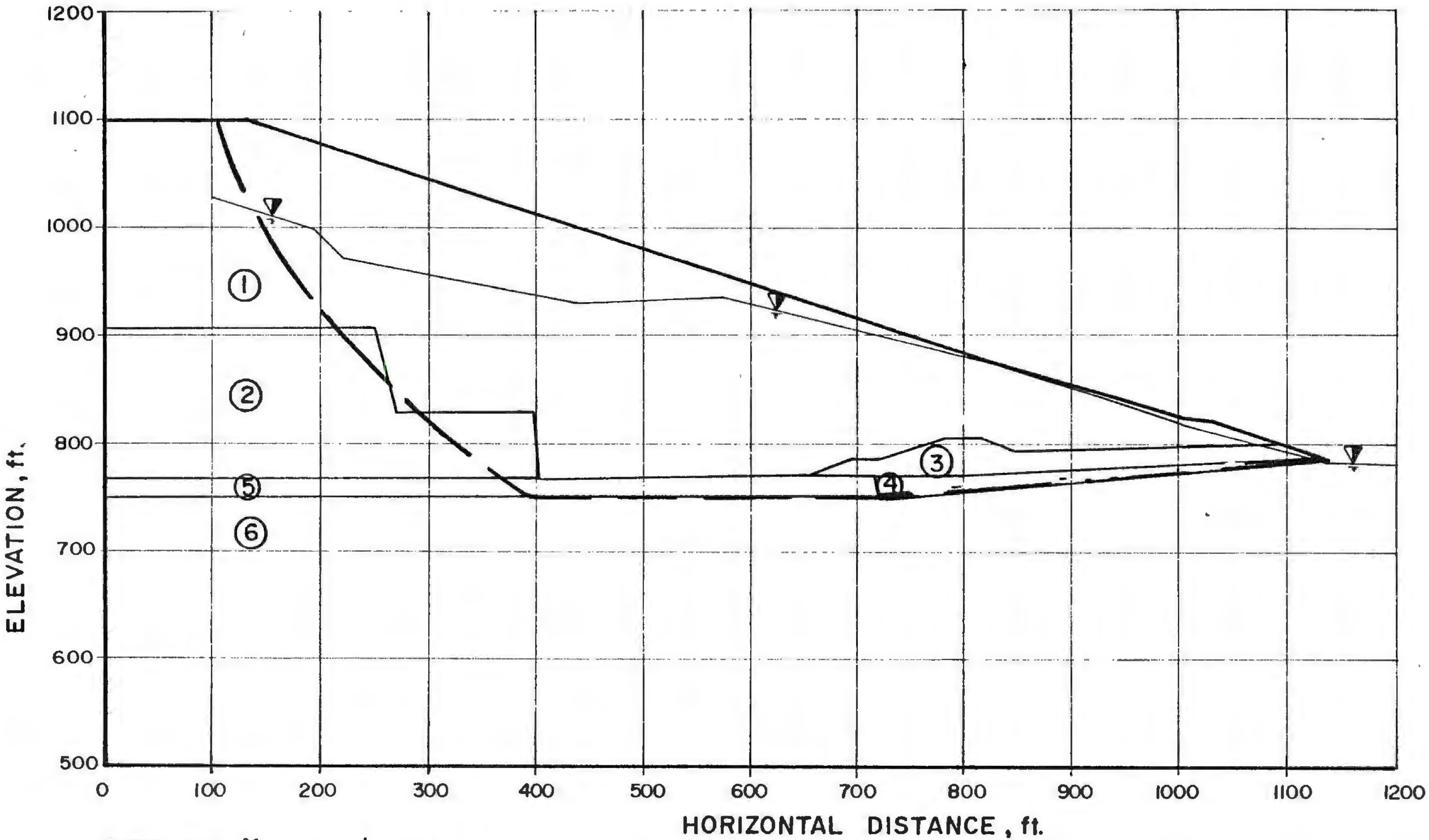
SLOPE CROSS SECTION

K 5501 E

Fig. B 9



NOTE No. 1 γ_T = 120, ϕ = 34°, c = 0
No. 2 γ_T = 120, ϕ = 30°, c = 0
No. 3 γ_T = 120, ϕ = 26°, c = 0
No. 4 γ_T = 120, ϕ = 24°, c = 0
No. 5 γ_T = 120, ϕ = 26.5°, c = 0
No. 6 Bedrock

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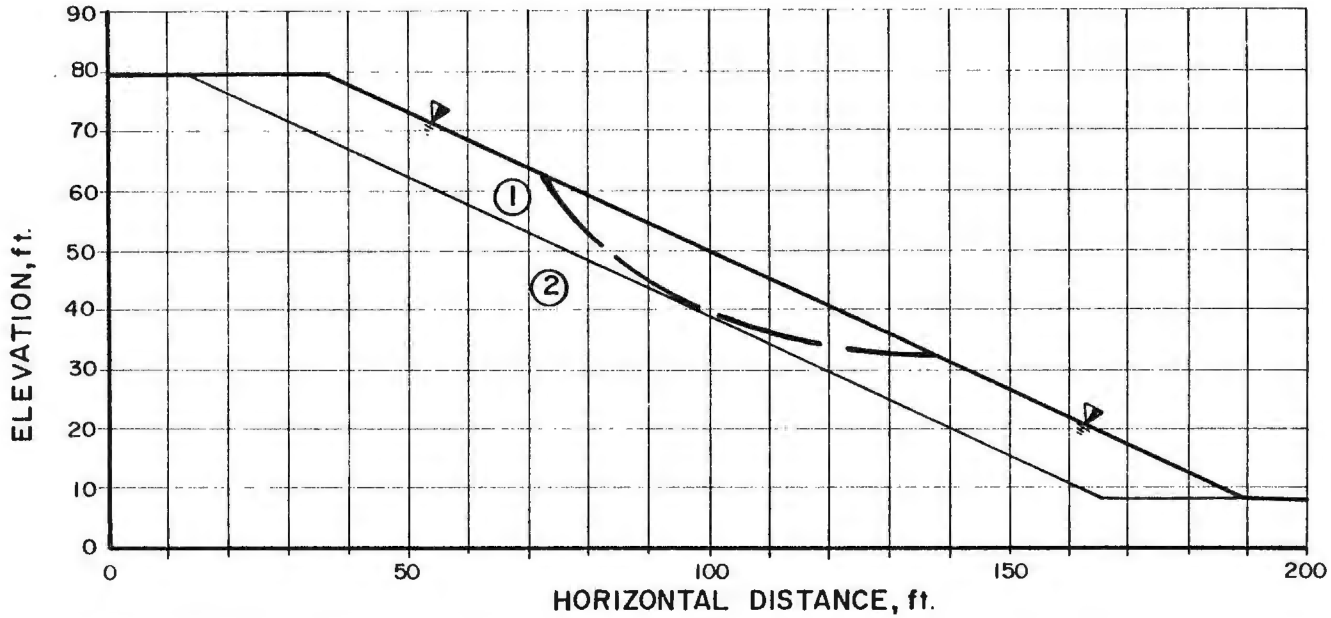
SLOPE CROSS SECTION

K 5501 E

Fig. B10



NOTE No.1 γ_T = 110 pcf, ϕ = 11°, c = 250 psf
No.2 Frozen soil.
From Dames and Moore (Alyeska)

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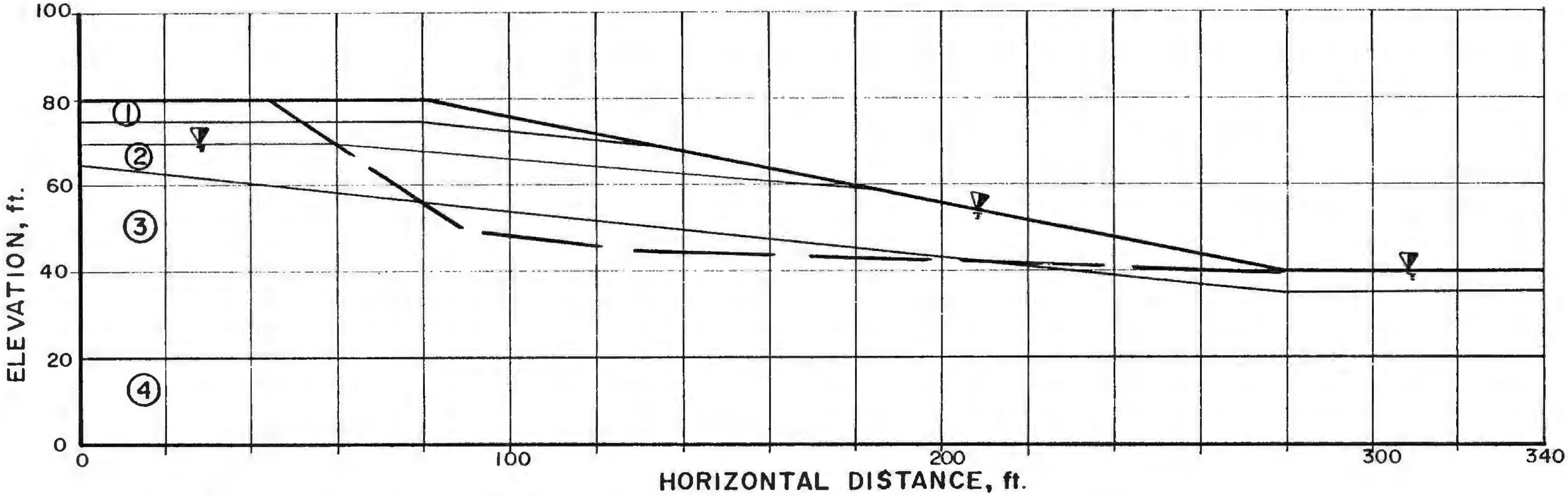
SLOPE CROSS SECTION

K5501 E

Fig. B 11



NOTE No. 1 γ_T = 100 pcf, ϕ = 11°, c = 250 psf
No. 2 γ_T = 125 pcf, ϕ = 20°, c = 700 psf
No. 3 γ_T = 127 pcf, ϕ = 35°, c = 0
No. 4 γ_T = 137 pcf, ϕ = 37°, c = 0

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SLOPE CROSS SECTION

K 5501 E

Fig. B12

APPENDIX "C"

AREA CATALOGUE - SAMPLE

AREA CATALOGUE - SECTION 3

Preliminary Assessment of Seismic Liquefaction (Cont'd)

K.P. at Start of Zone	Zone Length (m)	Existing Frozen State	Terrain Type	Available Data Base	Design Mode	Critical Strata (m)	Fine Granular Soil	High Satur- ation	Plastic Index 5	Requires Further Analysis	Percent Slope	Water Depth (m)	Possible Mitigative Measures
147.02	80	1	fvEV/aGH	Atlas	1	1-5.5	x		x				
147.10	170	1	fvEBV/ msGPB	Atlas	1	1.2-3.8	x		x				
147.27	230	1	pfOVB/ fABP	80-11-099	1	1.4-4.5	x	x	x	x	2-4	3.0*	Deep burial embankment
				79S-C-5		3.0-3.8	x	x					
147.50	730	1	fpAB/tMM	80-11-099(2)	1	4.8-7.9	x		x				
148.23	180	1	pfOVB/ fABP	81-01-119	1	1.8-3.0	x		x				
				78-A-46		6-9	x						
148.41	1930	1	fvEBV/tMM	81-01-120	1	1.3-5.8	x		x				Deep burial
				78-B-28		7-8.2	x		x				
				81-01-121		.5-1.8	x		x				
				78-A-47		1.7-7.4	x						
				80-01-50		2.4-8.4	x		x				
150.34	440	1	fpAB/tMP	80-11-100	1	1.7-4.5	x		x		(14)		
150.78	150	1	fAB/aGP	Atlas	1								
150.93	870	1	fpAB/aAFB	79-B-21	1/2	3.5-10.1	x						
				81-01-123		1.8-3.1	x		x				
				80-01-51		3.8-9.9	x						
151.80	300	1	fAP/atDP	81-01-124	1	2.8-4.4	x	x	x				Not required
152.10	5290	1	fvEBV/tMM	80-01-52	1	2.8-5.4	x		x		(12)		Deep burial
				81-01-125		2.1-3.7	x		x				
				80-01-53		3.6-9.7	x		x				
				80-01-54		-							
				80-01-126		-							
157.39	60	1	dCB/D(E)	Atlas	1/3						26		
157.45	370	1	fpAB/gAF	78-B-29	1	1-2	x	x					
157.82	2520	1	fEB/tMM	78-A-50	1	6-8.2	x		x		(3)24		
				80-11-101(3)									
				80-11-101									
				81-01-127									
				80-01-55									
				81-01-128									
				80-11-101(4)									
				78-GS-44									
				81-01-129									
				81-01-130									
160.34	190	2	f/aAP(A) + f/aAT	79-A-15	2	.5-3.8	x					3.5	
				81-01-131		1.0-2.0	x		x				
				78-A-52		1.1-3.2	x						
160.53	1850	1	fEB/tMM	78-A-51	1/2	3.8-4.6	x				23		
				80-01-56(1)		.9-5.2	x						
				80-01-56(2)		1.8-4.7	x	x					
				80-11-101(2)		2-4.5	x						
				80-11-GS-140									
				80-01-57		3.4-8.7	x						
162.38	860	1	fAB/taDP	80-11-101(5)	1								Deep burial
				78-B-31		2.5-5.5			x				
				81-01-132		1.7-10.0	x						
163.24	90	1	dCB/d(E)	Atlas	3	0-3.5		Do not require			30		
163.33	1670	1	fEB/ctMM	80-11-102	3			preliminary analysis			(24)11		
				80-11-103		2.4-5.5		for the embankment					
				78-B-32				mode of construction.					
				81-01-133		2.9-6.6							
				78-A-53									

NOTES: 1. Existing Frozen State - 1 for presently frozen soils
2 for presently non-frozen soils
1/2 frozen state uncertain

2. Design Mode - 1 for Unweighted Burial
2 for Weighted Burial
3 for Above Grade (concrete restrained)

3. Water Depth marked with an asterisk (*) denotes the value used for the analysis.

4. Longitudinal slopes are unbracketted and cross slopes are bracketted.