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KUGMALLIT CHANNEL

Geotechnical Investigation
Boring 6, Kenalooak J-94
Beaufort Sea

AUGUST 14, 1979

GEOTECHNICAL INVESTIGATION
BORING 6, KENALOOAK J-94
BEAUFORT SEA

Report to
CANADIAN MARINE DRILLING LTD.
Calgary, Canada

by

McClelland engineers, inc.

and

EBA Engineering Consultants Ltd.

GEOTECHNICAL INVESTIGATION
BORING 6, KENALOOAK J-94
BEAUFORT SEA

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CANADIAN MARINE DRILLING LTD.
Calgary, Alberta, Canada

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by

McCLELLAND ENGINEERS, INC.
Houston, Texas

and

EBA ENGINEERING CONSULTANTS LTD.
Edmonton, Alberta, Canada

December 1980

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Report No. 0179-0128-3
December 5, 1980

Canadian Marine Drilling Ltd.
P. O. Box 200
Calgary, Alberta, Canada T2P 2H8

Attention: Mr. Muharrem Gajtani

Geotechnical Investigation
Boring 6, Kenalooak J-94
Beaufort Sea

Gentlemen:

This report presents the results of our geotechnical investigation of soil and foundation conditions at the above offshore location. McClelland Engineers, Inc. and EBA Engineering Consultants Ltd. jointly performed this investigation under your Beaufort Sea Coring Services Contract dated June 19, 1979.

Various preliminary data and information have been provided to you from time to time upon request, including the results of the laboratory tests performed by EBA Engineering Consultants. All information developed in the field and laboratory is included here in detail along with our recommendations for design of conductor pipes and pile foundations.

We appreciate the opportunity to be of service to you in this interesting project. Please call on us when we can be of further assistance.

Very truly yours,

McCLELLAND ENGINEERS, INC.

Bernardo Susi

Bernardo Susi
Geotechnical Engineer

Clarence J. Ehlers

Clarence J. Ehlers, P.E.
Senior Engineer Manager

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Copies Submitted:

Canadian Marine Drilling Ltd. (10)
EBA Engineering Consultants Ltd. (3)

C O N T E N T S

ILLUSTRATIONS

SUMMARY

A geotechnical investigation of soil and foundation conditions was performed for Canadian Marine Drilling Ltd. at the Kenalooak J-94 exploratory drilling site in the Beaufort Sea. The study included a foundation boring drilled to 400-ft (121.9-m) penetration in 221 ft (67.4 m) of water. Field and laboratory tests were performed on soil samples retrieved from the boring, and engineering analyses were made of the field and laboratory data to compute the ultimate axial capacity of conductors and open-end pipe piles.

The boring disclosed fine-grained soils throughout the 400-ft penetration explored. The soils consist primarily of silty clays, clayey silts, and silts with some fine sand. The consistency of the clays is very soft at the seafloor, increasing with penetration and becoming very stiff at about 90-ft penetration. The sands and silts are generally medium dense to dense. Temperatures of retrieved soil samples were above the freezing point (0^{o} centigrade) to about 114-ft penetration. Below 114-ft penetration, the temperatures decreased below freezing, and the soils were classified as frozen. The ice content of the frozen soils is low. Visible ice is present from about 114 to 150-ft penetration. Below 150-ft penetration, the frozen soils are described as Nbn.

The axial capacity of conductor pipes and open-end pipe piles was analyzed for three design cases. The three design cases are (1) a driven conductor pipe with thawing of permafrost permitted, (2) a drilled-and-grouted insulated conductor pipe whose exterior surface is refrozen after installation, and (3) a driven open-end pipe pile assuming good contact between the pile surface and permafrost with no thawing of permafrost. Ultimate frictional capacity and unit end bearing curves are presented herein.

The driving of piles or conductors by conventional percussion techniques below about 114-ft penetration is expected to be very difficult, and driving refusal will probably be experienced because of the presence of permafrost. Methods to aid the driving of piles and conductors include prethawing or predrilling a hole in the permafrost and using a vibratory pile driver. An alternate method for placement of a conductor is by grouting the conductor in an oversized drilled hole.

INTRODUCTION

Project Description

Canadian Marine Drilling, Ltd. (Canmar) is developing their oil and gas leases in the southern Beaufort Sea. The development of these leases is currently in the exploration phase. Drilling operations for the exploration phase of development have been conducted from a fleet of drill ships which have been modified to be ice-classed.

The type of fixed structure that might be constructed at the Kenalooak J-94 location for support of drilling and production operations has not been decided. We understand that artificial islands constructed of dredged fill are being given consideration at this time. At the request of Canmar, this report does not present design analyses and recommendations for artificial islands.

Due to the presence of shallow permafrost soils, design of conductor pipe and foundation piles presents unique problems. Since the conductor pipe is supported in permafrost soils, special design precautions are taken. The design of conductors, therefore, includes the possibility of progressive thawing around the conductor during fluid extraction. The properties of thawed soils thus present the controlling factors for design. This report presents axial design analyses for conductor pipes and open-end pipe piles.

Purposes and Scope of Study

The purposes of this study were (1) to obtain information on soil stratigraphy and properties at the Kenalooak J-94 exploratory drilling site, (2) to determine the extent of permafrost within the 400-ft penetration explored, (3) to establish general geological control and extent of subsea permafrost from other borings in the southern Beaufort Sea, (4) to predict the ultimate axial capacity for conductor pipes and foundation piles installed to various penetrations, and (5) to develop installation recommendations for conductors and foundation piles. To accomplish these objectives, the study was conducted in the following phases:

- drilling of a soil boring to explore soil stratigraphy and to obtain soil samples

- laboratory testing of soil samples to determine pertinent engineering properties
- engineering analyses of the information obtained from the field and laboratory phases of the study to develop recommendations to guide axial design and installation of conductor pipes and foundation piles

Description of Job Responsibility

McClelland Engineers, Inc., Houston, Texas and EBA Engineering Consultants Ltd., Edmonton, Canada jointly performed this geotechnical investigation. McClelland Engineers provided two drillers, two helpers, one technician, and one geotechnical engineer, and EBA provided one technician or geotechnical engineer to perform the field investigation. EBA performed most of the laboratory soil tests with laboratory test assignments under the general direction of Professor N. R. Morgenstern at the University of Alberta. This report was prepared by McClelland Engineers. The boring log and many of the illustrations in Appendix A that present laboratory soil test results were prepared by EBA.

FIELD INVESTIGATION

An undisturbed-sample boring was drilled at the Kenalooak J-94 location at Latitude ~~13~~ 70°43'36.48" N and Longitude 133°58'09.96" W. The water depth at the site, measured by wire-line techniques, was 221 ft (67.4 m) at 1130 hrs on July 30, 1979. The drilling and sampling depths were not corrected for tidal variations. The boring was drilled to a total penetration of 400 ft (121.9 m) below the seafloor or El -621 ft (-189.3 m).

The boring was drilled with 4-in. IF drill pipe by a skid-mounted Failing 1500 rotary drilling rig operating through the centerwell in the deck of the Canmar supply boat, "Supplier V." Marine and drilling equipment were provided by Canmar. Drilling operations were conducted with a Reed core barrel equipped with a drag bit. The core barrel and collar have internal locking devices capable of accepting a wire line-retrievable rotary core barrel. Unfrozen soil samples were obtained by percussion technique at 3-ft intervals to 48-ft penetration, at 5-ft intervals from 58-ft to 114-ft penetration. A 2-1/2-in.-OD, 2-1/8-in.-ID liner sampler was used to obtain samples to 5-ft penetration, and a 2-1/4-in.-OD, 2-1/8-in.-ID thin-wall tube sampler was used to obtain all other unfrozen samples. These samplers were operated on a wire line through the bore of the drill pipe and driven with a 175-lb sliding weight. The weight was dropped approximately 5 ft a sufficient number of times to achieve the desired 24-in. penetration of the sampler.

Frozen soils were encountered below 114-ft penetration. Frozen soil samples from 114-ft to 168-ft penetration and from 208-ft to 258-ft penetration were obtained with a thin-walled tube sampler by the method discussed above. All other frozen soil samples were obtained with a Reed wire-line retrievable core barrel. Core runs of 10 ft were made from 170 ft to 200 ft and from 260-ft to 400-ft penetration. These runs generally achieved an average recovery rate of about 48 percent.

The temperature of most soil samples was measured prior to extruding them from the sampling tube or core barrel. A thermocouple with a 2-in.-long probe was inserted into the end of the sample while still in the sampling device. Temperature was then read to the nearest tenth of a degree Centigrade on a digital indicator. Since this is not an in situ technique of obtaining temperature, measurements are affected by ambient temperature, drilling fluid temperature (the drilling fluid was not chilled), and the time interval between obtaining a sample and making the temperature reading. Although temperature measurements are affected by the factors mentioned above, we feel that these temperature records may be valuable in the assessment of the presence of permafrost.

After the temperature was measured, each sample was removed from the sampling tube, photographed with a 35 mm SLR camera, and then examined and classified by our field engineer or soil technician. Frozen samples were quickly classified, appropriately packaged and stored in a freezer. The soil samples were then transported to EBA Engineering Consultants in Edmonton for subsequent laboratory testing. Photographs of the samples were presented in a bound report submitted previously.

Descriptions of the soils encountered in the boring are presented on the boring log shown on Plates 1 through 3. The boring log includes soil classifications according to the Unified Soil Classification (USC) System and ground ice descriptions. A key to the Unified Soil Classification and Ground

Ice Descriptions given on the log is presented on Plate 4. Sample temperatures measured in the field are plotted on Plate 5.

The amount of sampler penetration for driven samples and the number of blows of the sliding weight required to achieve that penetration is shown on the boring log. Blow counts with the wire-line sampler do not coincide with those obtained by the Standard Penetration Test (SPT). As an approximation, limited onshore tests indicate that driving resistance with the wire-line sampler is about 50 percent of driving resistance (N) from the Standard Penetration Test. As an example, we believe that 30 blows of the 175-lb wire-line sampler to achieve a penetration of 24 in. is roughly equivalent to 30 blows for 12-in. penetration for the SPT. We wish to stress that this is only a rough approximation based upon limited data obtained onshore at relatively shallow penetrations.

A brief chronological summary of the field operations at this location is given on Plate 6.

FIELD AND LABORATORY SOIL TESTS

The field and laboratory testing programs were designed to evaluate the pertinent engineering and physical properties of the foundation soils. We performed three types of strength tests in the field concurrently with drilling operations. Undisturbed shear strengths of most cohesive samples were determined with a motorized miniature vane device while the samples were still in the sampling tube. Unconfined compression tests were performed on selected specimens of cohesive soils after the samples were extruded from the sampling tube. Estimates of the shear strength of cohesive soils were also made using a Torvane.

All other tests, except constant rate of strain (CRS) thaw-consolidation tests, were conducted by EBA Engineering Consultants. The types and number of strength, thaw strain, and consolidation tests performed are as follows:

Type of Test	Number of Tests Laboratory	Number of Tests Field
Miniature Vane		
Undisturbed	9	18
Remolded	8	0
Torvane	0	26
Unconfined Compression	0	6
Unconsolidated-Undrained Triaxial Compression	14	0
Direct Shear	1	0
Thaw Strain	5	0
Consolidation	11	0
CRS Thaw-Consolidation	3	0

All strength tests were performed on thawed samples. A description of the CRS thaw-consolidation test procedure is presented in Appendix A.

Water content and unit weight determinations were made as routine parts of the test procedures for all specimens subjected to unconfined compression, triaxial compression, direct shear, thaw strain, and thaw-consolidation tests. Water content was also measured on the samples used in the miniature vane tests. Additional laboratory classification tests included the following:

Type of Test	Number of Tests
Water Content	22
Liquid and Plastic Limits	33
Grain-Size Analysis	
Sieve Analysis through No. 200 Sieve	16
Hydrometer	20
Pore Water Salinity	17
Specific Gravity	3
Bulk Density	8

Most test results are tabulated on Plate A-1 in Appendix A, and most results are presented in graphical form on the boring log on Plates 1 through 3. Field test results are plotted in red and laboratory tests in black to aid in the evaluation of the effect of thermal disturbance on measured shear strengths. The results of all tests including grain-size curves, liquidity index profile, pore water salinity profile, and specific gravity are presented on Plates A-2 through A-18. Results of all strength tests including unconsolidated-undrained triaxial compression, direct shear, miniature vane, unconfined compression, and Torvane are presented on Plates A-19 through A-28. The results of the thaw strain, consolidation, and thaw-consolidation tests are presented on Plates A-29 through A-43.

In addition to the above physical tests, selected samples were analyzed for geochemical constituents by Carbon Systems, Inc. of Baton Rouge, Louisiana. The samples were analyzed for light hydrocarbon gases, sulfate, salinity, dissolved inorganic carbon, and total organic carbon. The results of the geochemical analyses are presented in a report submitted by Carbon Systems, Inc. on December 13, 1979.

SOIL STRATIGRAPHY

The soils at the Kenalooak J-94 location are highly stratified and consist primarily of fine-grained soils that vary texturally from silt to clay. There are only minor quantities of fine sand, with no soils coarser than medium size sand. The soils are frozen below 114-ft penetration. A generalized summary of the major soil strata at Kenalooak J-94 based on the log of the boring presented on Plates 1, 2, and 3 is given in the following tabulation:

Stratum	Penetration, ft From	To	Description
1	0	3.5	Very soft silty clay
2	3.5	20.5	Clayey silt
3	20.5	45	Soft silty clay
4	45	72	Fine sand
5	72	116.5	Clayey silt; frozen below 114 ft
6	116.5	156	Frozen very stiff clay
7	156	177	Frozen silt
8	177	213	Frozen very stiff silty clay
9	213	269	Frozen silt
10	269	291	Frozen clay and silt
11	291	300	Frozen silty fine sand
12	300	328	Frozen silt
13	328	400+	Frozen clay and silt

Detailed descriptions that include color and textural variations, inclusions, and ground ice for each stratum are noted on the boring log. In addition, descriptions of sedimentary structure noted in the field are presented in Appendix B on Plate B-1.

Subsequent recommendations for foundation pile and conductor pipe design and installation contained in this report were developed assuming the soil conditions revealed by the boring. Considerations of possible stratigraphic changes, faulting, or other differences in soil conditions that could occur within the general area of the boring and that could influence foundation pile and conductor pipe design were beyond the scope of this investigation.

SOIL PROPERTIES

Definition of Permafrost

The classical definition of permafrost is any earth material that has been at or below the freezing temperature of water (0° C) for a prolonged period of time without regard to the state of any moisture present in the soil fabric. Generally, the smaller the grain size of a material, the lower the freezing point (the temperature at which there is no unfrozen moisture content). Therefore, a granular material such as sand can appear and feel frozen at the same temperature that a clay sample will appear and feel unfrozen. In this report, we have adopted the convention that materials would be classified as "frozen" if there was visible ice and the measured temperature of samples was below 0^{o} C.

According to the temperature measurements made of samples on the drilling boat, the temperatures of the sediments to about 114-ft penetration were above the freezing point (0^{o} C). Below 114-ft penetration, the temperatures decreased below freezing. A plot of temperatures of recovered soil samples measured during drilling operations is presented on Plate 5.

Unfrozen Soils

As discussed in the previous section, unfrozen soils were encountered from 0 to 114-ft penetration at the Kenalooak J-94 location. The unfrozen soils are silty clays, clayey silts, silts, and fine sands. The clays generally are of low to medium plasticity. Based on measured shear strengths and liquidity indices, the clays and silts appear to be normally consolidated with respect to the present overburden pressure. Based on consolidation tests, the calculated overconsolidation ratio (OCR) is generally near 1.0, which also indicates the soils are normally consolidated. A plot of overconsolidation ratio (OCR) versus penetration is presented on Plate A-44 in Appendix A.

The clays of Strata 1 and 3 range in consistency from very soft to soft. The clayey silt of Stratum 2 is loose based upon the driving resistance of the wire-line operated percussion soil sampler, or is soft in consistency based upon measured undrained shear strengths. The sand of Stratum 4 and the clayey silt of Stratum 5 are judged to be medium dense to dense.

Frozen Soils

Frozen soils or permafrost were encountered below 114-ft penetration. The frozen soils consist primarily of clays and silts of moderate plasticity. The liquid limits of the soils range from 27 to 57 with plasticity indices ranging from 8 to 33. The frozen soils contain visible ice from about 114 to 150-ft penetration. Below 150-ft penetration, there is no visible ice, and the frozen soils are described as Nbn (no excessive ice, well-bonded). The permafrost is not ice rich and is considered to be relatively thaw-stable due to its low ice content.

Based on measured shear strengths on thawed samples, the frozen clay soils appear to be normally consolidated to slightly underconsolidated. Incremental and CRS consolidation tests indicate that the clays are under-consolidated with overconsolidation ratios ratios ranging from about 0.2 to 0.70.

AXIAL CONDUCTOR AND PILE DESIGN ANALYSIS

A significant factor that affects the axial capacity of conductor pipes and foundaton piles in arctic regions is the thawing of frozen ground, or permafrost. Because the conductor pipe is used as a conduit to transfer superheated liquids or gases to the surface, a significant amount of thawing will occur where such a conductor comes in contact with frozen ground. When permafrost thaws, strength and volume changes may result. This thawing is not expected to occur with the foundation piles.

Canmar has requested that three design cases be considered in the analysis of the axial capacity of conductor pipes and foundation piles in frozen ground. The three design cases are:

Case 1: Driven conductor pipe with thawing of permafrost permitted.

Case 2: Drilled-and-grouted insulated conductor pipe whose exterior surface is refrozen after installation.

Case 3: Driven foundation pipe pile assuming good contact between pile surface and permafrost with no thawing of permafrost.

Method of Predicting Axial Capacity

Although the method of installation is different for a driven than for a drilled-and-grouted pile or conductor, we have assumed that the difference is negligible with respect to the method of predicting axial capacity for the two installation procedures (Cases 1, 2, and 3). There are other analytical methods available for computation of axial capacity of drilled-and-grouted piles and conductors; however, we feel that the method used for this study gives a reasonable prediction of axial capacity for drilled-and-grouted conductors (Case 2) as well as for driven piles and conductors (Cases 1 and 3).

The presence of permafrost will affect the axial capacity for the three design cases. The effect of permafrost is incorporated into the axial capacity analysis by changing the design parameters within the frozen soils for the three design cases. The method of predicting axial capacity is not affected and is identical for the three cases. Therefore, considering the methods of installation and the influence of permafrost, the parameters used in the axial capacity calculations are identical except in the frozen soils.

In addition to axial capacity considerations, pile and conductor settlement is an important design consideration. In frozen soils of high ice content, the creep rate under stress may cause excessive settlements over the design life of a pile-supported structure, which may govern design. For the low-ice-content permafrost at the Kenalooak J-94 site, settlement of piles and conductors is expected to be within tolerable limits if piles and conductors are designed in accordance with the method of predicting axial capacity described herein.

Foundation Pile. The ultimate axial capacity of foundation piles was predicted using the static method of analysis as described in API RP 2A (January 1980)[1]. In this method, the ultimate compressive capacity, Q, for a given penetration is taken as the sum of the friction on the wall of the pile, Q_s, and the end bearing, Q_p, so that:

$$Q = Q_s + Q_p$$

Conductor Pipe. The ultimate axial capacity of conductor pipes also was predicted using the static method of axial pile capacity analysis. In this method, the ultimate compressive and tensile capacity, Q, for a given penetration is taken as the friction on the wall of the conductor, Q_s, so that:

$$Q = Q_s$$

The end bearing component, Q_p, of axial pile capacity will not be developed by a conductor pipe and, therefore, is neglected.

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[1] American Petroleum Institute (1980), *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms,* API RP 2A, 11th Edition, 75 p.

Skin Friction

The skin friction, Q_s, is expressed as:

$$Q_s = fA_s$$

where f is the unit skin friction between soil and pile, and A_s is the embedded surface area of the pile (or the surface area of the drilled hole for the drilled-and-grouted conductor for Case 2). Computations based on the selected strength parameters shown in the left graph of Plate 7 and the criteria discussed in the following paragraphs produced the curves of unit skin friction shown in the left graph of Plate 8.

Cohesive Soils. Computation of unit skin friction, f, by the API RP 2A (January 1980) Method is in accordance with Sec. 2.6.4, Para. b.1. With this method, the unit skin friction may be equal to or less than the undrained shear strength of the clay, but may not exceed 1.0 kip per sq ft for shallow penetrations or the undrained shear strength equivalent to a normally consolidated clay for deeper penetrations, whichever is greater. The values of undrained shear strength used in our computations are based on the interpreted shear strength presented on the left graph of Plate 7. The unit skin friction values are plotted on the left graph of Plate 8.

Granular Soils. Computation of unit skin friction for pipe piles and conductors embedded in granular soils was in general accordance with Sec. 2.27, Para. c of API RP 2A (January 1980) and was obtained from the equation:

$$f = K \bar{\sigma}_v \tan \delta$$

where K = coefficient of lateral earth pressure
$\bar{\sigma}_v$ = effective vertical stress
δ = angle of friction between foundation material and steel pile

The value of K was taken as 0.7 for compressive loads and 0.5 for tensile loads. Effective vertical stress was computed from the submerged unit weight profile shown on the right graph of Plate 7. The angle of friction, δ, as well as the maximum limiting value of unit skin friction, f_{max}, considered appropriate for the granular materials are presented on the left graph of Plate 7. The values of δ and f_{max} are in accordance with those cited by McClelland[2].

Frozen Soils. The frozen soils encountered at the Kenalooak site range in grain size from clay to fine sand. The method of computation of adfreeze bond or unit skin friction, f, for frozen granular soils was assumed to be the same as described above for unfrozen granular soils. The maximum limiting values of unit skin friction, f_{max}, were assumed to be different for unfrozen and frozen granular soils. The limiting values of adfreeze bond (f_{max}) used for design Cases 1, 2, and 3 for different gradations of frozen granular soils are tabulated below:

	Fine Sand $\phi = 35^{\circ}$	Silty Sand $\phi = 30^{\circ}$	Sandy Silt $\phi = 25^{\circ}$	Silt and Clayey Silt $\phi = 20^{\circ}$
Case 1	0.80 ksf	0.70 ksf	0.55 ksf	0.40 ksf
Case 2	1.25 ksf	1.10 ksf	0.90 ksf	0.65 ksf
Case 3	1.80 ksf	1.60 ksf	1.25 ksf	0.90 ksf

The limiting adfreeze bond values for sand were used in our 1978 geotechnical investigations at Ukalerk, Tarsiut, Kopanoar, and Natsek where the frozen soils consisted primarily of fine sands. These limiting adfreeze bond values were based upon research conducted at the University of Alberta and were conferred to us by Professor N. R. Morgenstern. At the Kenalooak location, the frozen granular soils consist primarily of clays and silts with some silty sand. The adfreeze bond between the pile surface and soil is dependent on soil grain size and decreases with decreasing grain size. The limiting adfreeze bond values presented above for silty sand, sandy silt, and silt were proportioned to the values for sand in accordance with the proportions cited by McClelland[2] for limiting skin friction values for unfrozen granular soils.

The adfreeze bond and corresponding axial load capacity of piles in frozen soils is temperature dependent. The warmer the temperature, the lower the adfreeze bond. The adfreeze bond values presented herein assume that the frozen soils are warm, i.e., slightly below freezing. In the future,

[2] McClelland, Bramlette (1974), "Design of Deep Penetration Piles for Ocean Structures," *Proceedings*, ASCE, Vol. 100, No. GT7.

if in situ measurements of temperature indicate that the frozen soils are colder than assumed, an increase in the adfreeze bond values may be possible.

The ice content of the frozen cohesive soils was low, and based on field descriptions, a portion of the moisture present in the soil fabric was unfrozen. Therefore, when computing adfreeze bond in frozen cohesive soils, we assumed that the soils would behave similar to unfrozen cohesive soils. Therefore, unit skin friction was computed using the same procedure used for unfrozen cohesive soils.

End Bearing

The end bearing, Q_p, is expressed as:

$$Q_p = q \, A_p$$

where A_p is the gross end area of the pile and q is the unit end bearing. Computations based on the selected strength parameters shown in the left graph of Plate 7 and the criteria discussed in the following paragraphs produced the curve of unit end bearing in the right graph of Plate 8.

For the conductor (Cases 1 and 2), there will be no end bearing. For driven open-end pipe piles, the end bearing must be limited to the frictional resistance of the soil plug inside the pile. This limiting value was taken as the cumulative frictional resistance along the embedded length of the pile.

Cohesive Soils. The unit end bearing of piles in clay was computed using the following equation:

$$q = s_u \, N_c$$

where s_u = undrained shear strength

N_c = a dimensionless bearing capacity factor

A value of 9 was used for N_c as recommended by API RP 2A. The values of s_u correspond to the shear strength profile plotted on the left graph of Plate 7.

Granular Soils. The unit end bearing for piles in granular material was computed from the equation:

$$q = \bar{\sigma}_v \, N'_q$$

where $\bar{\sigma}_v$ = effective vertical stress

N'_q = dimensionless bearing capacity factor which is a function of ϕ, the angle of internal friction of the material

The effective vertical stress, $\bar{\sigma}_v$, was calculated using the submerged unit weight profile given in the right graph of Plate 7. Values of N'_q selected for the granular materials are taken from API RP 2A (January 1980) and are presented on the left graph of Plate 7 along with the maximum limiting values of unit end bearing, q_{max}, considered appropriate for these materials (see McClelland[(2)]). Full end bearing as computed for granular strata cannot be relied upon unless the pile has penetrated a distance of at least three pile diameters into the strata, and the strata continues at least three pile diameters below the pile tip.

Frozen Soils. The bearing capacity of frozen soil is a function of many variables including temperature of permafrost, ice content, salinity of pore water, rate of loading, and grain size. Generally, the bearing capacity of soil in a frozen state will be higher than in an unfrozen state; however, the axial pile movement or deformation required to develop the higher bearing capacity in frozen soils is expected to be greater than in unfrozen soils. As a simplifying assumption to reduce pile movements, the end bearing for soil in the frozen state was assumed to be the same as in the unfrozen state.

An important consideration pertaining to end bearing in frozen soil is resistance to pile driving. We anticipate that refusal to conventional percussion pile driving will be encountered in the frozen soils below about 114-ft penetration. Installation techniques to aid pile driving in permafrost are discussed in a subsequent section of this report.

Ultimate Frictional Pile Capacity Curves

Using the design criteria discussed above, the ultimate frictional capacity was computed for a unit circumference pipe pile or conductor (12-in. circumference) assuming the three design cases previously discussed. Cases 1, 2, and 3 differ only below the depth of initial contact with permafrost due to the changes in limiting adfreeze bond, f_{max}, in the frozen granular soils. There is a very small difference between the compressive and tensile capacities for Cases 1 and 2. Therefore, we have assumed that the ultimate frictional compressive capacity is the same as the ultimate frictional tensile capacity for these two cases. Curves of ultimate frictional pile capacity are presented on Plate 9.

The ultimate frictional pile capacity curves may be used in determining the frictional capacities for any other size pile or conductor by multiplying the curve coordinates by the ratio of the new pile circumference to a 12-in. pile circumference. The resulting curve will depict total axial pile capacity for Cases 1 and 2, while for Case 3, end bearing must be added. End bearing may be computed by multiplying the gross end area of the pile tip by the value of unit end bearing shown on the right graph of Plate 8. The end bearing component of capacity must not exceed the frictional resistance of a soil plug inside the pile at any penetration.

PILE AND CONDUCTOR INSTALLATION CONSIDERATIONS

As defined by the three design cases, there are two proposed methods of installation for the conductor pipes and foundation piles, namely: (1) driving and (2) a drilled-and-grouted technique. Comments and recommendations concerning the two installation techniques are discussed in the following paragraphs.

Driven Piles and Conductors

Installation of driven piles and conductors at the Kenalooak J-94 location is complicated by the presence of frozen soils. Below about 114-ft penetration, the advancement of piles or conductors by conventional percussion driving is expected to be very difficult, and driving refusal will probably be experienced because of the presence of permafrost.

Alternate methods used onshore to aid the advancement of driven piles in permafrost are many. Some of these methods are:

- prethawing a hole in permafrost with a steam jet, either larger or smaller than the pile, and driving the pile in place
- rotary drilling a hole and driving a pile in an undersized hole, or "slurring" back in an oversized hole
- driving piles using a vibratory pile driver

Some of these techniques could be adaptable for use offshore. The most feasible alternatives would probably be pile driving assistance by vibratory techniques or predrilling of a pilot hole. The availability of high energy vibratory hammers capable of driving large diameter piles offshore is doubtful, but the possible use of a vibratory hammer should be given further consideration.

Predrilling a pilot hole, having a diameter less than that of the pile or conductor, may aid the advancement of a pile because of the reduction of end bearing. Caution must be exercised to insure that the pilot hole is not drilled larger than the pile or conductor, resulting in reduced skin friction. The optimum diameter of the pilot hole is primarily a function of the pile or conductor diameter, the strength and temperature of the permafrost, and the energy and type of driving hammer. If pilot holes are used as an aid to driving, we recommend that pilot holes have a diameter approximately one-half the diameter of the pile and be drilled no deeper than 10 pile diameters below the pile tip. If pile driving is not aided, additional increases (about 10 to 15 percent of pile diameter) in the size of the pilot hole may be made up to a maximum of 90 percent of the pile diameter.

For a foundation pile where an appreciable amount of end bearing is counted upon for capacity, we recommend that drilling of pilot holes be kept to a level of at least 10 ft above design tip elevation and that driving alone be used to advance piles for the final several feet of penetration. The soil plug that results from this final driving should be left in place and a cement grout plug should be placed in the bottom of the piles to restore end bearing. The length of the grout plug will depend on the end bearing capacity of the soils beneath the pile tip. We recommend that an engineer familiar with pile driving and the effects of predrilling on pile design parameters be on site at the time of pile installation.

Drilled-and-Grouted Conductors

An alternate method for placement of the conductor pipe is by grouting the conductor pipe in an oversized drilled hole. During the installation of a drilled-and-grouted conductor, a number of construction-related phenomena can influence its axial capacity. Some of these phenomena are:

- stress release due to drilling
- mechanical disturbance to the soil fabric
- formation of a mudcake on the borehole wall
- interference of drilling fluid with the development of grout-pile bond
- migration of soil, water, and cement due to chemical potential gradients

When conductors are drilled and grouted in frozen soils, additional problems related to thawing of frozen materials may occur.

To minimize construction problems and develop the maximum axial capacity for drilled-and-grouted conductors, the following recommendations for construction surveillance are made:

(1) Minimize the amount of time between the completion of the drilled excavation and the completion of the grouting activities.

(2) Monitor the viscosity and weight of drilling fluid during construction, weighing the benefit of the drilling mud's reduction of stress relief versus its detrimental effect on grout-pile bonding.

(3) Utilize grout placement techniques that minimize drilling fluid contamination of bonding surfaces.

(4) Control drilling procedures that could influence borehole stability and create mechanical disturbance of soil fabric.

(5) Insure that the entire length of the annulus has been grouted and grout returns to seafloor.

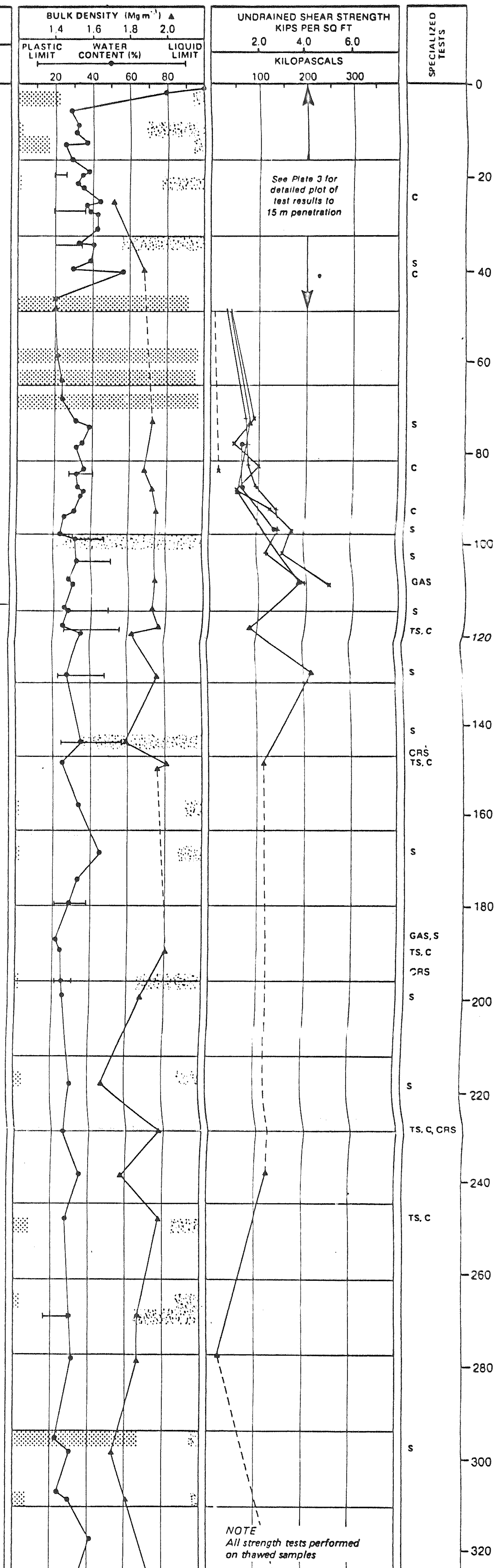
(6) Design grout mixes to promote water migration from soil to grout.

I L L U S T R A T I O N S

MCCLELLAND ENGINEERS

LOCATION: BORING 6, KENALOOAK J-94
LAT. 69°43' 36.48" N LONG. 133° 58' 09.96" W WATER DEPTH: 67.4 m**

Penetration below seafloor, metres	USC	SMPL	SOIL DESCRIPTION	*BLOW COUNT	GROUND ICE
0	CL	J	CLAY - silty, trace organics, shell fragments, grey (3.5')	0	NOT FROZEN
	ML	J	SILT - clayey, trace to some sand, sandy silt	0	
		J, O	- interbedded with soft clay, grey	0, 9/24"	
			- dark olive grey 5Y 3/2	10/24"	
5		J	- 6.5% organic content	11/24"	
		O	(20.5')	7/24"	
	CL	O	CLAY - silty, trace sand, some silt seams and pockets, soft, very dark grey, 5Y 3/1	7/24"	
		J	- grey 5Y 5/1, medium plasticity	7/24"	
10		J	- organic streaks in top 0.61 m	5/24"	
		O	- some organics from 10.5 m to 11.1 m	7/24"	
		J	- dark olive grey 5Y 3/2	7/24"	
		O	- 11.1% organic content	8/24"	
			- olive grey 5Y 4/2, olive grey 5Y 5/2 (45')	8/24"	
15	SP	J	SAND - trace silt, fine-grained, uniform, olive 5Y 4/3	30/20"	
		J		30/12"	
		J		30/12"	
20		J		30/24"	
		J	(72')	25/24"	
	ML	O, J	SILT - clayey, stiff, some silt seams and pockets to 29 m, dark olive grey, 5Y 3/2, grey 5Y 6/1	17/24"	
		J		17/24"	
25		O, J	- very stiff, medium plasticity	17/24"	
		O, J	- very dark grey 5Y 3/1, grey 5Y 5/1	17/24"	
		O		25/18"	
30		J, J	- 11.4% organic content	25/24"	
		J	- high plasticity	25/24"	
	MH	O, J	- olive 5Y 4/3, light grey 5Y 7/1	30/12"	
			- random shear planes		
35		PF	- (ML), trace organic streaks (116.5')	30/12"	Vx 10%
	CH	PF	CLAY - silty, random shear planes, dark olive grey 5Y 3/2, grey 5Y 6/1	30/15"	Vr 5-10%
			- clear ice	30/12"	Vx
40		PF	- (CL), olive grey 5Y 4/2, olive grey 5Y 5/2, ice 15° to horizontal	30/12"	Vr
				30/18"	Vx, Vr
			- greyish brown		
		PF		30/18"	Vx, Vr-10%
45		PF	- dark olive grey 5Y 3/2, olive grey, 5Y 5/2 (156')	30/12"	Vs-5%
	ML	J	SILT - trace clay, trace sand, nonplastic, dark olive grey, 5Y 3/2	30/6"	
50					Nbn
		J	- some sand to sandy with silty clay seams and pockets	30/6"	
		J	(177')		
55	CL	J	CLAY - silty, trace sand, numerous silt seams and pockets, gas bubbles, organic stains, blocky structure to 58 m, dark grey 5Y 4/1, grey 5Y 5/1		
		J			Nbn
			- dark olive grey, 5Y 3/2		
60		J, PF			
		PF	(213')	30/18"	Nf
65	ML	PF	SILT - some clay, some clay seams, trace fine sand, some organics, nonplastic, greyish brown	30/12"	
70		PF	- laminations and interbedding with light greyish brown silty clay, organic streaks, dark olive grey 5Y 3/2, grey 5Y 6/1	30/18"	Nf
		PF	- sandy, clay and silty clay seams and pockets, dark grey 5Y 4/1, grey 5Y 5/1	30/12"	Nbn
75		PF	- some very fine sand, greyish brown with light brown horizontal banding (1 mm) and grey clay lenses to 3 mm	30/18"	Nbn
80				30/12"	
			- black 5Y 2.5/1, 7.9% organic content (269')		
	CL	PF	CLAY AND SILT - medium plasticity, black 5Y 2.5/1		
85		PF	- wood and shell fragments, nonplastic, dark olive grey 5Y 3/2, grey 5Y 5/1 (291')		Nbn
90	SM	J, PF	SAND - silty, trace clay, fine-grained, uniform, dark olive grey 5Y 3/2 (300')		Nbn
	ML	J	SILT - trace sand, trace clay, nonplastic, very dark grey 5Y 3/1, light brown horizontal banding		
95		J			Nbn
100					



eba	McCLELLAND ENGINEERS JOB NO. 0179-0128 EBA JOB NO. 101-2581 DRILLING COMPLETED: AUGUST 14, 1979 BOREHOLE PENETRATION: 400' FIELD ENGINEER: D.B. PETERS	LEGEND GRAIN SIZE: % Sand, % Silt, % Clay SHEAR STRENGTH: + Torvane, × Min. Vane, ● Unconfined Compr., ▲ UU Triaxial, ■ CU Triaxial, Field Laboratory Test Results TEST IDENTIFICATION: C - Consolidation, TS - Thaw Strain, DS - Direct Shear, CRS - Consolidation, T - Triaxial Shear, S - P.W. Salinity, X - X-Ray Diffr.	

BOREHOLE LOG AND LABORATORY TEST RESULTS

PLATE 1

McCLELLAND ENGINEERS

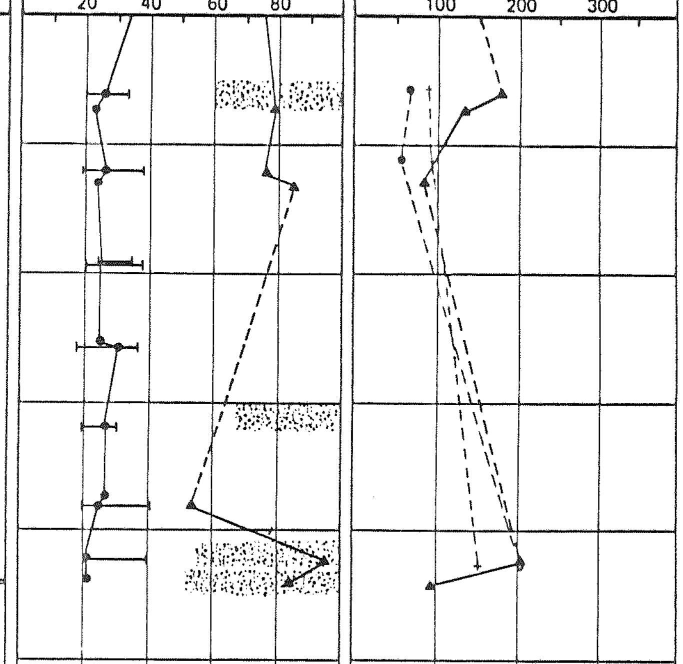
BOREHOLE LOG AND LABORATORY TEST RESULTS

LOCATION: BORING 6, KENALOOAK J-94
LAT. 13° 43' 36.48" N LONG. 133° 58' 09.96" W
WATER DEPTH: 64.7 m**

Penetration below seafloor, metres	USC	SMPL.	SOIL DESCRIPTION	*BLOW COUNT	GROUND ICE	Specialized tests	Penetration below seafloor, feet
100			CLAY AND SILT - medium plasticity, stiff, numerous silt seams, dark olive grey 5Y 3/2, light grey 5Y 6/1		Nbn	S	
		J, PF				C	340
105		PF, Q	· laminations at 25° to horizontal, some organic streaks, very stiff, dark grey 5Y 4/1, grey 5Y 5/1		Nbn	DS, S	
110	CL	J			Nbn		360
		J				S	
115		PF	· some horizontal partings		Nbn		380
		J					
120		Q, PF	· dark olive grey 5Y 3/2, olive grey 5Y 5/2 (400')		Nbn	C, S	400
			END OF BOREHOLE 121.9				
125							

BULK DENSITY (Mg m⁻³) ▲ 1.4 1.6 1.8 2.0 — PLASTIC LIMIT, WATER CONTENT (%), LIQUID LIMIT 20 40 60 80

UNDRAINED SHEAR STRENGTH KIPS PER SQ FT 2.0 4.0 6.0 — KILOPASCALS 100 200 300



*Number of blows of a 175-lb weight dropped approximately 5-ft as required to produce a 24-in. penetration, except as noted, of a 2.25-in.-OD, 2.125-in.-ID thin-wall tube sampler. A 2.5-in.-OD, 2.125-in.-ID liner sampler was used to 5-ft penetration.

**Water depth measured at 1130 hours on July 30, 1979.

NOTE
All strength tests performed on thawed samples

McCLELLAND ENGINEERS JOB NO. 0179-0128
EBA JOB NO. 101-2581
DRILLING COMPLETED: AUGUST 14, 1979
BOREHOLE PENETRATION: 400'
FIELD ENGINEER: D.B. PETERS

LEGEND

GRAIN SIZE: % Sand, % Silt, % Clay

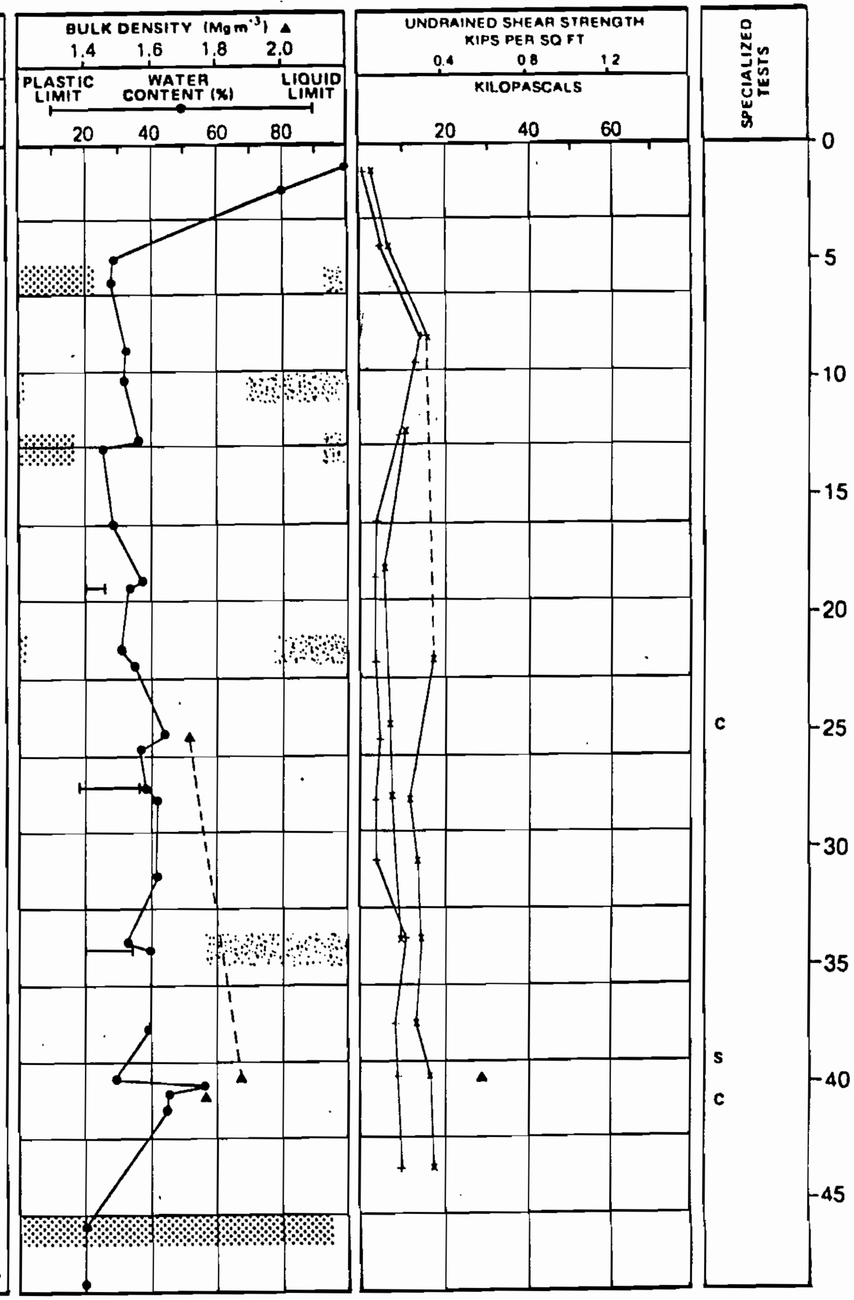
SHEAR STRENGTH: + Torvane, x Min. Vane, ● Unconfined Compr., ▲ UU Triaxial, ■ CU Triaxial — Field Laboratory Test Results

TEST IDENTIFICATION: C - Consolidation, TS - Thaw Strain, DS - Direct Shear, CRS - Consolidation, T - Triaxial Shear, S - P.W. Salinity, X - X-Ray Diffr.

PLATE 9

LOCATION: BORING 6, KENALOOAK J-94
LAT. 13° 43' 36.48" N LONG. 133° 58' 09.96" W WATER DEPTH: 67.4 m**

Penetration below seafloor, metres	USC	SMPL.	SOIL DESCRIPTION	*BLOW COUNT	GROUND ICE
0–1	CL	J, J	CLAY - silty, trace organics, shell fragments, grey (3.5')	0	NOT FROZEN
1–6.7	ML	J, J, J, J, Q, J, J, Q	SILT - clayey, trace to some sand, sandy silt interbedded with soft clay, grey - dark olive grey 5Y 3/2 - 6.5% organic content (20.5')	0 0 0 9/24" 10/24" 11/24" 7/24"	
6.7–13.5	CL	J, Q, Q, J, J, Q, J, Q, J, J, Q, J, J	CLAY - silty, trace sand, some silt seams and pockets, soft, medium plasticity, very dark grey, 5Y 3/1, grey 5Y 5/1, organic streaks in top 0.6 m - some organics from 10.5 m to 11.1 m - dark olive grey 5Y 3/2 - 11.1% organic content - olive grey, 5Y 4/2, olive grey 5Y 5/2 (45')	7/24" 7/24" 7/24" 5/24" 7/24" 7/24" 8/24" 8/24"	
13.5–15	SP	J, J	SAND - trace silt, fine-grained, uniform, olive 5Y 4/3	30/20" 30/12"	



McCLELLAND ENGINEERS JOB NO. 0179-0128
EBA JOB NO. 101-2581
DRILLING COMPLETED: AUGUST 14, 1979
BOREHOLE PENETRATION: 400'
FIELD ENGINEER: D.B. PETERS

LEGEND

GRAIN SIZE: % Sand, % Silt, % Clay

SHEAR STRENGTH: + Torvane; x Min. Vane; ● Unconfined Compr.; ▲ UU Triaxial; ■ CU Triaxial; Field Laboratory Test Results

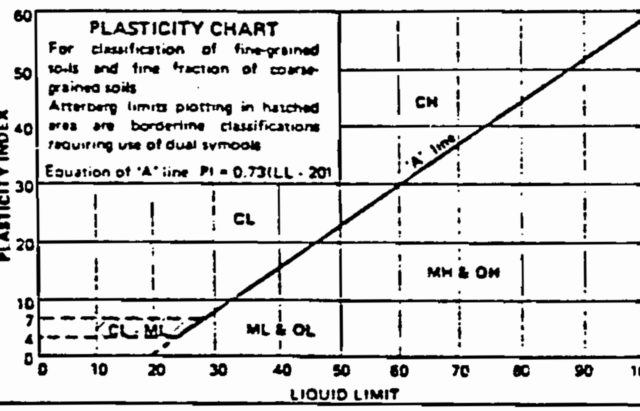
TEST IDENTIFICATION: C - Consolidation; TS - Thaw Strain; DS - Direct Shear; CRS - Consolidation; T - Triaxial Shear; S - P.W. Salinity; X - X-Ray Diffr.

BOREHOLE LOG AND LABORATORY TEST RESULTS

UNIFIED SOIL CLASSIFICATION†

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES	CLASSIFICATION CRITERIA	
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve*	GRAVELS 50% or more of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	$C_u = D_{60}/D_{10}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP	Poorly-graded gravels and gravel-sand mixtures, little or no fines	Not meeting both criteria for GW	
		GRAVELS WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures	Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures	Atterberg limits plot above "A" line and plasticity index greater than 7	
	SANDS More than 50% of coarse fraction passes No. 4 sieve	CLEAN SANDS	SW	Well-graded sands and gravelly sands, little or no fines	$C_u = D_{60}/D_{10}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP	Poorly-graded sands and gravelly sands, little or no fines	Not meeting both criteria for SW	
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures	Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures	Atterberg limits plot above "A" line and plasticity index greater than 7	
FINE-GRAINED SOILS 50% or more passes No. 200 sieve*	SILTS AND CLAYS	Liquid limit 50% or less	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands		
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
			OL	Organic silts and organic silty clays of low plasticity		
	SILTS AND CLAYS	Liquid limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts		
			CH	Inorganic silts of high plasticity, fat clays		
			OH	Organic clays of medium to high plasticity		
HIGHLY ORGANIC SOILS			PT	Peat, muck and other highly organic soils		

Classification on basis of percentage of fines: Less than 5% pass No. 200 sieve: GW, GP, SW, SP; More than 12% pass No. 200 sieve: GM, GC, SM, SC; 5% to 12% pass No. 200 sieve: Borderline classification requiring use of dual symbols



*Based on the material passing the 3 in. (75 mm) sieve
†ASTM Designation D 2487, for identification procedure see D 2488

GROUND ICE DESCRIPTION

ICE NOT VISIBLE

GROUP SYMBOLS	SYMBOLS	SUBGROUP DESCRIPTION
N	Nf	Poorly-bonded or friable
	Nbn	No excess ice, well-bonded
	Nbe	Excess ice, well-bonded

VISIBLE ICE LESS THAN 50% BY VOLUME

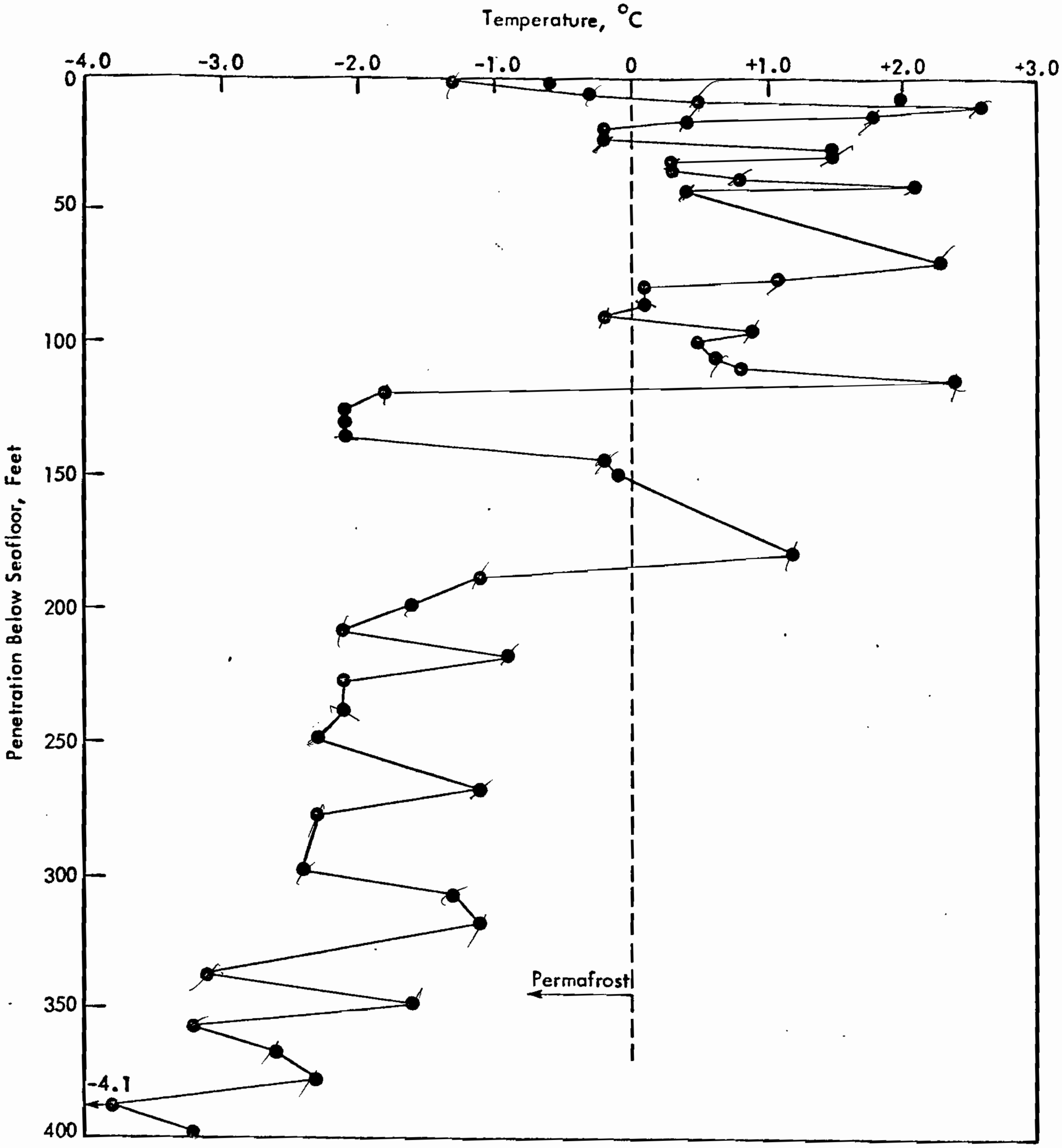
GROUP SYMBOLS	SYMBOLS	SUBGROUP DESCRIPTION
V	Vx	Individual ice crystals or inclusions
	Vc	Ice coatings on particles
	Vr	Random or irregularly oriented ice formations
	Vs	Stratified or distinctly oriented ice formations

VISIBLE ICE GREATER THAN 50% BY VOLUME

GROUP SYMBOLS	SYMBOLS	SUBGROUP DESCRIPTION
ICE	ICE + Soil Type	Ice with soil inclusions
	ICE	Ice without soil inclusions (greater than 25 mm (1 in.) thick)

NOTE:
1. *Dual symbols are used to indicate borderline or mixed ice classifications*
2. *Visual estimates of ice contents indicated on borehole logs ± 5%*
3. *This system of ground ice description has been modified from NRC Technical Memo 79, Guide to the Field Description of Permafrost for Engineering Purposes*

LEGEND
Soil Ice



TEMPERATURE vs DEPTH
Boring 6, Kenalooak J-94
Beaufort Sea

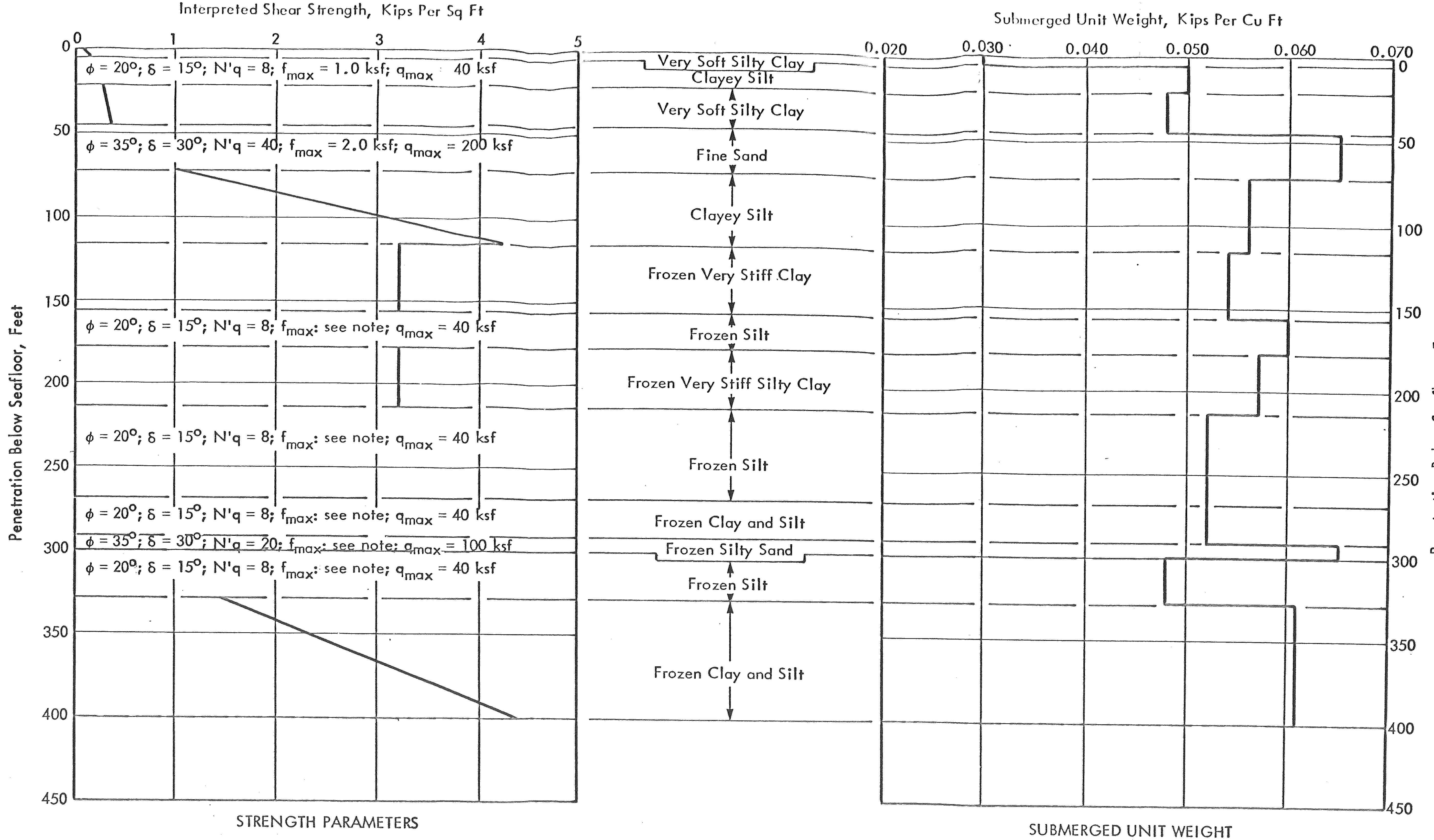
Date	Date From	Date To	Description
July 30, 1979		0145	Depart Kilannak for Nerlerk
	0145	1200	Traveling to Nerlerk
	1200	1230	Standing by near Explorer I and Supplier VI
	1230	1615	Taking water and supplies from Supplier VI at Nerlerk
	1615	1845	Traveling to Kenalooak J-94
	1845	2015	Anchoring
	2015	2200	Replacing anchor cable
	2200	2345	Anchoring
	2345	2400	Running pipe to mudline
July 31, 1979	0000	0215	Running pipe to mudline
	0215	0700	Drilling Boring 6 to 48-ft penetration
	0700	0815	Cleaning water and residue from main engine fuel lines
	0815	0915	Waiting on weather
	0915	0930	Taking sample at 48 to 50-ft penetration
	0930	1230	Pulling pipe
	1230	2400	Waiting on weather
August 1, 1979	0000	1000	Waiting on weather
	1100	1300	Pulling anchors and waiting on weather
	1300	1330	Raising drill collar and core barrel in derrick
	1330	1730	Traveling to Nerlerk
	1730	2400	Standing by to moor to Explorer I at Nerlerk
August 2, 1979	0000	1715	Standing by to moor to Explorer I at Nerlerk; pick up D. Tucker and supplies at Explorer I at 1600 hours
	1715	2000	Traveling to Kenalooak
	2000	2400	Waiting on weather

(Continued on Plate 6a)

SUMMARY OF FIELD OPERATIONS
Boring 6, Kenalooak J-94
Beaufort Sea

Date	Date From	Date To	Description
	(Continued from Plate 6)		
August 3, 1979	0000	0100	Waiting on weather
	0100	0200	Anchoring
	0200	0230	Establishing coordinates and positioning
	0230	0400	Running pipe to mudline
	0400	2400	Drilling Boring 6 to 300-ft penetration
August 4, 1979	0000	1145	Drilling Boring 6 to 400-ft penetration
	1145	1515	Pulling pipe
	1515	1715	Pulling anchors
	1715		Traveling to Nerlerk

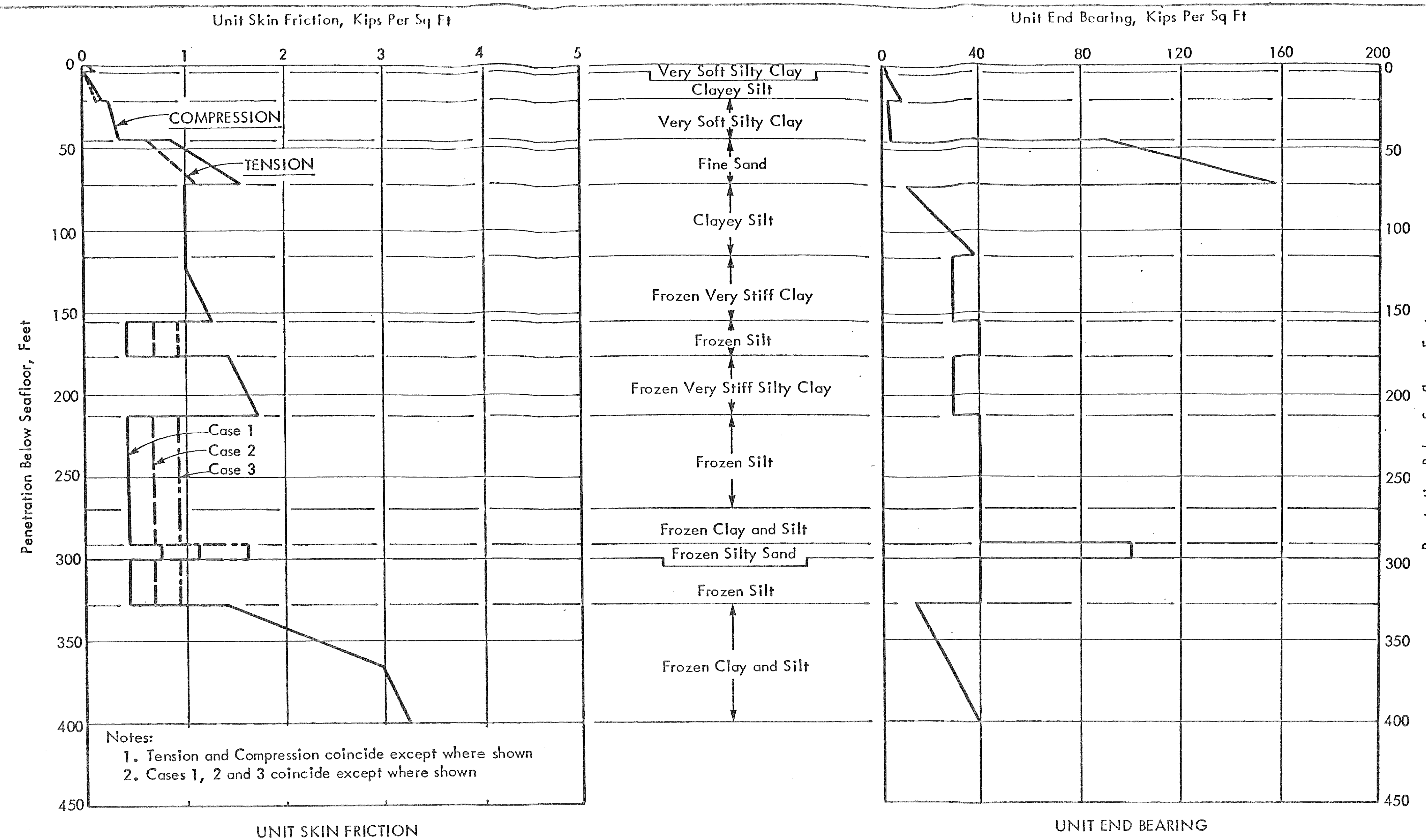
SUMMARY OF FIELD OPERATIONS
Boring 6, Kenalooak J-94
Beaufort Sea



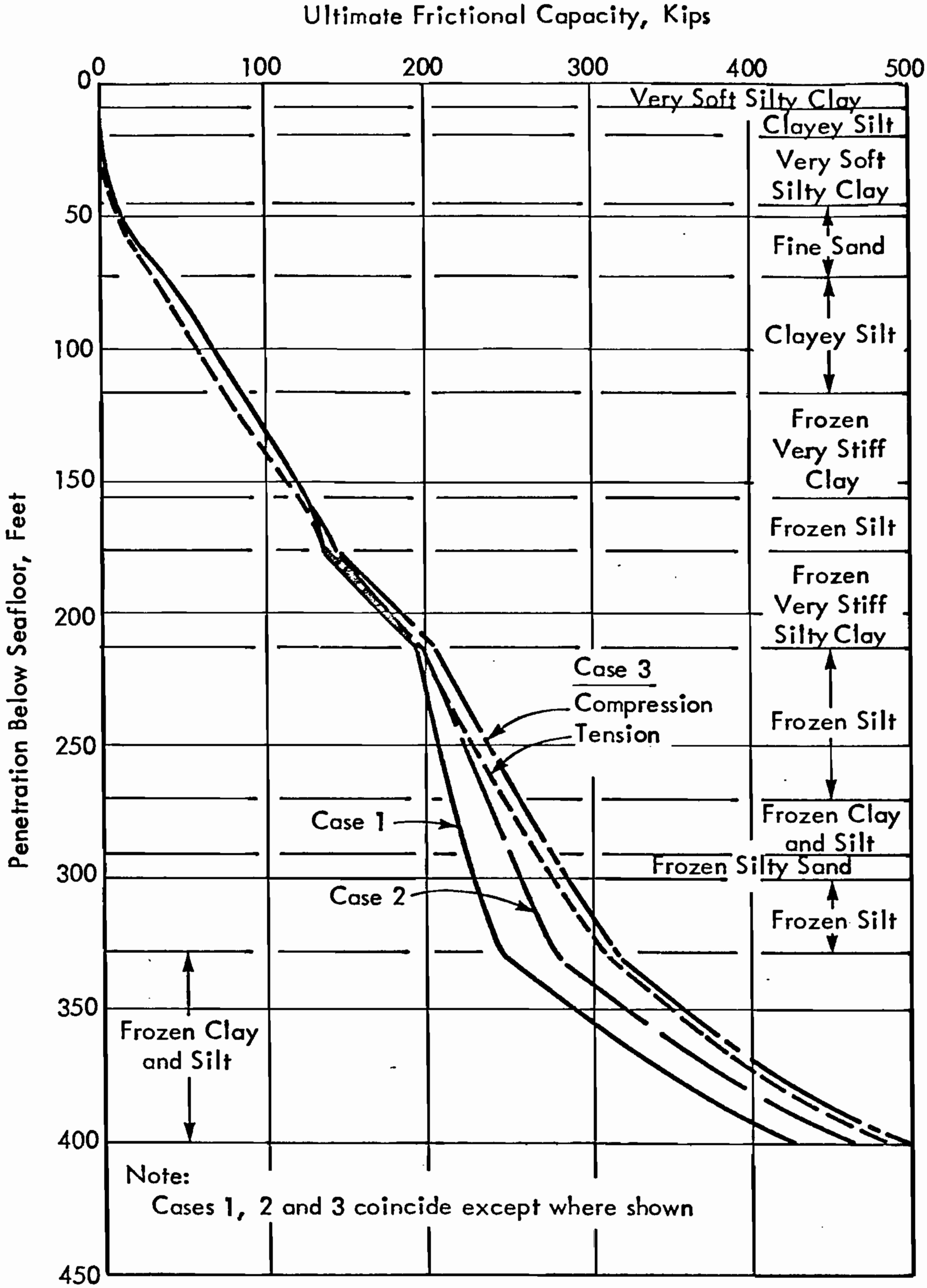
Note:

	Sand	Silty Sand	Sandy Silt	Silt
Case 1: f_{max} =	0.80 ksf	0.70 ksf	0.55 ksf	0.40 ksf
Case 2: f_{max} =	1.25 ksf	1.10 ksf	0.90 ksf	0.65 ksf
Case 3: f_{max} =	1.80 ksf	1.60 ksf	1.25 ksf	0.90 ksf

INTERPRETATION OF DATA
Boring 6, Kenalooak J-94
Beaufort Sea



UNIT PILE DESIGN DATA
Boring 6, Kenalooak J-94
Beaufort Sea



ULTIMATE FRICTIONAL CAPACITY CURVES
12-in. Circumference Pile
API RP 2A (January 1980) Method
Boring 6, Kenlooak J-94
Beaufort Sea

A P P E N D I X A

FIELD AND LABORATORY SOIL TEST RESULTS

APPENDIX A

FIELD AND LABORATORY SOIL TEST RESULTS

CONTENTS

ILLUSTRATIONS

APPENDIX A

Description of CRS Thaw-Consolidation Test

CRS (Constant Rate of Strain) thaw-consolidation tests were performed on three selected frozen samples. For this type of test, loading is not applied in the conventional manner of incremental loads; instead load is applied to the specimen at a constant rate of strain.

The consolidation cell was prepared by soaking the base and bottom porous stone in a solution of deaired water and ethylene glycol containing 20 to 22 percent glycol by weight. The base was then deaired by flushing glycol solution through the drainage holes and pore pressure transducers. The bottom porous stone, cell, and base were then placed in a cold room.

The frozen specimen was carefully trimmed into a 1.875-in.-ID stainless steel consolidation ring and fitted with porous stones at each end. The consolidation cell was assembled, insulated, and packed with ice for transportation to the loading frame in the laboratory. Upon arrival at the laboratory, the consolidation cell was placed in the loading frame. During the thaw cycle, the cell was filled with water and a seating load of 50 kPa was applied to the specimen using a pneumatic piston. A linear vertical displacement transducer (LVDT) measured vertical deformation during thaw. These deformation readings were plotted versus time to indicate when the thaw cycle was complete. The load piston was then locked off to begin the saturation cycle. Back-pressure saturation was applied in 50 kPa increments to achieve a "B" pore pressure parameter of 0.9 or better. The constant rate of strain load cycle began at this point.

During the load cycle, a data logger monitored vertical load, deformation, and excess pore pressure continuously through electronic transducers. All specimens were loaded to the estimated preconsolidation pressure, unloaded, and then reloaded. The rate of strain was selected to produce a minimum excess pore pressure of 7 kPa and to limit the ratio of excess pore pressure to vertical pressure (u_e/σ_v) to 20 percent. Test data were used to compute vertical strain and vertical effective pressure. The results of the CRS test are presented on Plates A-41 through A-43.

APPENDIX A

ILLUSTRATIONS

SUMMARY OF LABORATORY TESTING RESULTS — PERMAFROST

SAMPLE NO.	SAMPLE TYPE	DEPTH INTERVAL (metres)	MOIST. CONT. (%)	BULK DENS. (Mg/m³)	GROUND ICE DESCRIPTION (%)	ATTERBERG LIMITS LL (%)	ATTERBERG LIMITS PL (%)	ATTERBERG LIMITS PI (%)	GRAIN SIZE DISTRIBUTION CLAY (%)	GRAIN SIZE DISTRIBUTION SILT (%)	GRAIN SIZE DISTRIBUTION SAND (%)	GRAIN SIZE DISTRIBUTION GRAV. (%)	ORGANIC CONTENT (%)	SOIL DESCRIPTION
1	J	0.3	100		Not Frozen									
33	Tw	0.5			Not Frozen									
2	J	0.6	79		Not Frozen									
35	Tw	1.2			Not Frozen									
36	Tw	1.4			Not Frozen									
37	Jv	1.5	29		Not Frozen									
38	J	1.8	28		Not Frozen	Nonplastic			6	71	23	0		(ML) SILT
3	T	2.6			Not Frozen									
4	Jv	2.7	33		Not Frozen	Nonplastic								(ML) SILT
5	J	2.9			Not Frozen									
6	J	3.1	32		Not Frozen				31	67	2	0		(ML) SILT
7	Q	3.8 - 4.0	37		Not Frozen									
8	J	4.0	26		Not Frozen	Nonplastic			6	76	18	0		(ML) SILT
9	J	4.7			Not Frozen									
10	J	5.0	29		Not Frozen									
11	Q	5.6 - 5.8	38		Not Frozen									
12	J	5.8	35		Not Frozen	26	20	6					6.5	(CL-ML) SILT
13	Jv	6.6	32		Not Frozen	Nonplastic			22	76	2	0		(ML) SILT
14	Q	6.7 - 6.9	36		Not Frozen									
15	Q	7.6 - 7.8	45	1.72	Not Frozen									
16	J	7.9	37		Not Frozen									
17	Jv	8.4	39		Not Frozen	37	19	18						(CL) CLAY
18	Q	8.5 - 8.7	43		Not Frozen									
19	Q	9.3 - 9.5			Not Frozen									
20	Jv	9.6	43		Not Frozen									

$I_L = \frac{35-20}{6} = 2.5$

$I_L = \frac{3?-??}{?3} = 1.1$ [illegible]

PROJECT NUMBER 101-2581 SITE NUMBER KENALOOAK BOREHOLE B6

EBA Engineering Consultants Ltd.

PLATE A-1a

McCLELLAND ENGINEERS

SUMMARY OF LABORATORY TESTING RESULTS – PERMAFROST

SAMPLE NO.	SAMPLE TYPE	DEPTH INTERVAL (metres)	MOIST. CONT. (%)	BULK DENS. (Mg/m³)	GROUND ICE DESCRIPTION (%)	ATTERBERG LIMITS LL (%)	ATTERBERG LIMITS PL (%)	ATTERBERG LIMITS PI (%)	GRAIN SIZE DISTRIBUTION CLAY (%)	GRAIN SIZE DISTRIBUTION SILT (%)	GRAIN SIZE DISTRIBUTION SAND (%)	GRAIN SIZE DISTRIBUTION GRAV. (%)	ORGANIC CONTENT (%)	SOIL DESCRIPTION
21	Q	10.3 - 10.5	33		Not Frozen									
22	Jv	10.5	41		Not Frozen	35	21	14	43	56	1	0		(CL) CLAY
23	Q	11.4 - 11.6			Not Frozen									
24	Jv	11.6	39		Not Frozen									
25	G	12.0			Not Frozen									
26	Q	12.2 - 12.4	30	1.88	Not Frozen									
27	Jv	12.3	57		Not Frozen									
28	Q	12.4 - 12.6	46	1.77	Not Frozen									
29	Jv	12.6	46		Not Frozen								11.1	
30	J	14.2	20		Not Frozen				-	4	96	0		(SP) SAND
31	J	14.9	20		Not Frozen				-	4	96	0		(SP) SAND
39	J	18.0	21		Not Frozen				-	3	97	0		(SP) SAND
40	J	19.7	24		Not Frozen				-	4	96	0		(SP) SAND
41	J	21.0	24		Not Frozen				-	3	97	0		(SP) SAND
42	Q	22.4 - 22.6	32	1.92	Not Frozen									
43	Jv	22.6	39		Not Frozen									
44	JT	23.9	35		Not Frozen									
45	Jv	24.1	32		Not Frozen									
46	Q	25.6 - 25.8	36	1.88	Not Frozen									
47	Jv	25.8	32		Not Frozen	41	28	13						(ML) SILT
48	Q	26.9 - 27.1	34	1.93	Not Frozen									
49	JT	27.1	36		Not Frozen									
50	Jv	27.3	35		Not Frozen									

PROJECT NUMBER 101-2581 SITE NUMBER KENALOOAK BOREHOLE B6

1

2

4a

$I_L = \frac{-7}{14} =$

$I_L =$

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PLATE A-1b

SUMMARY OF LABORATORY TESTING RESULTS – PERMAFROST

	SAMPLE		DEPTH INTERVAL	MOIST. CONT.	BULK DENS.	GROUND ICE DESCRIPTION	ATTERBERG LIMITS			GRAIN SIZE DISTRIBUTION				ORGANIC CONTENT	SOIL DESCRIPTION	
	NO.	TYPE	(metres)	(%)	(Mg/m³)	(%)	LL (%)	PL (%)	PI (%)	CLAY (%)	SILT (%)	SAND (%)	GRAV. (%)	(%)		
	51	Q	28.4 - 28.6	31	1.95	Not Frozen										
	52	Jv	28.6	26		Not Frozen										
	53	JT	30.0	24		Not Frozen										
	54	Jv	30.1	32		Not Frozen	48	30	18	~~80~~ 60	~~20~~ 70	0	0	11.4	(ML) SILT	Ic = ??
4a	55	Q	31.5 - 31.7			Not Frozen										
	56	Jv	31.7	33		Not Frozen	51	32	19						(MH) SILT	Ic =
	57	G	33.0			Not Frozen										
	58	Q	33.0 - 33.2	28	1.94	Not Frozen										
	59	Jv	33.2	30		Not Frozen										
	60	PF	34.9	26	1.93	Vx 10										
4b	61	J	35.0	28		Vx 10	49	29	20						(ML) SILT	Ic =
	62	PF	36.2 - 36.5	36	1.82	Vr 5-10	56	26	30						(CH) CLAY	
	63	PF	37.8			Vx										
	64	PF	39.3 - 39.6	28	1.96	Vr	48	23	25						(CH) CLAY	
	65	PF	40.8			Vx - Vr										
	66	PF	43.9 - 44.3	35	1.79	Vs-Vr 10	57	24	33	65	35	0	0		(CH) CLAY	
	67	PF	45.4 - 45.7	25	2.03	Vs-5										
	68	J	48.4	34		Nbn	Nonplastic			9	89	2	0		(ML) SILT	
	69	J	51.5	45		Nbn	Nonplastic			13	85	2	0		(ML) SILT	
	70	J	53.5	33		Nbn										
	71	J	54.9	29		Nbn	38	22	16						(CL) CLAY	
	72	G	57.0			Nbn										
	73	J	57.3	22		Nbn										
	74	PF	57.9 - 58.1	25	2.00	Nbn										
	75	J	60.0	25		Nbn	30	22	8	35	63	2	0		(CL) CLAY	

PROJECT NUMBER 101-2581 SITE NUMBER KENALOOAK BOREHOLE B6

EBA Engineering Consultants ltd.

SUMMARY OF LABORATORY TESTING RESULTS – PERMAFROST

SAMPLE NO.	SAMPLE TYPE	DEPTH INTERVAL (metres)	MOIST. CONT. (%)	BULK DENS. (Mg/m³)	GROUND ICE DESCRIPTION (%)	ATTERBERG LIMITS LL (%)	ATTERBERG LIMITS PL (%)	ATTERBERG LIMITS PI (%)	GRAIN SIZE DISTRIBUTION CLAY (%)	GRAIN SIZE DISTRIBUTION SILT (%)	GRAIN SIZE DISTRIBUTION SAND (%)	GRAIN SIZE DISTRIBUTION GRAV (%)	ORGANIC CONTENT (%)	SOIL DESCRIPTION
76	PF	61.0	26	1.87	Nbn									
77	PF	63.7 - 64.1			Nf	27	17	10						(CL) CLAY
78	PF	66.7	31	1.67	Nf				12	84	4	0		(ML) SILT
79	PF	69.8 - 70.2	26	1.98	Nf	Nonplastic								(ML) SILT
80	PF	72.8 - 73.1	36	1.78	Nbn									
81	PF	75.9 - 76.3	28	1.98	Nbn				15	77	8	0		(ML) SILT
82	J	81.7	35		Nbn	Nonplastic			13	84	3	0	7.9	(ML) SILT
83	PF	82.3 - 82.5	29	1.87	Nbn	28	16	12	36	64	0	0		(CL) CLAY
84	PF	85.2 - 85.4	32	1.86	Nbn									
85	J	85.3	28		Nbn	25	Nonplastic							(CL) CLAY
86	J	90.5	23		Nbn				6	27	67	0		(SM) SAND
87	PF	91.3 - 91.6	31	1.73	Nbn									
88	J	91.4	27		Nbn	Nonplastic								(ML) SILT
89	J	94.0	24		Nbn									
90	PF	94.5	30	1.81	Nbn	Nonplastic			4	89	7	0		(ML) SILT
91	J	97.2	41		Nbn									
92	PF	97.5			Nbn									
93	PF	100.1			Nbn									
94	PF	100.6			Nbn									
95	Q	102.8 - 103.0	22	2.08	Nbn									
96	JT	103.0	27		Nbn	33	20	13	40	60	0	0		(CL) CLAY
97	PF	103.6 - 103.7	24	1.99	Nbn									
98	PF	106.1	28	1.96	Nbn	38	19	19						(CL) CLAY
99	Q	106.4 - 106.6	25	2.05	Nbn									
100	JT	106.7			Nbn									

4b
sand

PROJECT NUMBER 101-2581 SITE NUMBER KENALOOAK BOREHOLE B6

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SUMMARY OF LABORATORY TESTING RESULTS – PERMAFROST

SAMPLE NO.	SAMPLE TYPE	DEPTH INTERVAL (metres)	MOIST. CONT. (%)	BULK DENS. (Mg/m^3)	GROUND ICE DESCRIPTION (%)	ATTERBERG LIMITS LL (%)	ATTERBERG LIMITS PL (%)	ATTERBERG LIMITS PI (%)	GRAIN SIZE DISTRIBUTION CLAY (%)	GRAIN SIZE DISTRIBUTION SILT (%)	GRAIN SIZE DISTRIBUTION SAND (%)	GRAIN SIZE DISTRIBUTION GRAV. (%)	ORGANIC CONTENT (%)	SOIL DESCRIPTION
101	J	109.6	25		Nbn	35	24	11						(CL) CLAY
102	PF	109.7 - 109.8	25		Nbn	38	20	18						(CL) CLAY
103	J	112.6	25		Nbn									
104	J	112.8	31		Nbn	37	18	19						(CL) CLAY
105	PF	115.7 - 115.8			Nbn				33	67	0	0		(CL) CLAY
106	J	115.8	27		Nbn	30	19	11						(CL) CLAY
107	J	118.6	27		Nbn									
108	PF	118.9 - 119.1	25	1.74	Vs 5	41	20	21						(CL) CLAY
109	Q	121.1 - 121.3	21	2.15	Nbn	40	21	19	45	55	0	0		(CL) CLAY
110	JT	121.3	21		Nbn									
111	PF	121.9	22	2.04	Nbn				48	52	0	0		(CL) CLAY

PROJECT NUMBER 101-2581 SITE NUMBER KENALOOAK BOREHOLE B6

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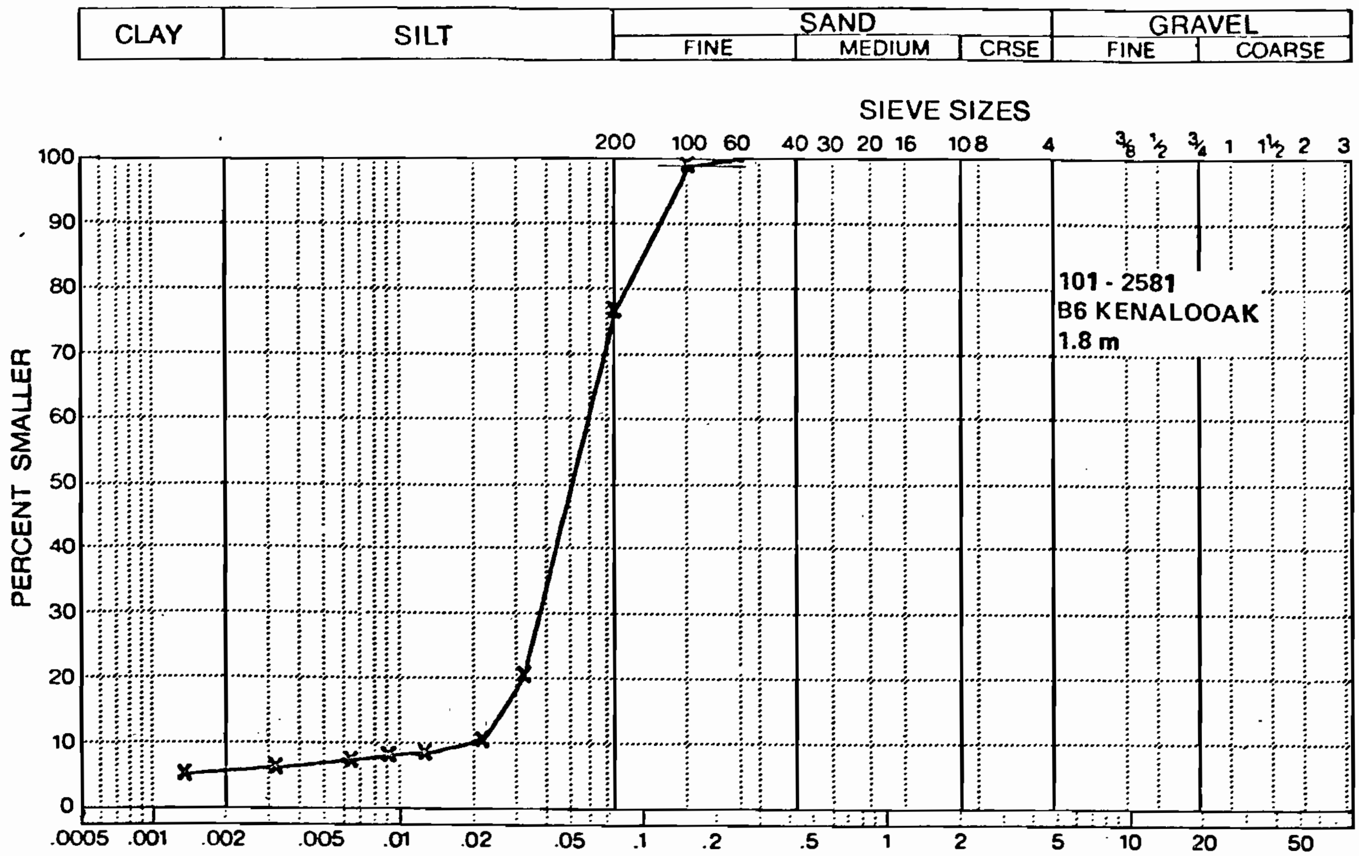
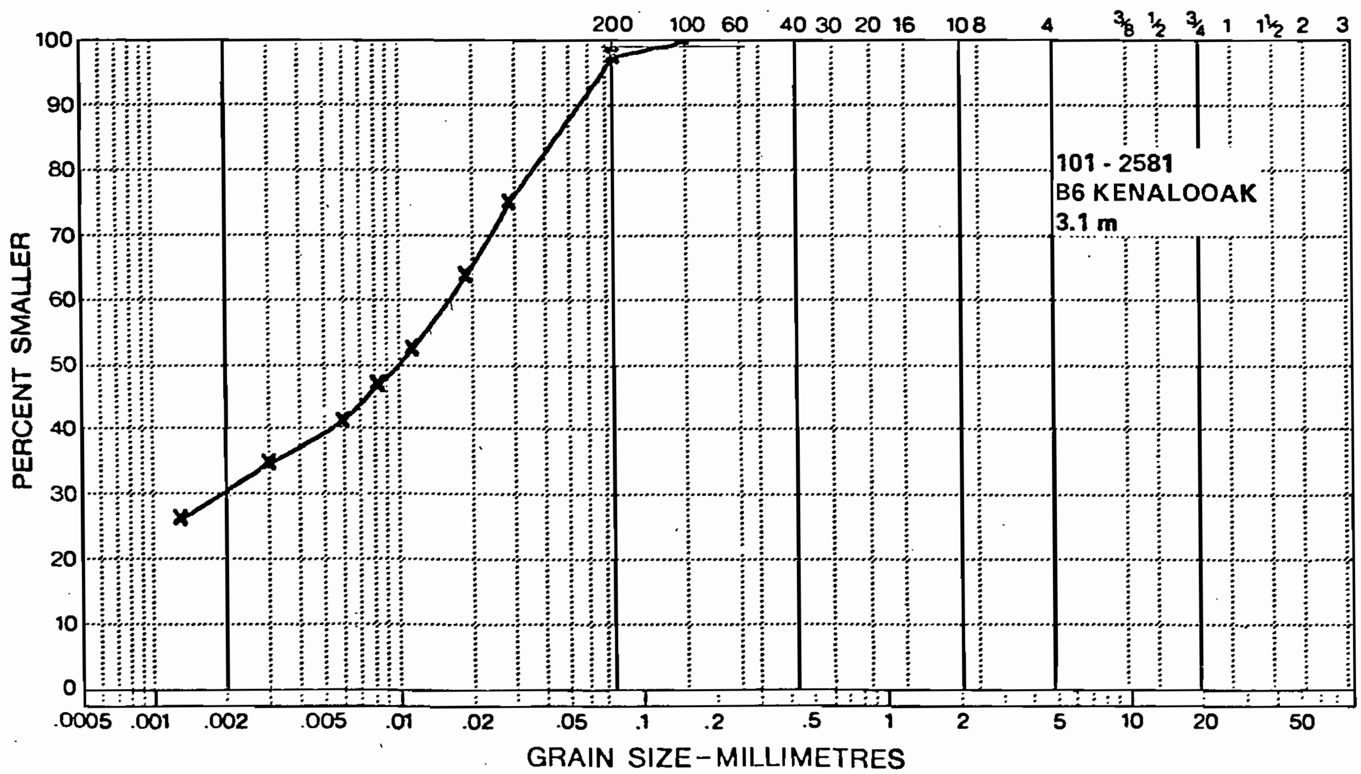
PLATE A-1e





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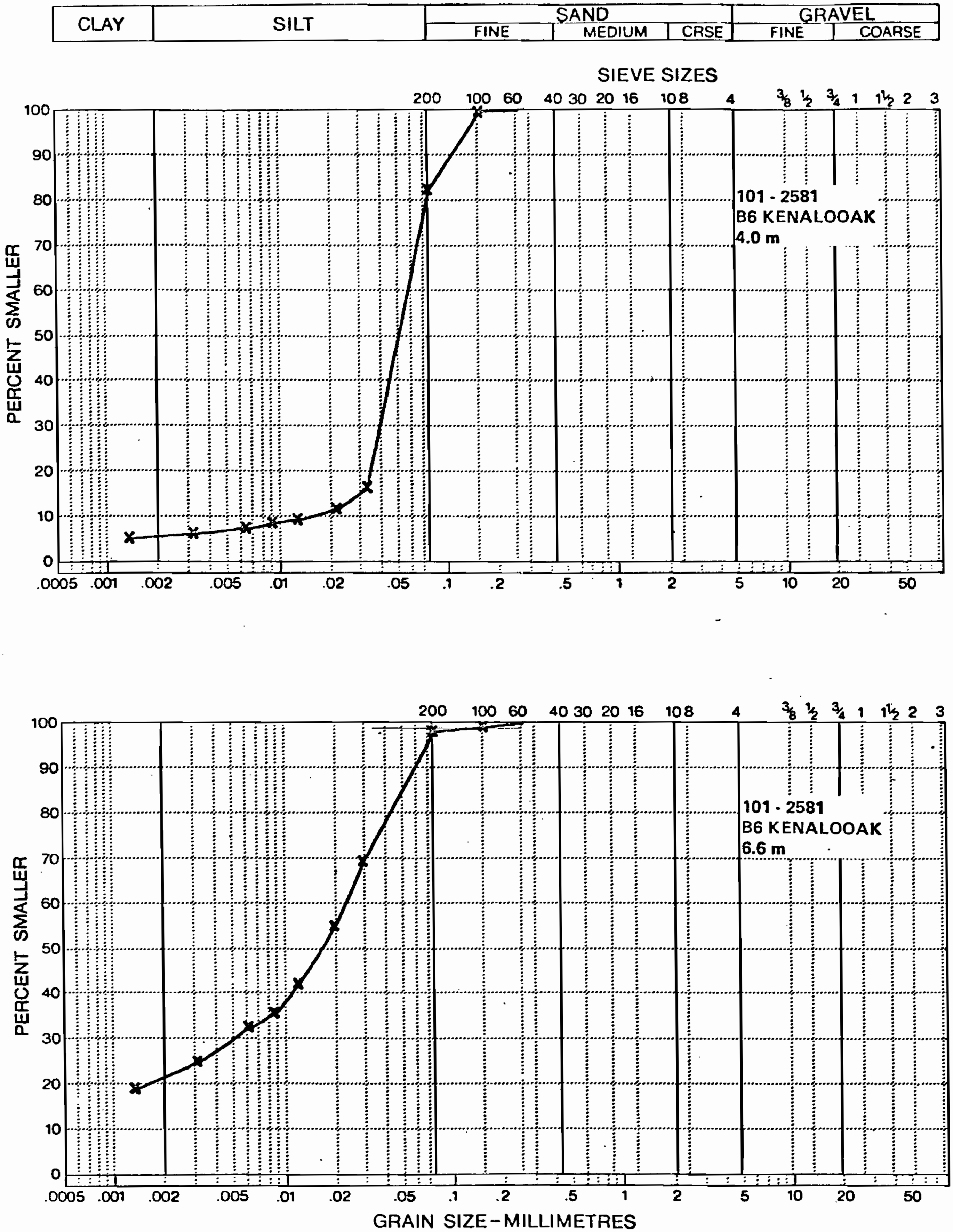
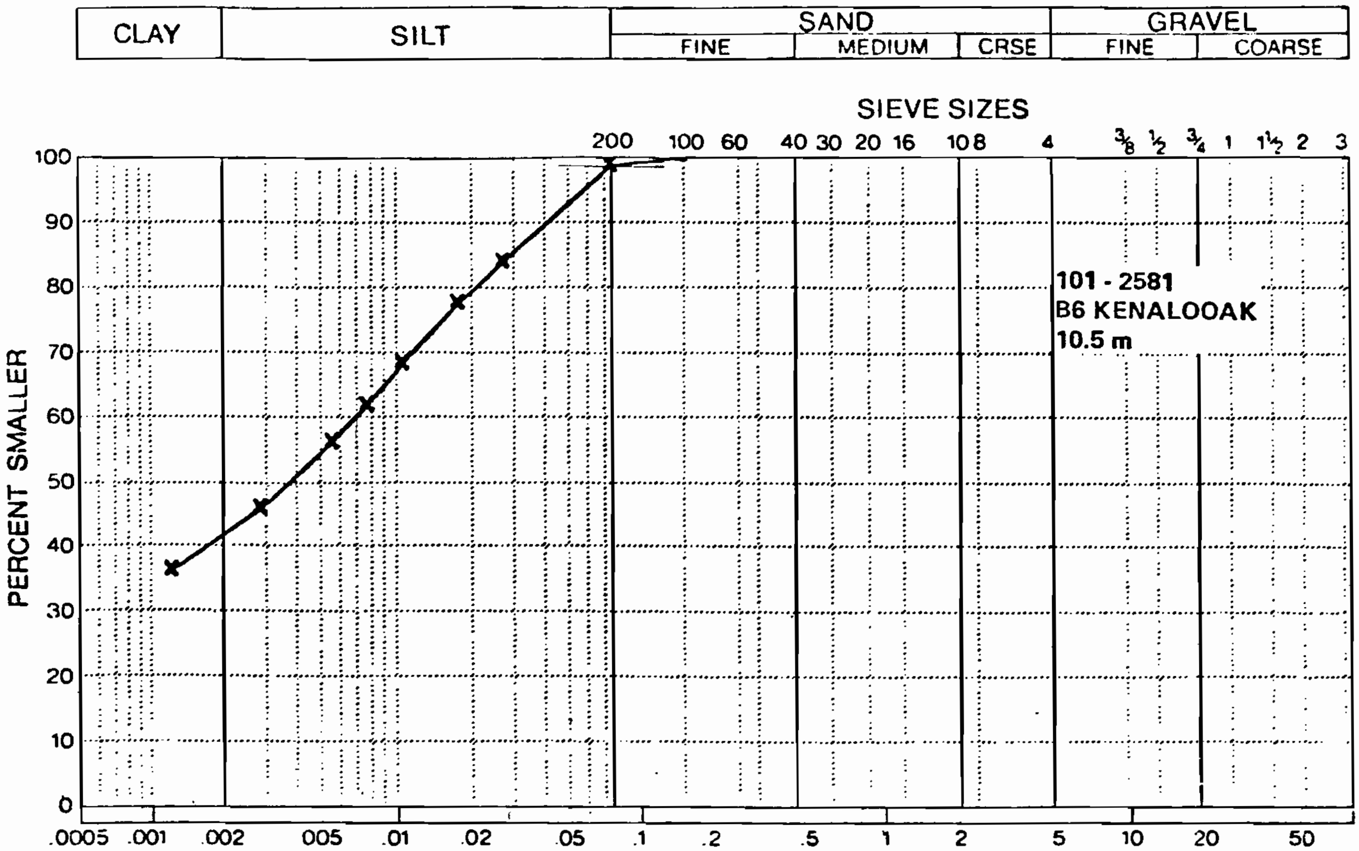
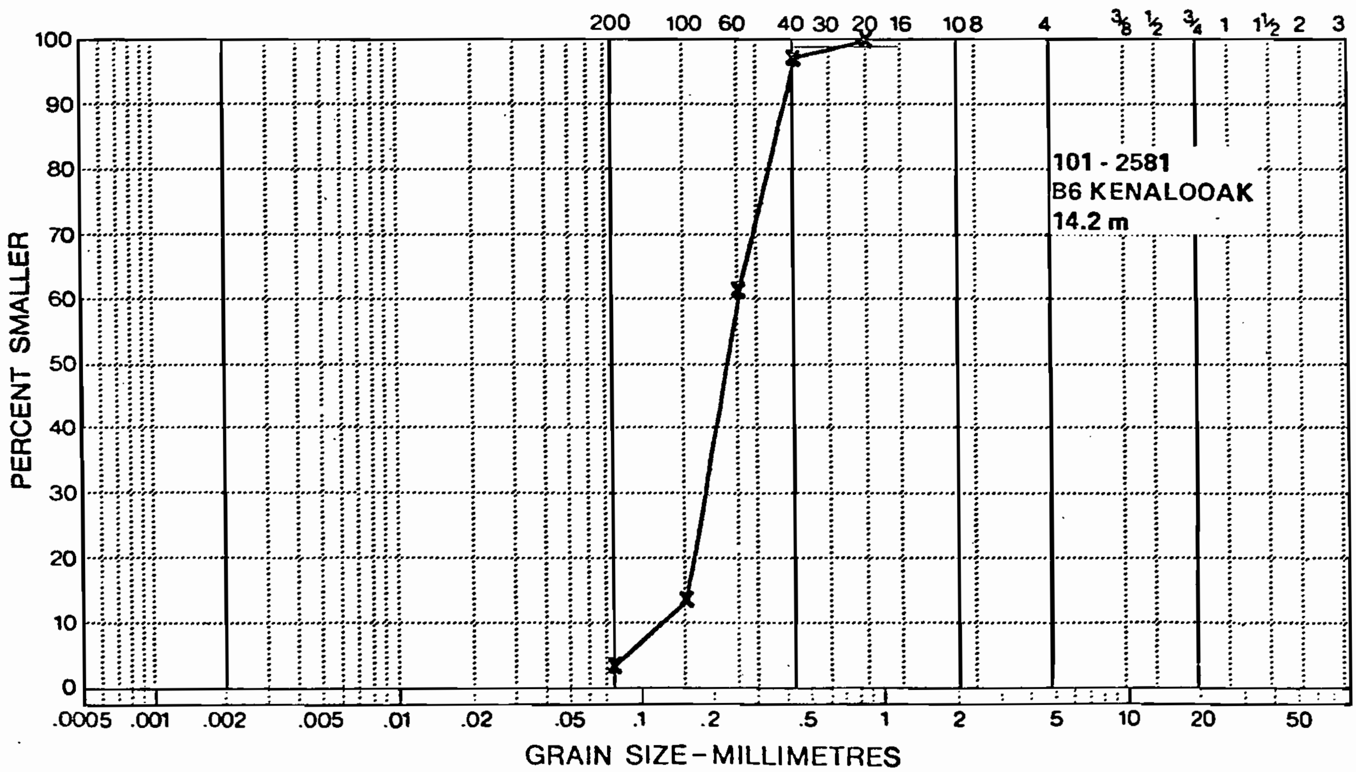


EBA Engineering Consultants Ltd.





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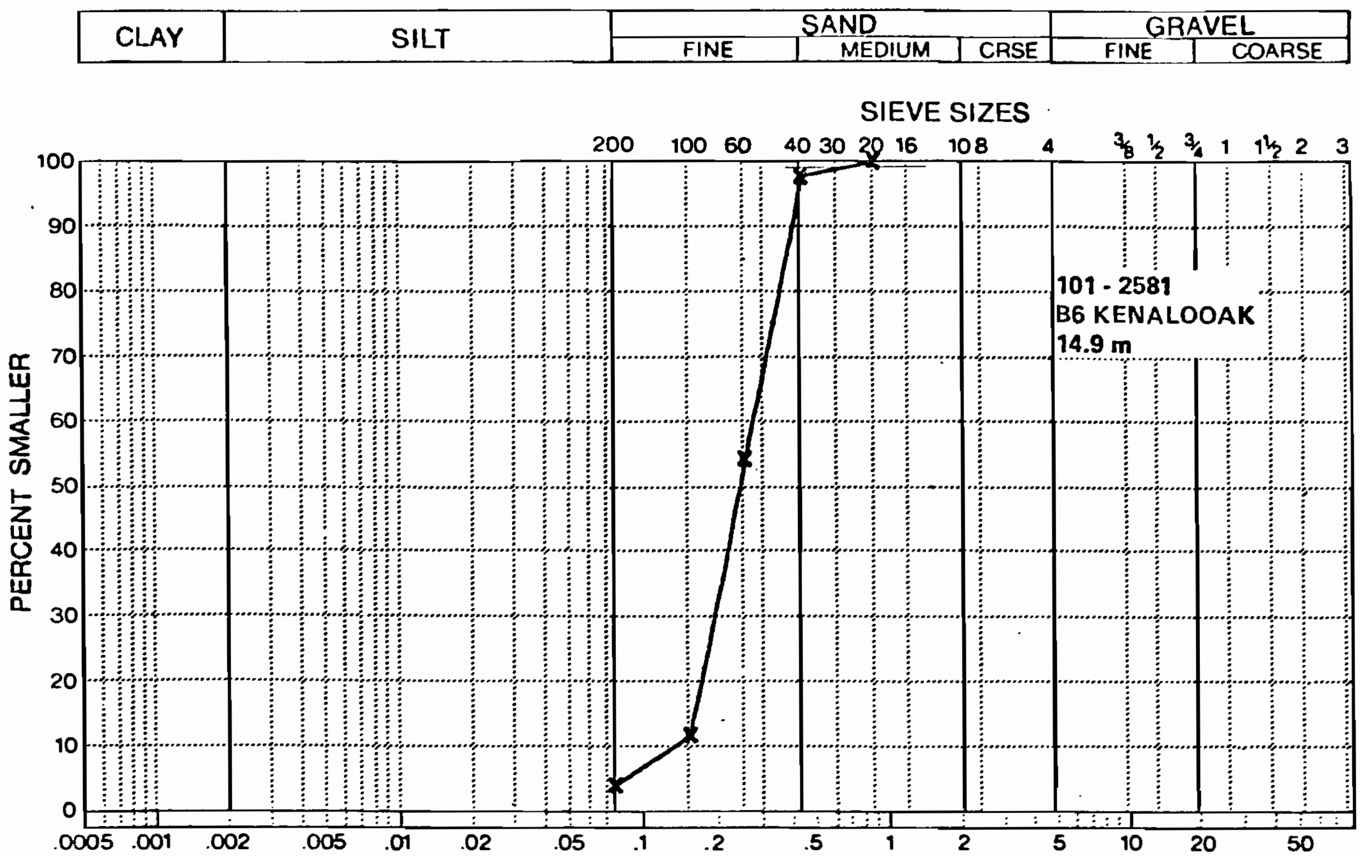

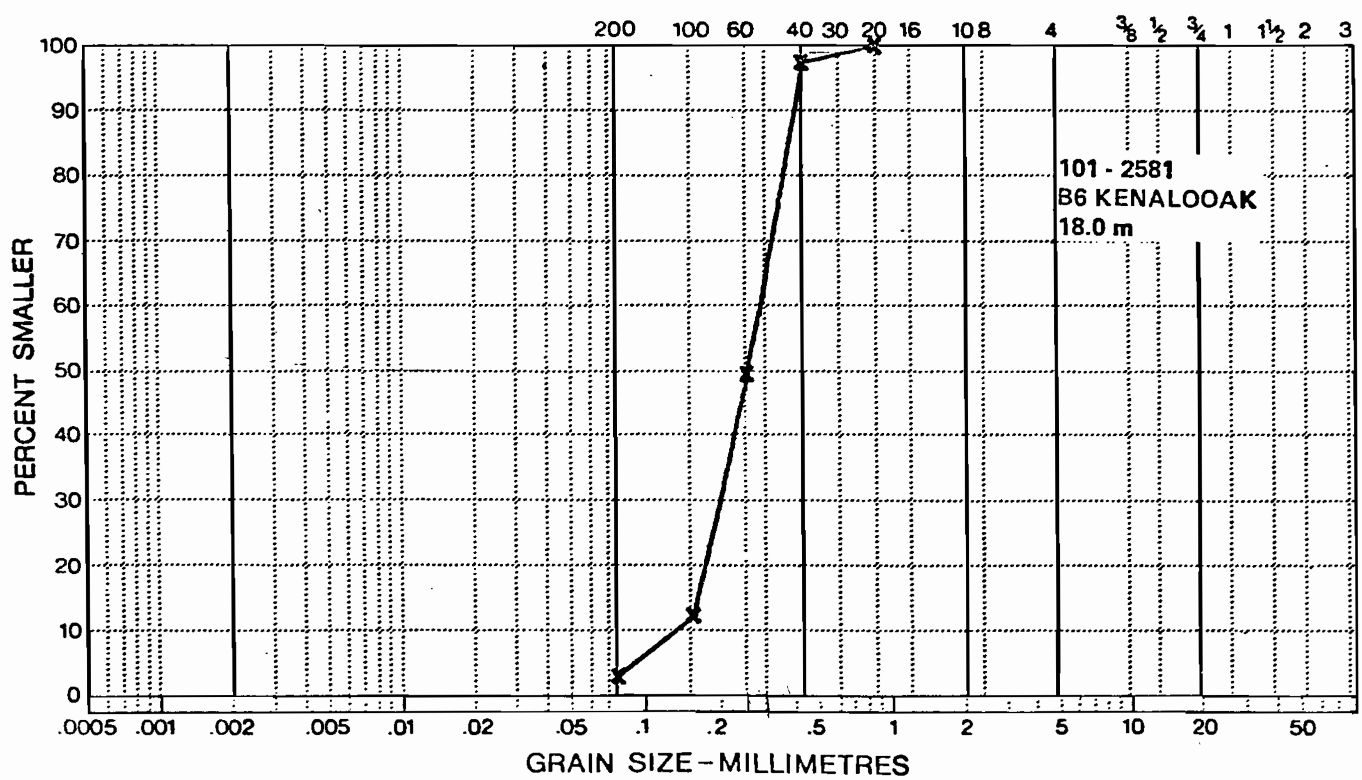

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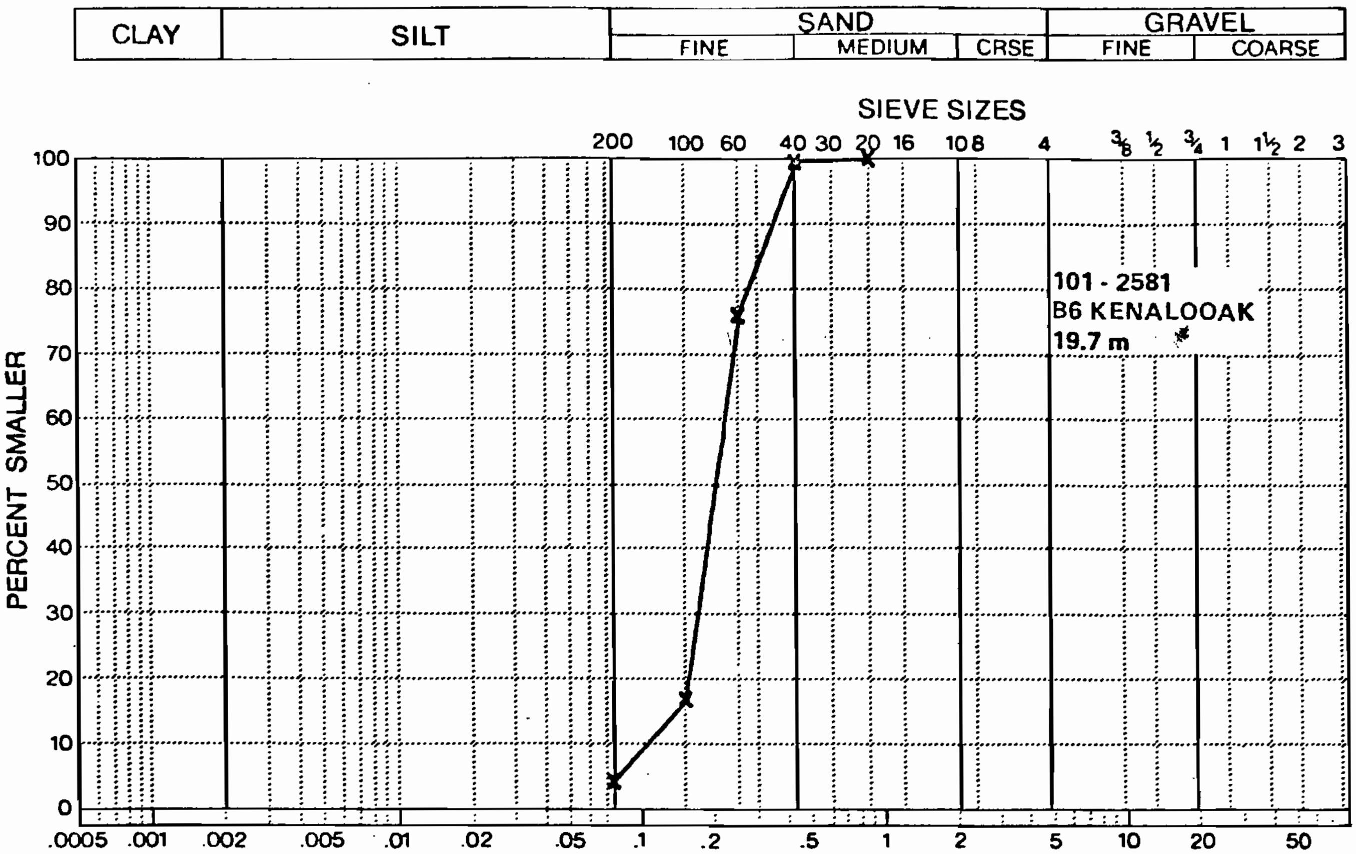

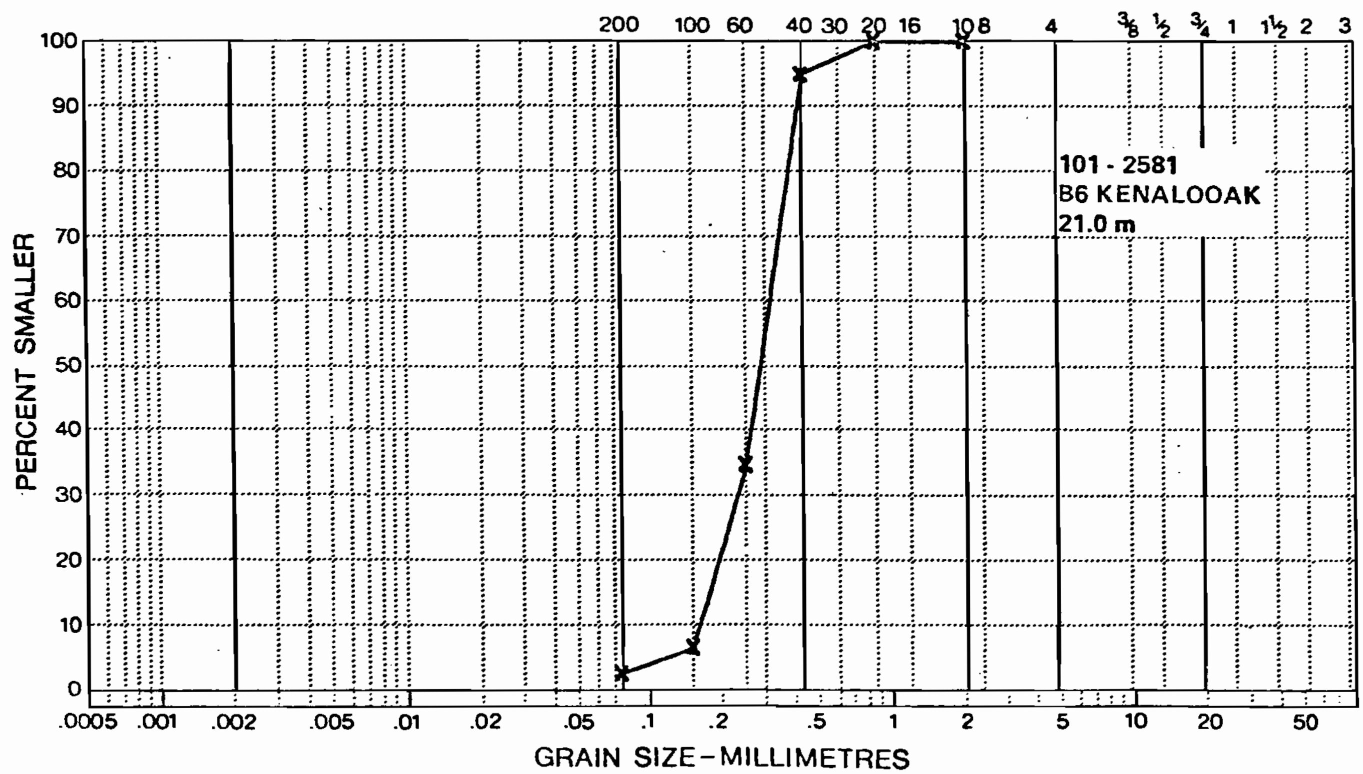

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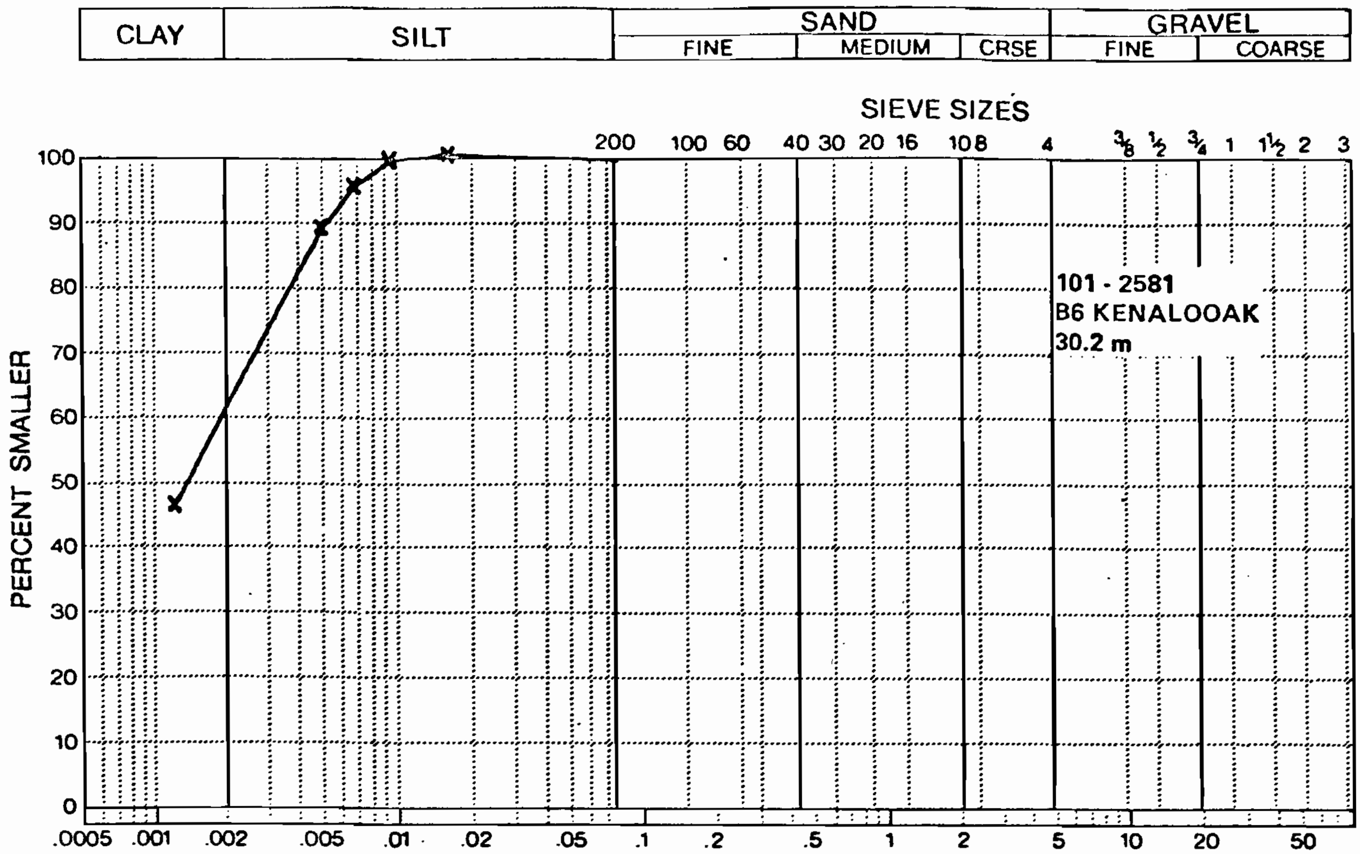
PLATE A-6



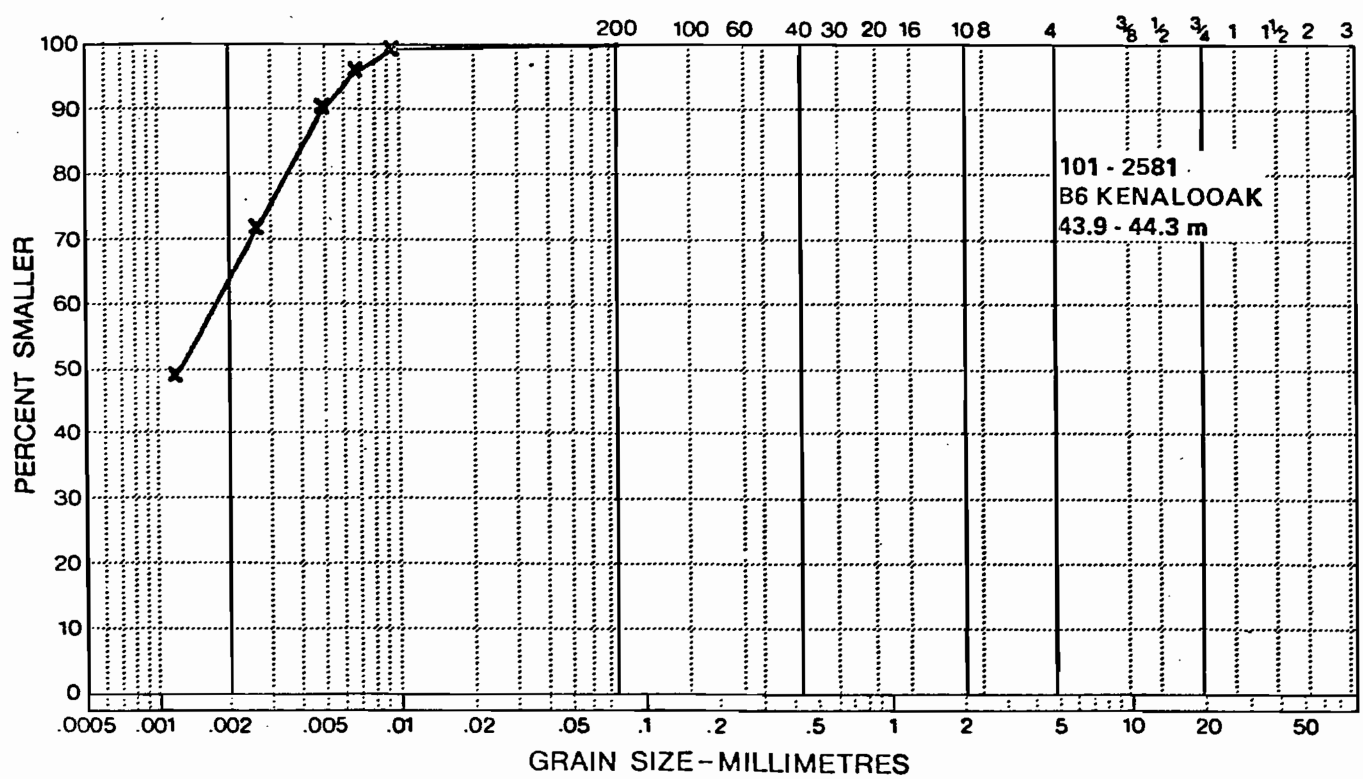


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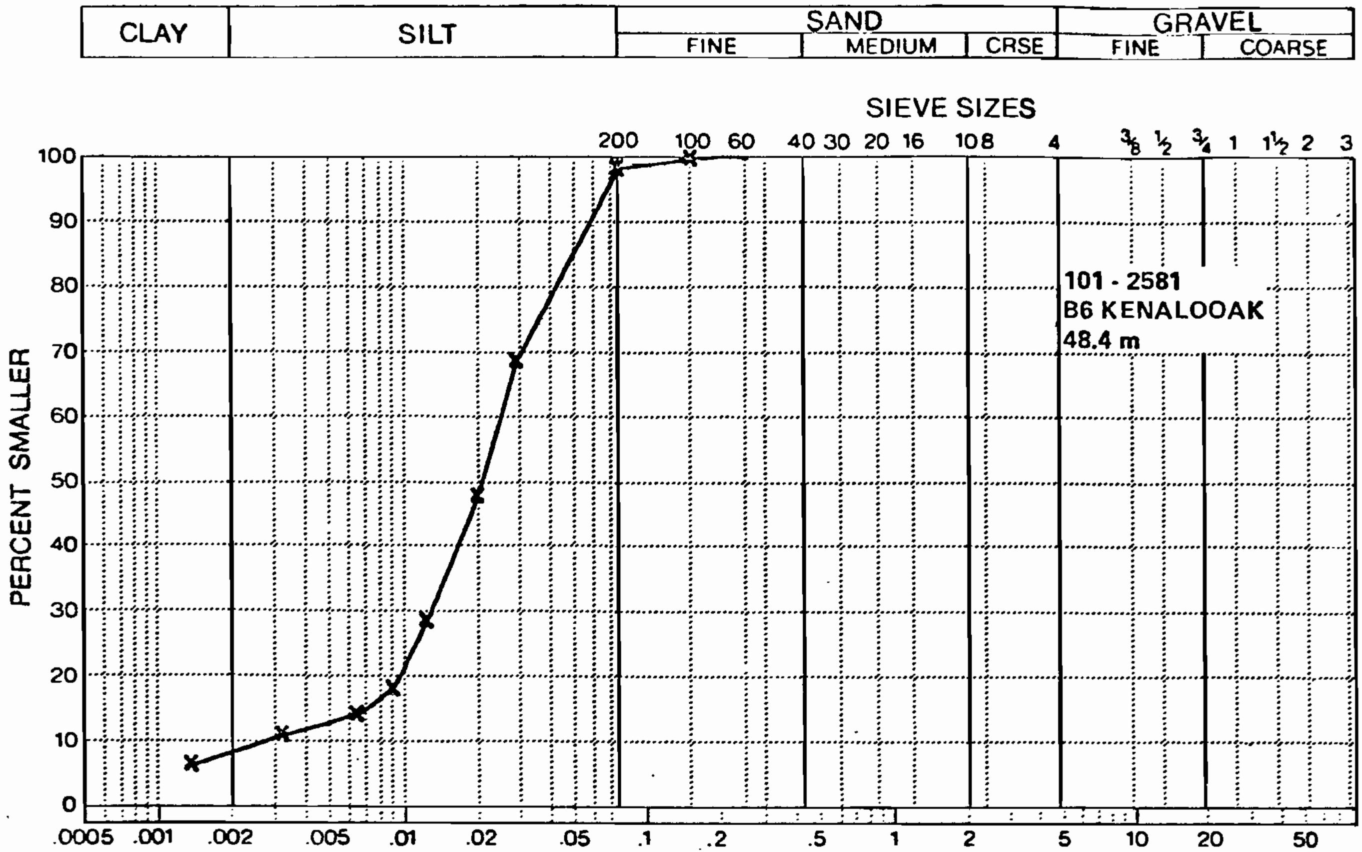
PLATE A-7



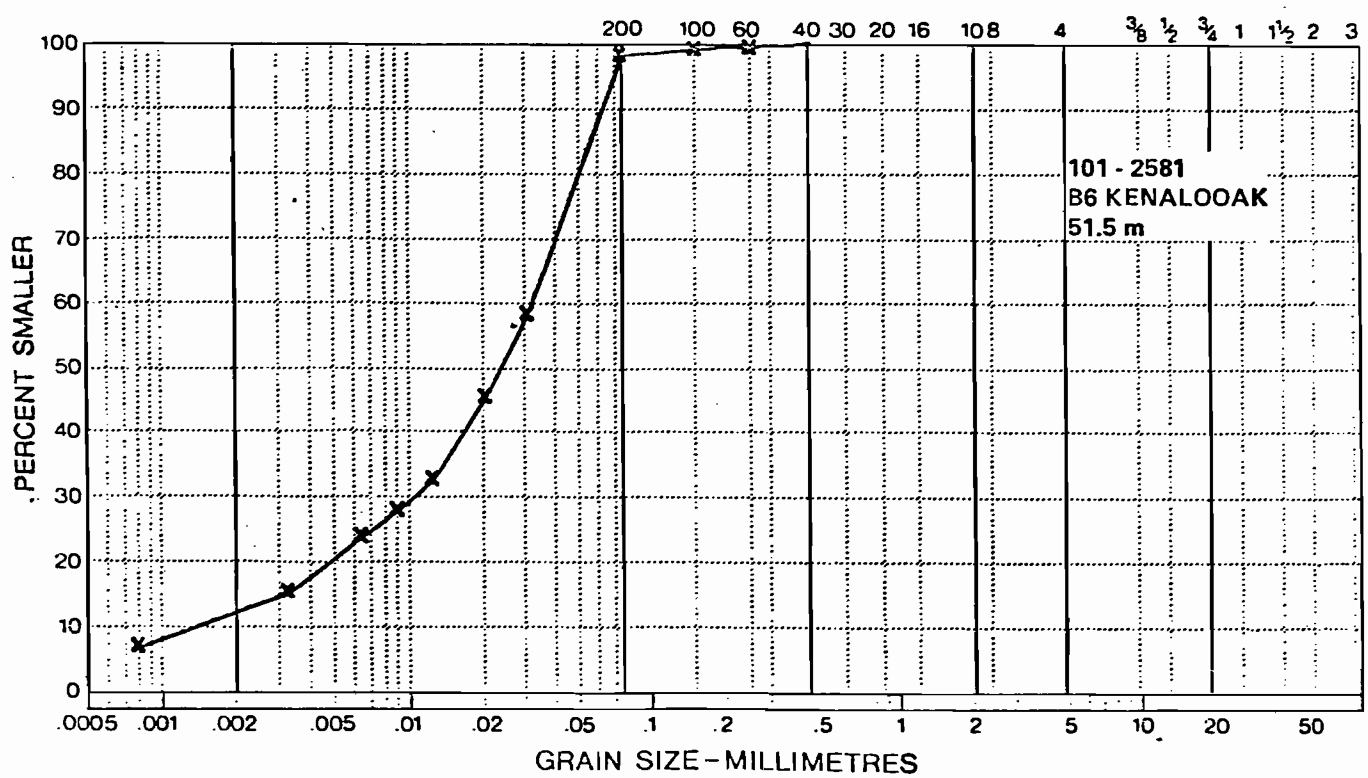


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PLATE A-8

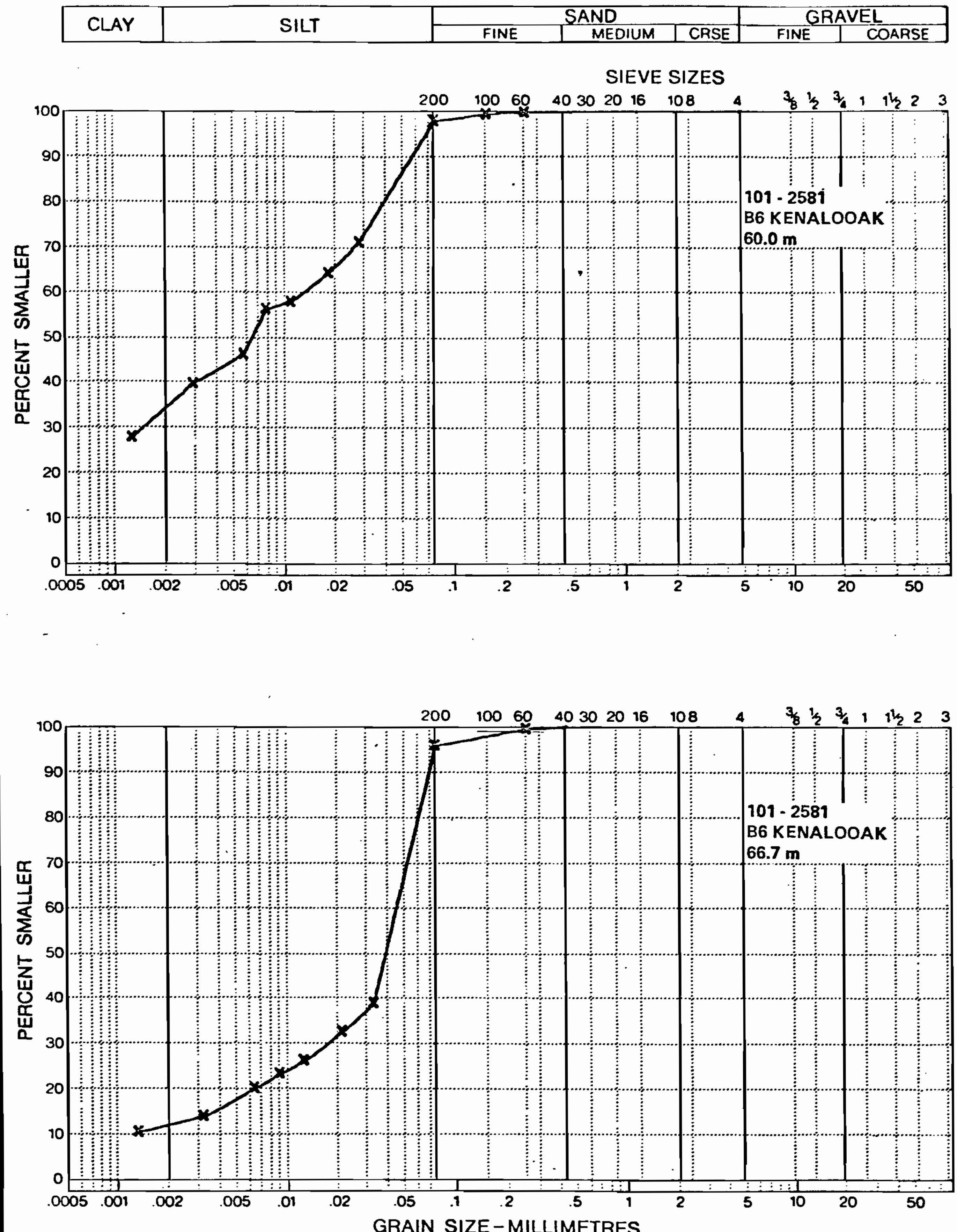

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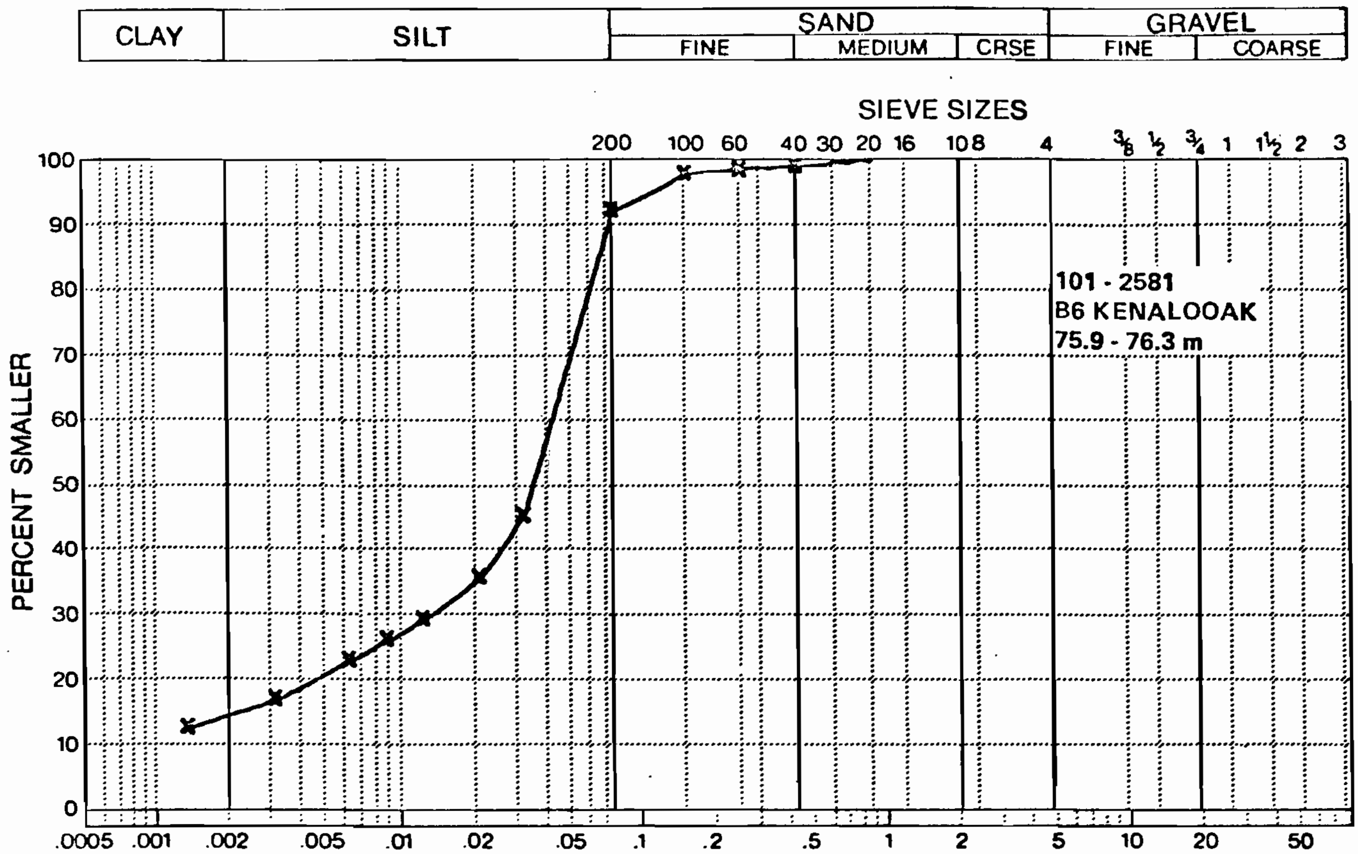
PLATE A-9



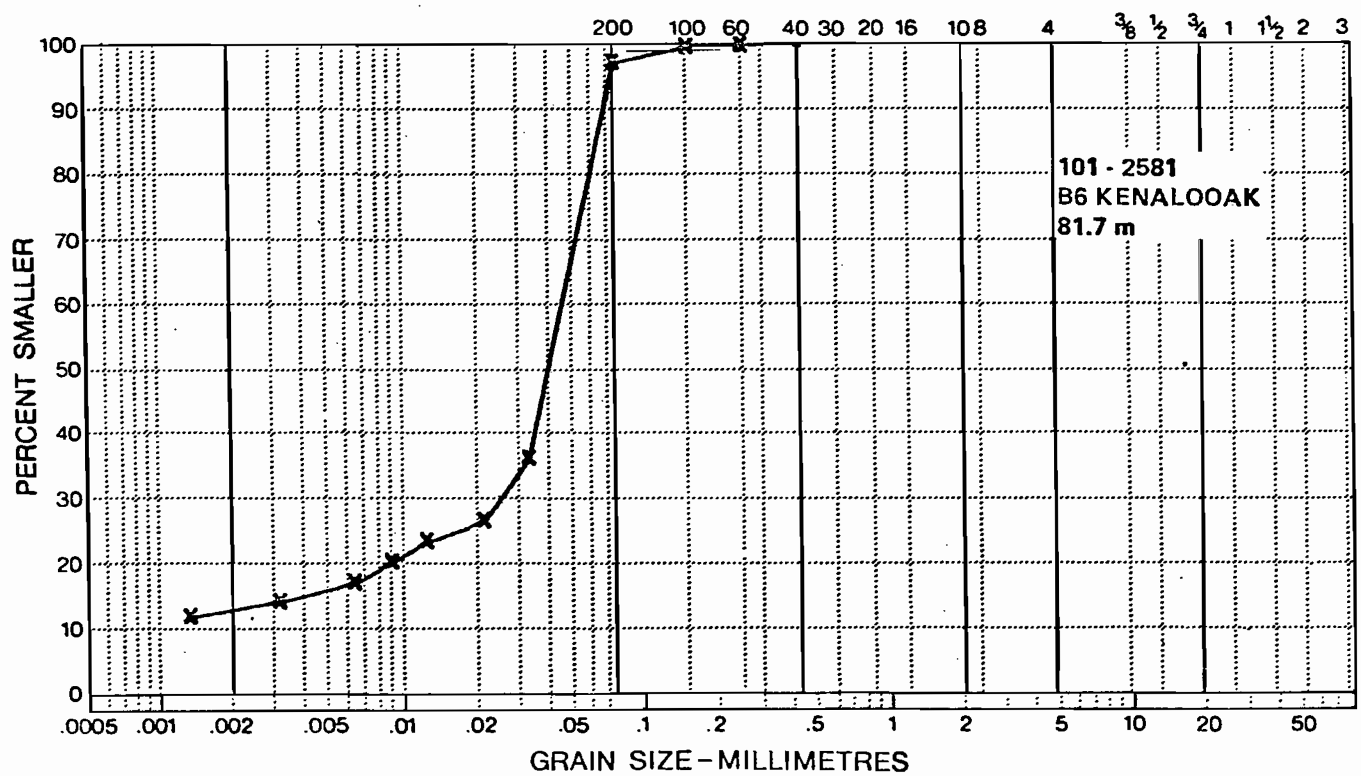


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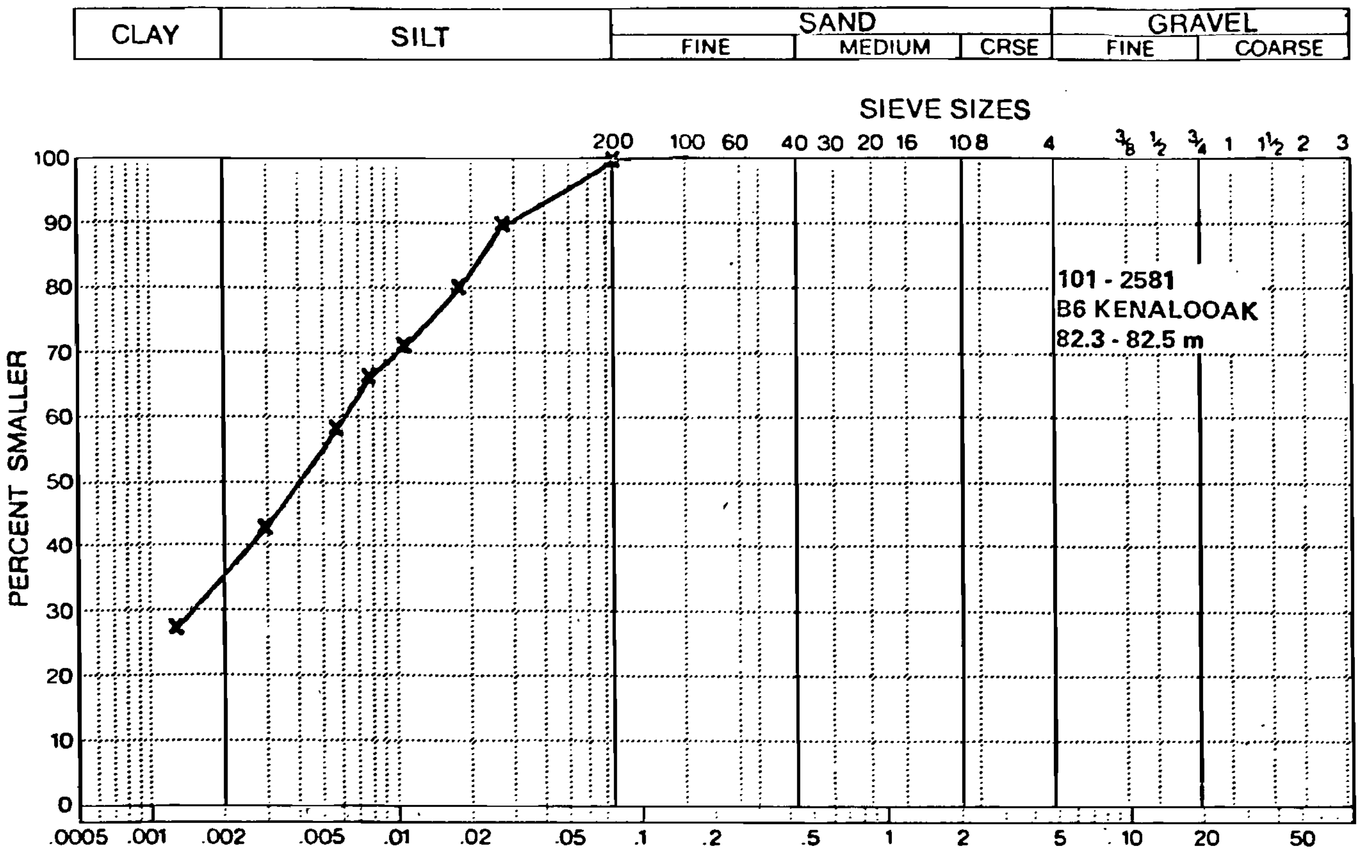
PLATE A-10



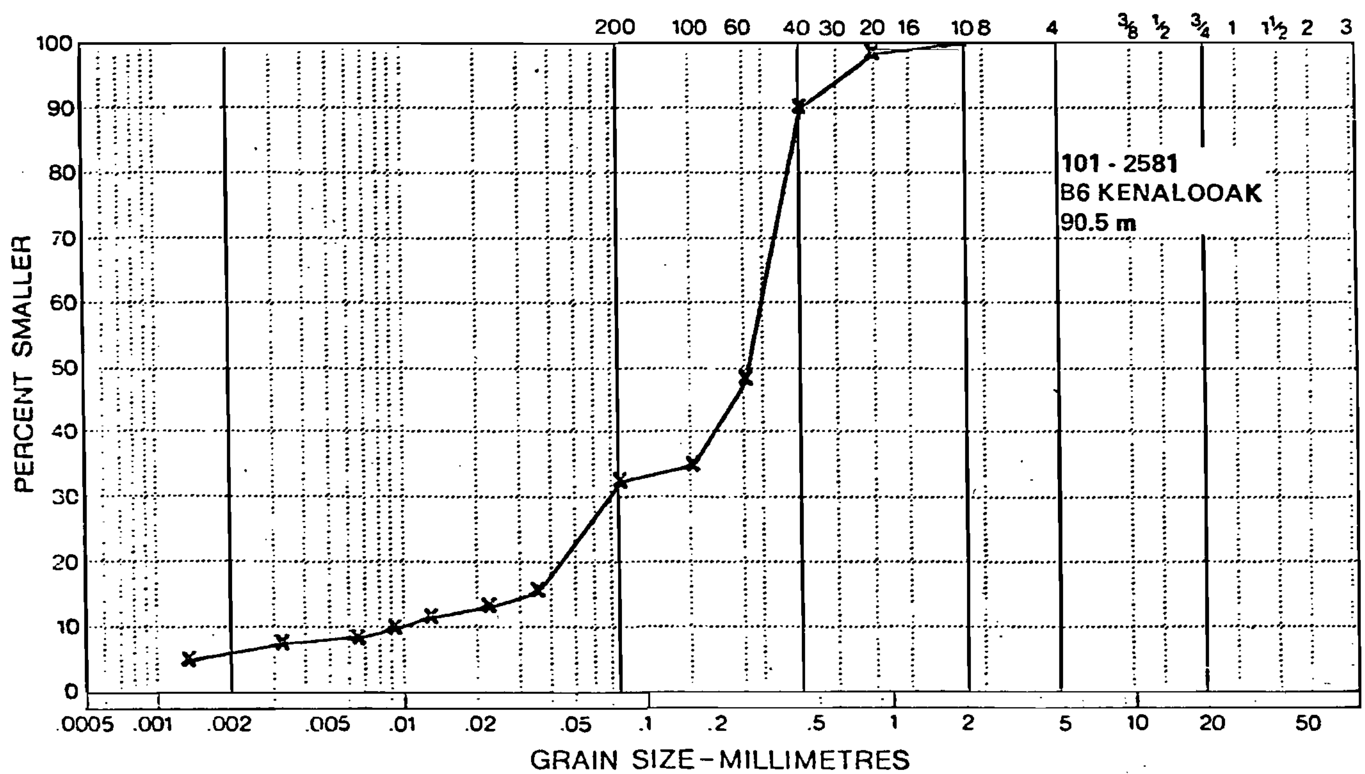


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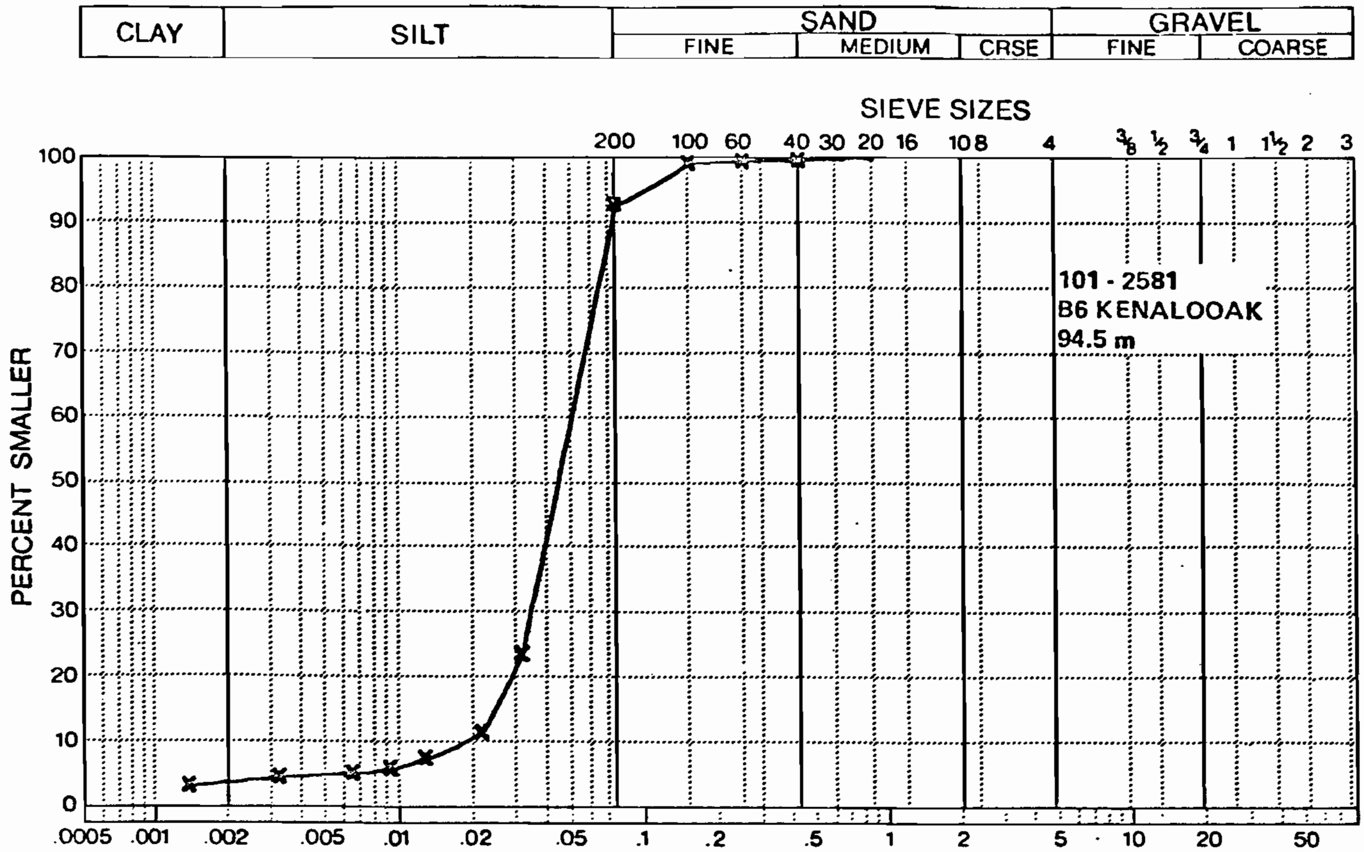
PLATE A-11



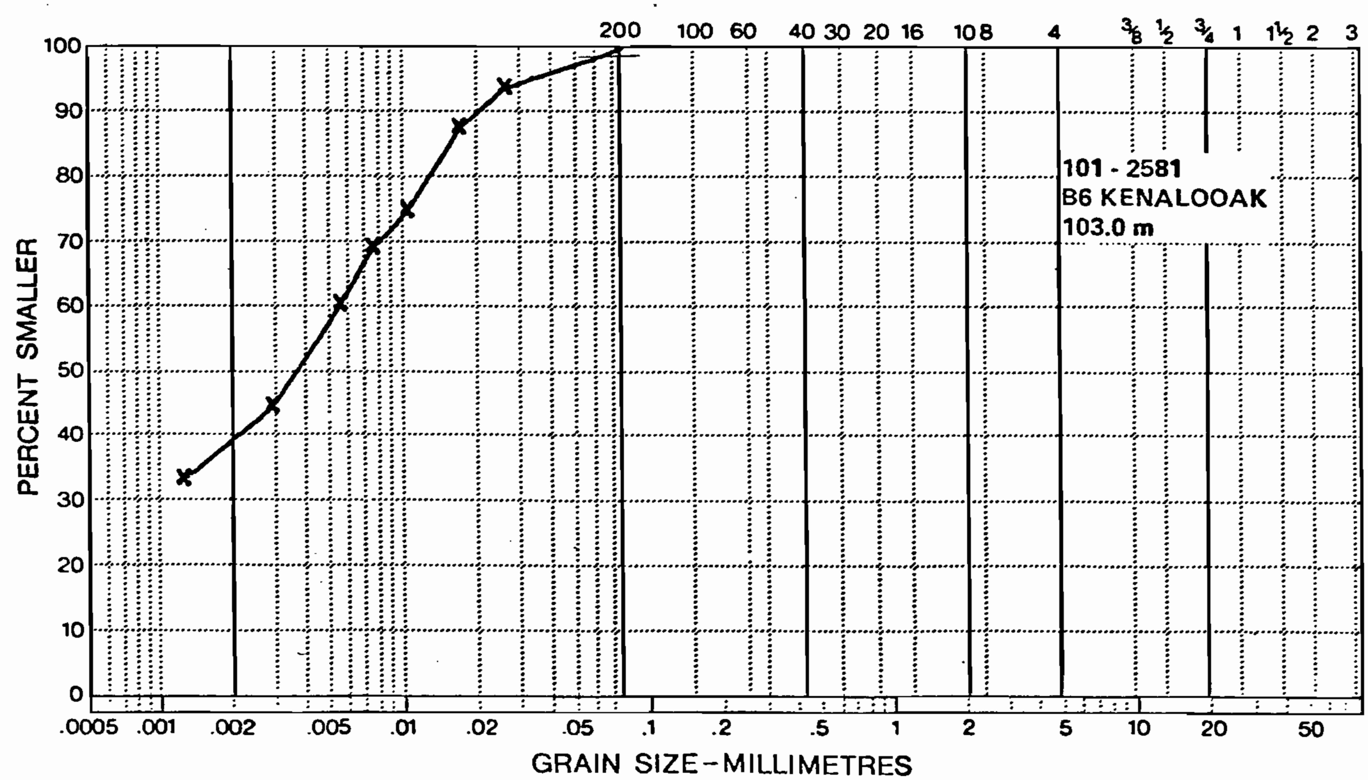


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PLATE A-12

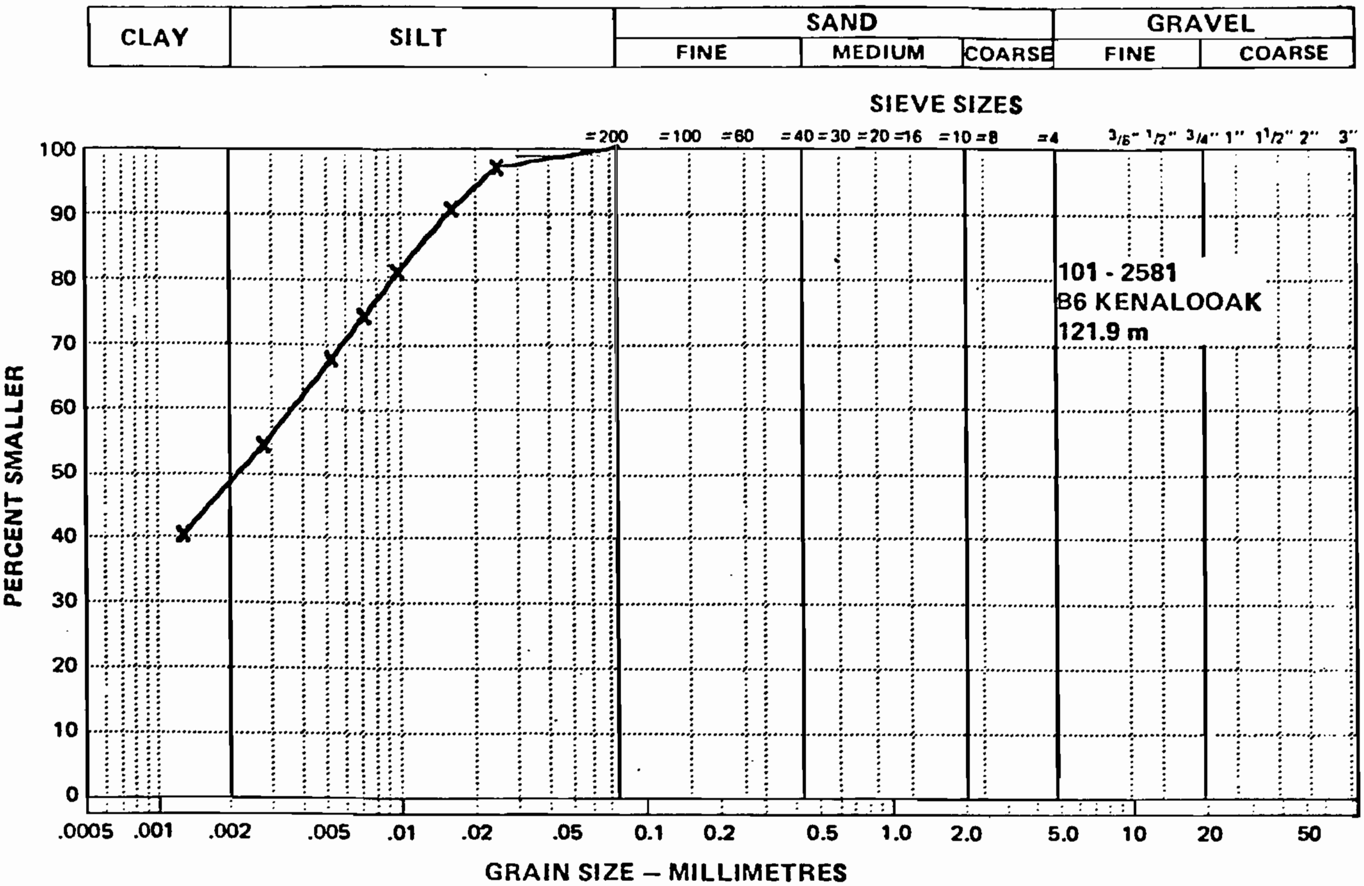

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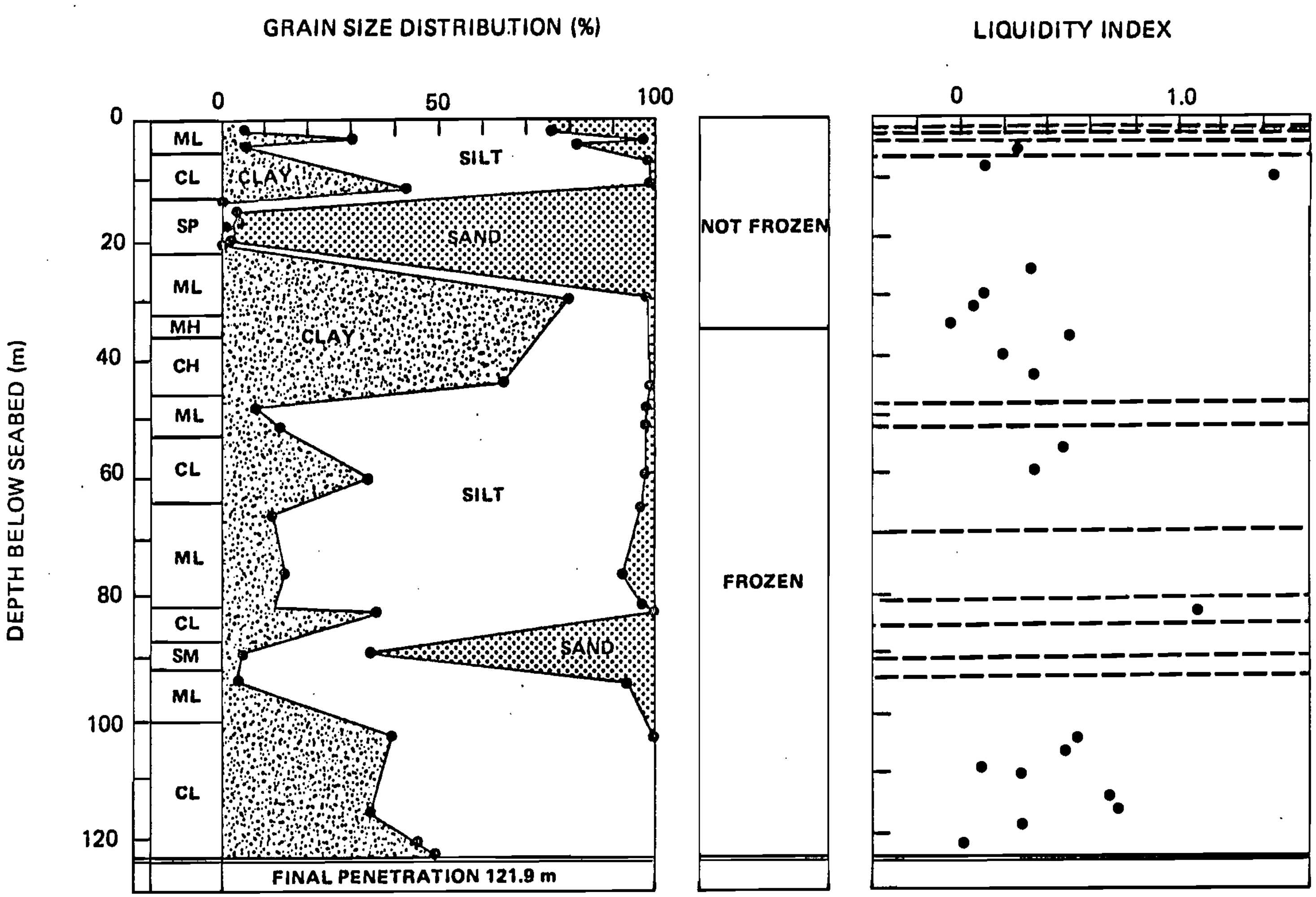
PLATE A-14



Note: FROZEN *indicates that ice bonding was distinguished.*

— — — — — Denotes nonplastic

LIQUIDITY INDEX PROFILE

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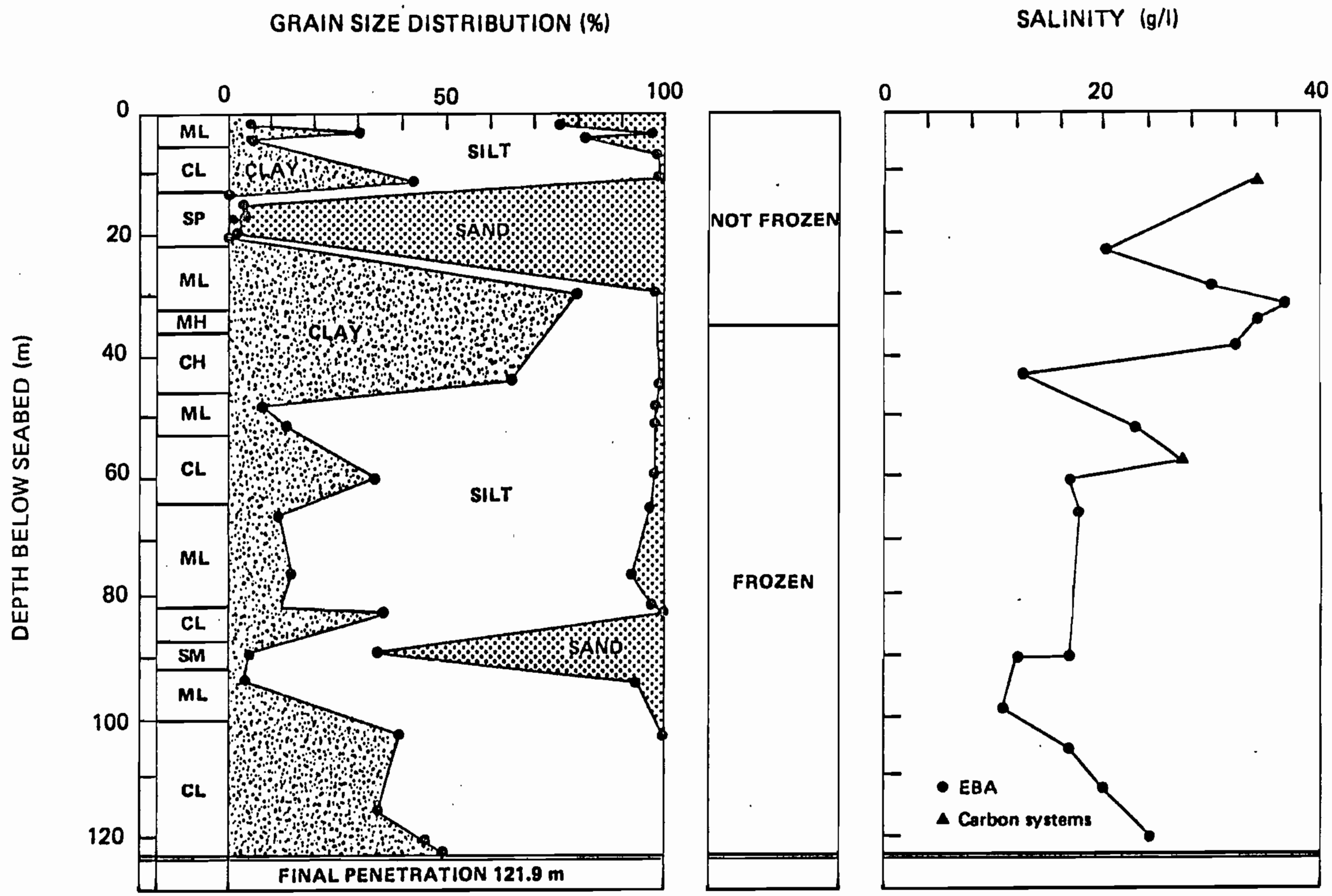

Note: FROZEN *indicates that ice bonding was distinguished.*

PORE WATER SALINITY PROFILE

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PLATE A-16

McCLELLAND ENGINEERS

SALINITY TEST RESULTS

DEPTH (metres)	DEPTH (feet)	SALINITY (ppt)	CLAY FRACTION (%)
12.0	39.0	34 *	34
22.6	74.0	20	2
28.7	94.0	30	45
31.5	103.5	37	80
34.9	114.5	34	78
39.3	129.0	32	73
43.9	144.0	13	65
51.5	169.0	23	12
57.0	187.0	27 *	32
61.0	200.0	17	35
66.8	219.0	18	12
91.3	299.5	17	5
91.3	299.5	12	5
100.1	328.5	11	21
106.7	350.0	17	40 **
112.8	370.0	20	35 **
121.9	400.0	25	35 **

NOTE: *Test performed by Carbon Systems, Inc.
**Clay fraction interpolated

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SPECIFIC GRAVITY TEST RESULTS

SAMPLE NUMBER	DEPTH (m)	SPECIFIC GRAVITY OF SOIL PARTICLES
62	36.3	2.79
67	45.4	2.78
79	69.8	2.77

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KENALOOAK

UNCONSOLIDATED - UNDRAINED

TRIAXIAL TESTS

SAMPLE NUMBER	DEPTH (metres)	USC	MOISTURE CONTENT (%)	INITIAL BULK DENSITY (Mg/m^3)	CONFINING PRESSURE (kPa)	AXIAL STRAIN AT FAILURE (%)	UNDRAINED SHEAR STRENGTH (JACKETED. TRIAXIAL) (kPa)	REMARKS
S-26	12.2	CL	32.5	1.88	400	13	29	silt/clay contact
S-42	22.4	ML	32.9	1.93	500	15	76	
S-48	27.0	ML	34.2	1.93	530	24*	55	
S-58	33.1	MH	28.4	1.94	580	12	183	
S-62	36.3	CH	28.4	1.93	610	12*	80	
S-64	39.3	CH	26.2	1.97	640	14	216	laminated
S-67	45.4	CH	24.8	1.96	690	20*	114	
S-80	72.8	ML	35.8	1.78	920	15	125	laminated
S-84	85.0	CL	30.7	1.86	1020	12	25	
S-95	102.9	CL	21.7	2.08	1170	15	173	laminated
S-97	103.6	CL	21.7	2.04	1180	20	123	laminated
S-99	106.4	CL	25.4	2.05	1200	20	84	laminated
S-109	121.2	CL	19.5	2.15	1320	24	204	
S-111	121.9	CL	19.9	2.07	1340	10	97	slickensided

* No peak obtained. Test terminated at indicated strain.

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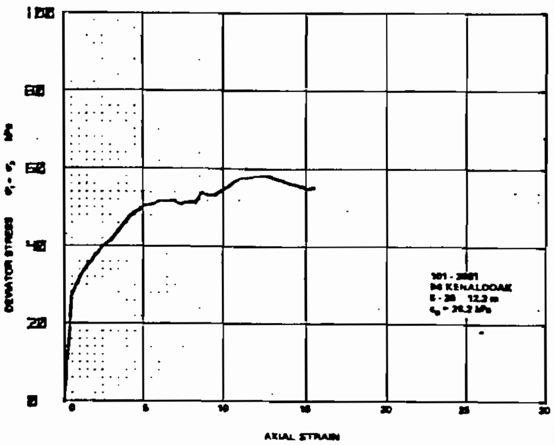
Phone (403) 451-2121

5664 BURLEIGH CRES S.E.
CALGARY, ALBERTA
Phone (403) 253-7121

UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project ____________
Address ____________
Job No. 101-2581
Date Tested 800327 By S.K.
Client ____________
Attention ____________

Test No. 25
Test Hole No. 5-26
Depth of Sample m 12.2
Confining Pressure kPa 400.0
Water Content % 32.5
Wet Density Mg/m³ 1.884
Peak Stress kPa 58.37



Reviewed by ____________ P.Eng.

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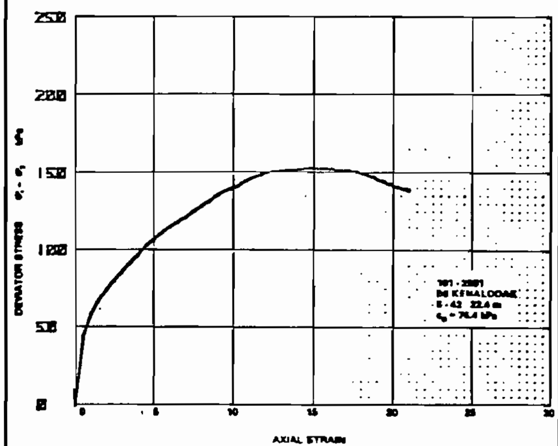
CALGARY, ALBERTA
Phone (403) 253-7121

UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project ____________
Address ____________
Job No. 101-2581
Date Tested 800317 By S.K.
Client ____________
Attention ____________

Test No. 8
Test Hole No. 5-42
Depth of Sample m 22.4
Confining Pressure kPa 500.0
Water Content % 32.9
Wet Density Mg/m³ 1.925
Peak Stress kPa 152.78



Reviewed by ____________ P.Eng.

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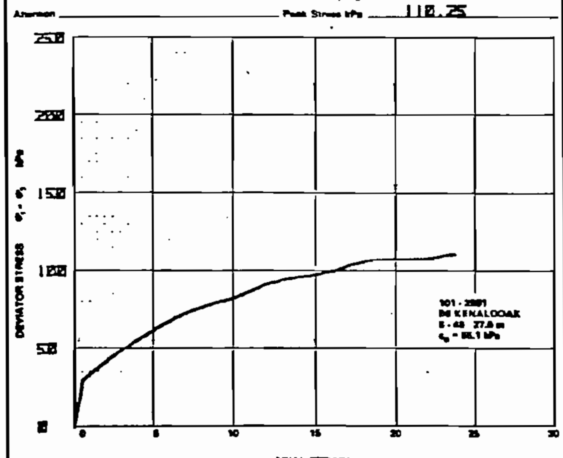
Phone (403) 253-7121

UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project ____________
Address ____________
Job No. 101-2581
Date Tested 800328 By S.K.
Client ____________
Attention ____________

Test No. 26
Test Hole No. 5-48
Depth of Sample m 27.8
Confining Pressure kPa 530.0
Water Content % 34.2
Wet Density Mg/m³ 1.929
Peak Stress kPa 110.25



Reviewed by ____________ P.Eng.

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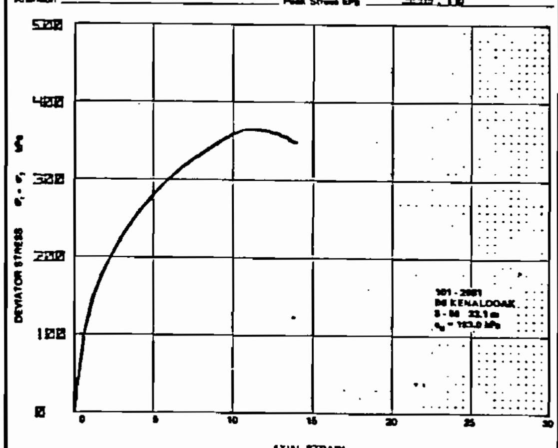
UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project ____________
Address ____________
Job No. 101-2581
Date Tested 800331 By S.K.
Client ____________
Attention ____________

Test No. 27
Test Hole No. 5-58
Depth of Sample m 33.1
Confining Pressure kPa 580.0
Water Content % 28.4
Wet Density Mg/m³ 1.943
Peak Stress kPa 356.10



Reviewed by ____________ P.Eng.

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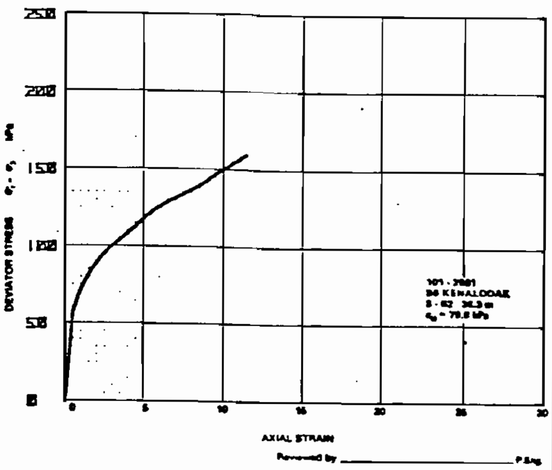
Phone (403) 451-2121

5684 BURLEIGH CRES. S.E.
CALGARY, ALBERTA
Phone (403) 253-7121

UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project: ______
Address: ______
Test No: 6
Test Hole No: 5-62
Job No: 101-2581
Depth of Sample m: 36.3
Date Tested: 800317 By: S.K.
Confining Pressure kPa: 610.0
Client: ______
Water Content %: 28.4
Wet Density Mg/m³: 1.931
Attention: ______
Peak Stress kPa: 159.57



Reviewed by ______ P.Eng.

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Phone (403) 451-2121

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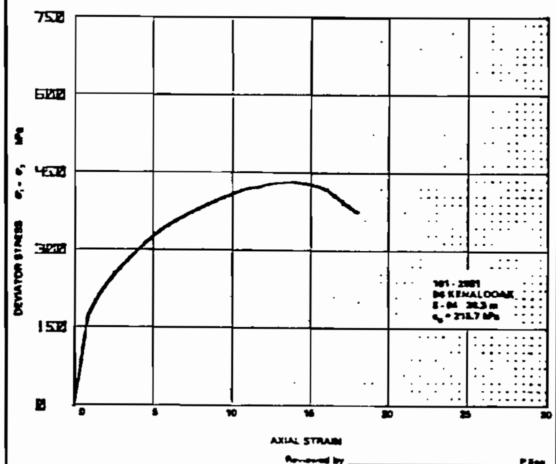
CALGARY, ALBERTA
Phone (403) 253-7121

UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project: ______
Address: ______
Test No: 7
Test Hole No: 5-64
Job No: 101-2581
Depth of Sample m: 39.3
Date Tested: 800317 By: S.K.
Confining Pressure kPa: 640.0
Client: ______
Water Content %: 26.2
Wet Density Mg/m³: 1.971
Attention: ______
Peak Stress kPa: 431.41



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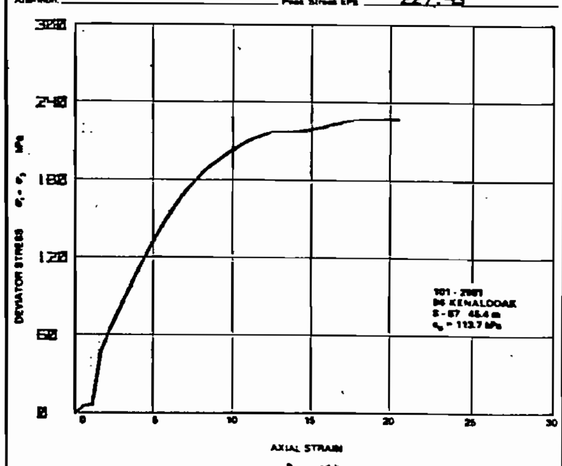
UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project: ______
Address: ______
Test No: 9
Test Hole No: 5-67
Job No: 101-2581
Depth of Sample m: 45.4
Date Tested: 800317 By: S.K.
Confining Pressure kPa: 690.0
Client: ______
Water Content %: 24.8
Wet Density Mg/m³: 1.958
Attention: ______
Peak Stress kPa: 227.43



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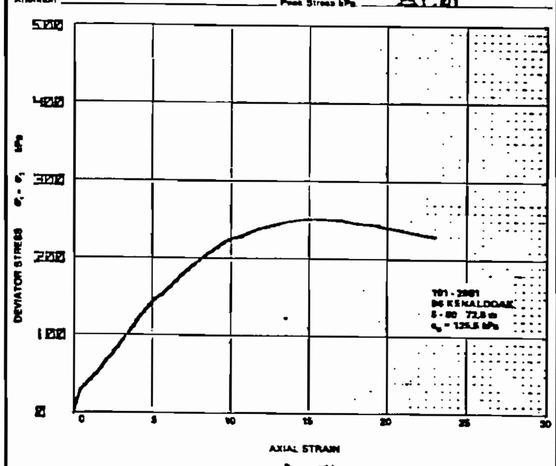
UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project: ______
Address: ______
Test No.: 36
Test Hole No.: 5-88 KEN.
Job No: 101-2581
Depth of Sample m: 72.8
Date Tested: 800516 By: H.R.H.
Confining Pressure kPa: 520.0
Client: ______
Water Content %: 35.8
Wet Density Mg/m³: 1.782
Attention: ______
Peak Stress kPa: 251.01



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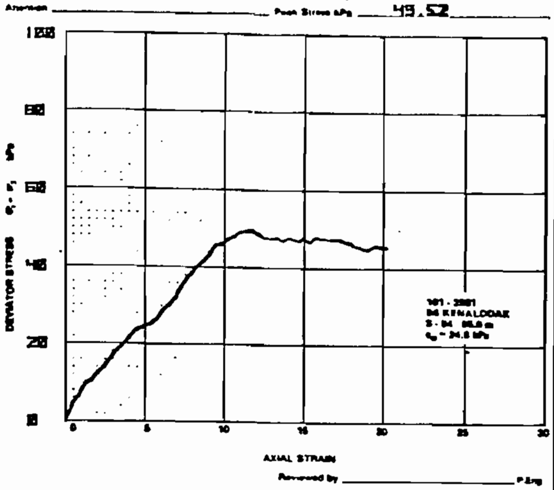
UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project		Test No	38
Address		Test Hole No	5-84 KEN.
Job No	101-2581	Depth of Sample m	85.8
Date Tested	822503 By H.R.H.	Confining Pressure kPa	1020.0
Client		Water Content %	30.7
		Wet Density Mg/m³	1.862
Attention		Peak Stress kPa	49.52



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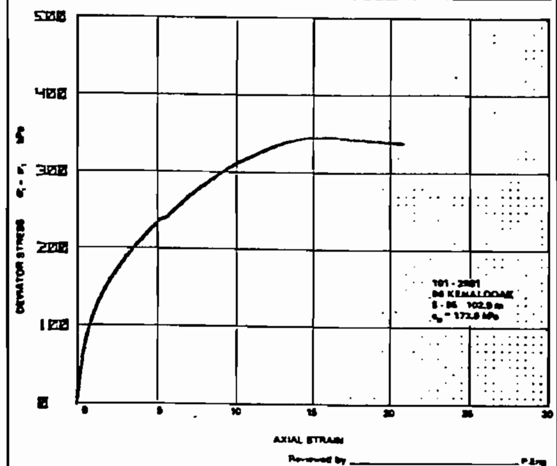
UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project		Test No	39
Address		Test Hole No	5-95 KEN.
Job No	101-2581	Depth of Sample m	102.8
Date Tested	820516 By H.R.H.	Confining Pressure kPa	1170.0
Client		Water Content %	21.7
		Wet Density Mg/m³	2.077
Attention		Peak Stress kPa	345.94



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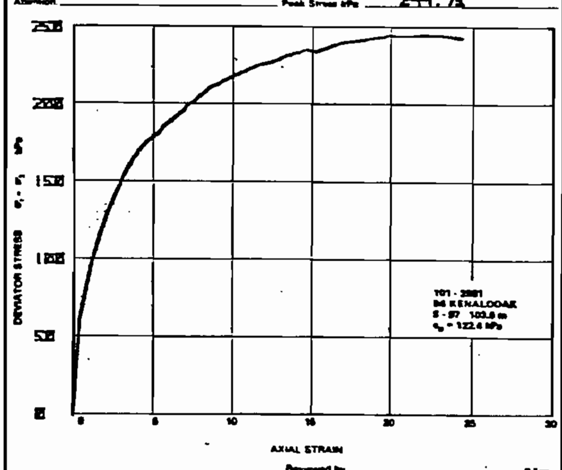
UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project		Test No	31
Address		Test Hole No	5-97 KEN.
Job No	101-2581	Depth of Sample m	103.6
Date Tested	822508 By H.R.H.	Confining Pressure kPa	1180.0
Client		Water Content %	21.7
		Wet Density Mg/m³	2.041
Attention		Peak Stress kPa	244.75



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UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project		Test No	40
Address		Test Hole No	5-98 KEN
Job No	101-2581	Depth of Sample m	105.4
Date Tested	822520 By S.K.	Confining Pressure kPa	1200.0
Client		Water Content %	25.4
		Wet Density Mg/m³	2.046
Attention		Peak Stress kPa	167.81

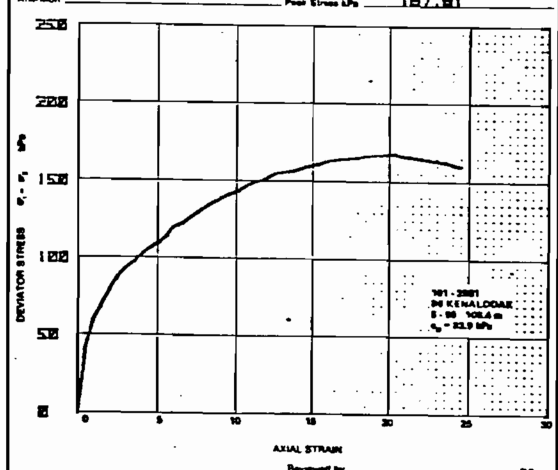

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PLATE A-22

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UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project ______

Address ______ Test No 41

______ Test Hole No 5-109 KEN

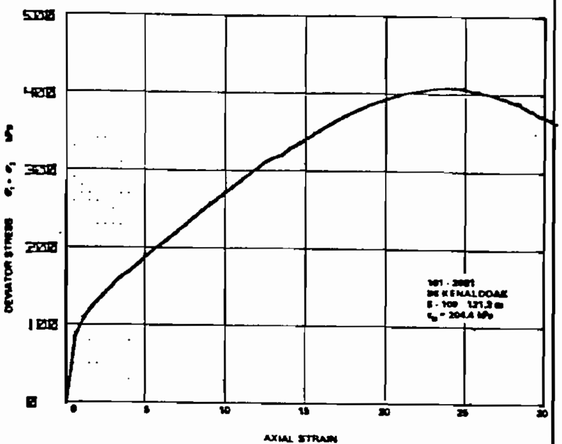
Job No 101-2581 Depth of Sample m 121.2

Date Tested 880520 By S.K. Confining Pressure kPa 1320.0

Client ______ Water Content % 19.5

______ Wet Density Mg/m³ 2.148

Attention ______ Peak Stress kPa 408.86



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UNCONSOLIDATED — UNDRAINED TRIAXIAL TEST

Project ______

Address ______ Test No 32

______ Test Hole No 5-111 KEN.

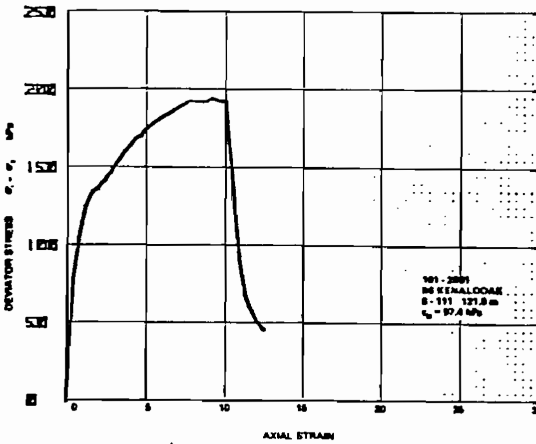
Job No 101-2581 Depth of Sample m 121.8

Date Tested 880518 By H.R.H. Confining Pressure kPa 1340.0

Client ______ Water Content % 19.9

______ Wet Density Mg/m³ 2.067

Attention ______ Peak Stress kPa 194.72



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KENALOOAK

DIRECT SHEAR TESTS

SAMPLE NUMBER	DEPTH (metres)	USC	MOISTURE CONTENT INITIAL (%)	MOISTURE CONTENT FINAL (%)	INITIAL BULK DENSITY (Mg/m^3)	NORMAL STRESS (σ_v) (kPa)	PEAK SHEAR STRESS (τ_p) (kPa)	RESIDUAL SHEAR STRESS (kPa)	VOLUMETRIC STRAIN ($\frac{\Delta v}{v}$) AT τ_p (%)	c'_p (kPa)	ϕ'_p (degrees)	ϕ'_r (degrees)
S-9B	106.1	CL	27.2	27.7	1.96	80	53	46	0.3			
			23.2	22.3	2.00	160	108	90	0.2	12	29	23
			27.4	25.2	1.97	240	141	114	0.4			

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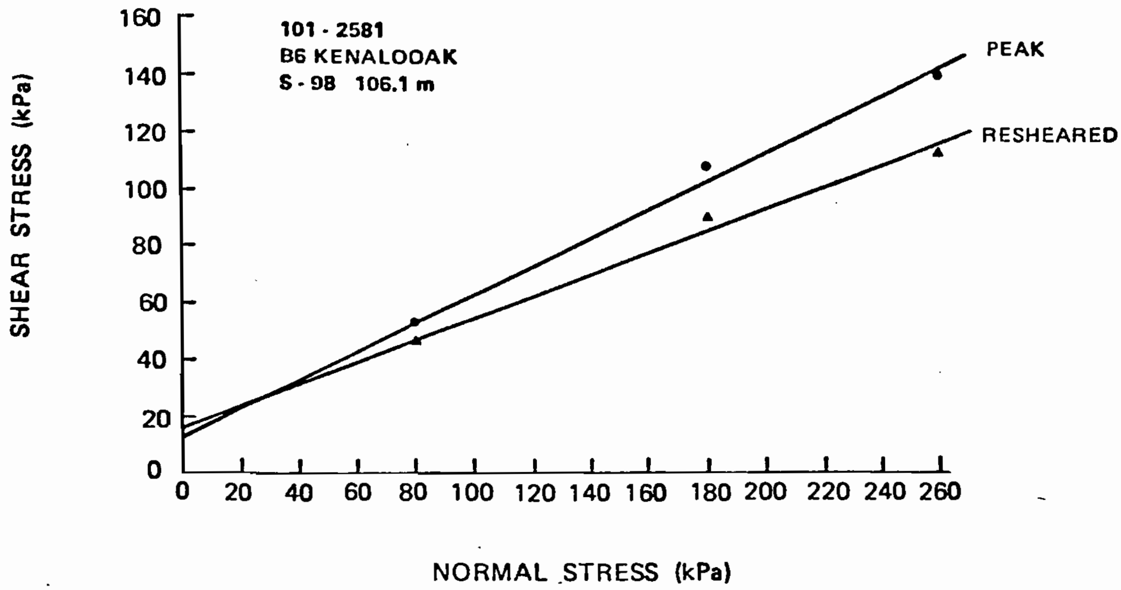

$\phi_p' = 29^\circ$ $c_p' = 12$ kPa
$\phi_r' = 23^\circ$ $c_r' = 16$ kPa

SUMMARY OF DIRECT SHEAR TEST RESULTS

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PLATE A-25

Sample Number	Penetration (meters)	Penetration (feet)	Water Content %	Miniature Vane Peak (kPa)	Miniature Vane Peak (ksf)	Miniature Vane Remolded (kPa)	Miniature Vane Remolded (ksf)	Sensitivity
33	0.5	2.0		4	0.092			
37	1.5	5.0		7	0.15			
37	1.5	5.0	29	8*	0.17	3*	0.06	2.7
4	2.6	9.0		16	0.33			
7	3.8	1.15	37	11*	0.23	3*	0.06	3.7
11	5.6	1.71	38	6*	0.13	2*	0.04	3.0
13	6.6	21.5	36	18*	0.38			
14	7.6	2.3	36	8*	0.17	2*	0.04	4.0
17	8.4	27.5	43	12	0.25			
18	8.5	2.59	43	8*	0.17	3*	0.06	2.7
20	9.6	31.5	43	15	0.33			
21	10.4	3.17	33	9*	0.19	3*	0.06	3.0
22	10.5	34.5	41	15	0.32			
24	11.6	38.0	39	13	0.28			
27	12.3	40.5	57	33*	0.69	8*	0.17	4.1
27	12.3	40.5	57	16	0.35			
29	12.6	41.5	46	18	0.39			
43	22.6	74.0	39	77	1.60			
45	24.1	80.0	32	100+*	2.10+	24*	0.50	4.2+
45	24.1	80.0	32	39	0.81			
47	25.8	84.5	32	89	1.86			
50	27.3	89.5	35	45	0.95			
52	28.6	94.0	26	120	2.53			
54	30.2	99.0	32	164	3.94			
56	31.7	104.0	33	146	3.05			
59	33.2	109.0	30	239	5.00+			
61	35.0	115.0	30	239	5.00+			

Notes:

1. Tests performed on samples that were either thawed subsequent to recovery or not frozen in situ.

2. Asterix denotes tests performed in EBA laboratory. All other tests were performed in the field.

FIELD AND LABORATORY VANE SHEAR TEST RESULTS
Boring 6, Kenalooak J-94
Beaufort Sea

	Penetration		Water Content	Shear Strength	
Sample Number	(meters)	(feet)	%	(kPa)	(ksf)
44	23.9	78.5	35	58.9	1.23
49	27.1	89.0	36	58.4	1.22
53	30.0	98.5	24	134.5	2.81
96	103.0	338.0	27	74.2	1.55
100	106.7	350.0	-	67.5	1.41
110	121.3	398.0	21	201.0	4.2

FIELD UNCONFINED COMPRESSION TEST RESULTS
Boring 6, Kenalooak J-94
Beaufort Sea

Sample Number	Penetration (meters)	Penetration (feet)	Shear Strength (kPa)	Shear Strength (ksf)
33	0.61	2.0	2.5	0.052
37	1.5	5.0	6.2	0.13
4	2.7	9.0	15.3	0.22
5	2.9	9.5	15.3	0.32
7	3.8	12.5	10.3	0.22
9	4.7	15.5	4.8	0.10
12	5.8	19.0	4.8	0.10
13	6.6	21.5	5.2	0.11
16	7.9	26.0	6.7	0.14
17	8.4	27.5	5.7	0.12
20	9.6	31.5	4.8	0.10
22	10.5	34.5	11.5	0.24
24	11.6	38.0	9.6	0.20
26	12.2	40.0	10.5	0.22
29	12.6	41.5	11.5	0.24
42	22.4	73.5	65.1	1.36
44	23.9	78.5	64.1	1.34
47	25.8	84.5	67.0	1.40
50	27.2	89.5	86.2	1.80
52	28.7	94.0	129.2	2.70
53	30.0	98.5	136.0	2.84
56	31.6	104.0	114.9	2.40
59	33.2	109.0	196.3	4.10
96	103.0	338.0	83.8	1.75
98	106.0	348.0	95.7	2.00
110	121.0	398.0	155.1	3.24

FIELD TORVANE TEST RESULTS
Boring 6, Kenalooak J-94
Beaufort Sea

KENALOOAK

THAW STRAIN TESTS

$(\epsilon_v = A + \sigma_v a_o)$

SAMPLE NUMBER	DEPTH (metres)	USC	MOISTURE CONTENT INITIAL (%)	MOISTURE CONTENT FINAL (%)	DENSITY FROZEN BULK (Mg/m^3)	DENSITY INITIAL DRY (Mg/m^3)	A	a_o	EFFECTIVE NORMAL STRESS (σ_v) (kPa)	C_v (m^2/yr)	k[1] (m/s)
S-62	36.3	CH	34.5	31.7	1.85	1.55	2.4	0.05	50		(9.1E-06)
										450	1.3E-07
									100		
										1.9	2.1E-10
									200		
S-67	45.4	CH	25.5	27.0	1.98	1.62	2.4	0.01	50		
										0.69	2.7E-11
									100		
										0.87	4.3E-11
									200		
S-74	57.9	CL	23.8	22.2	1.99	1.75	5.3	0.02	50		(3.5E-09)
										6.4	6.2E-10
									100		(2.0E-09)
										8.4	4.7E-10
									200		(1.2E-09)
S-79	69.8	ML	24.3	23.7	2.01	1.70	3.6	0.01	50		
										5.8	3.7E-10
									100		
										4.3	1.8E-10
									200		
S-81	75.9	ML	30.0	30.7	1.94	1.56	3.7	0.02	50		(3.3E-10)
										1.3	8.6E-11
									100		
										1.4	6.9E-11
									200		

[1] Permeabilities shown in parentheses represent direct permeability readings; the remainder have been calculated.

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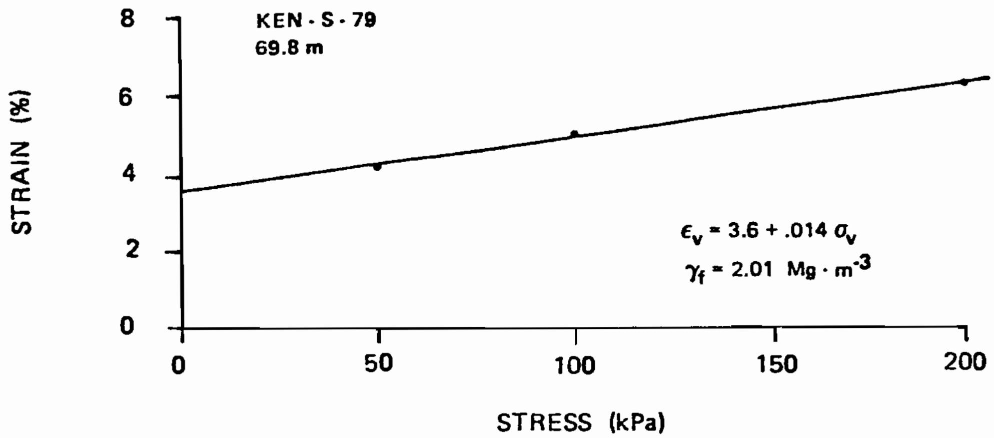

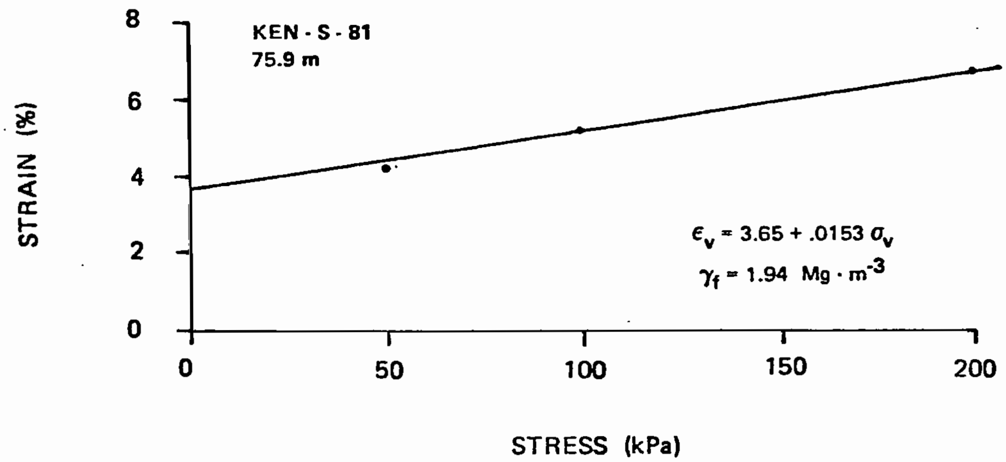

THAW STRAIN TEST RESULTS

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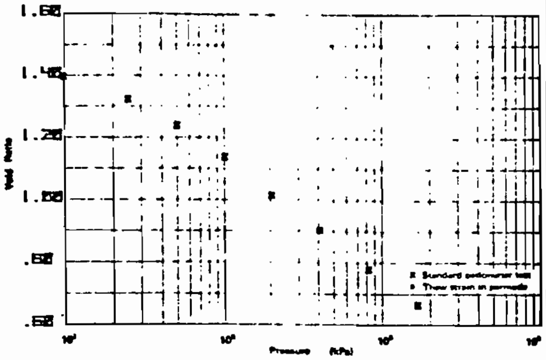
CONSOLIDATION TEST RESULTS

Project:
Address:
Test No.: 25
Borehole No.: 5-15 KEN.
Depth (m): 7.62
Project No.: 101-2581
Date Tested: 800508 By H.R.H.
Diameter (mm): 49.9
Specific Gravity: 2.8



	INITIAL	FINAL
Height (mm):	25.6	21.3
Water Content (%):	51.9	34.1
Wet Density (Mg/m³):	1.72	1.90
Dry Density (Mg/m³):	1.13	1.42
Void Ratio	1.4393	.9505
Saturation (%):	100.7	100.0

Sample Description: SILTY CLAY
Overburden Pressure P_o ____ kPa
Swelling Pressure P_s 1.5 kPa
Pre-Consolidation Pressure P_c ____ kPa
Compression Index C_c ____

Note: 1 kPa = 1.044 x 10⁻² T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted.

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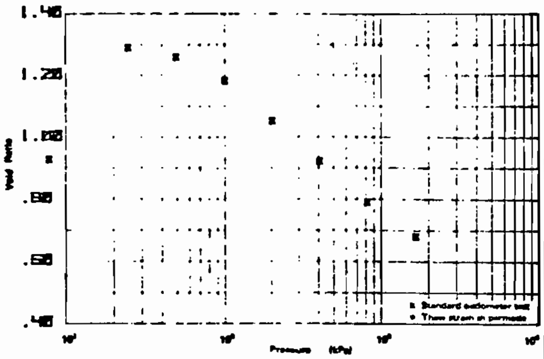
CONSOLIDATION TEST RESULTS

Project:
Address:
Test No.: 26
Borehole No.: 5-28 KEN.
Depth (m): 12.50
Project No.: 101-2581
Date Tested: 800508 By H.R.H.
Diameter (mm): 49.9
Specific Gravity: 2.8



	INITIAL	FINAL
Height (mm):	25.6	22.3
Water Content (%):	45.7	33.3
Wet Density (Mg/m³):	1.77	1.91
Dry Density (Mg/m³):	1.21	1.44
Void Ratio	1.3022	.9308
Saturation (%):	100.3	100.0

Sample Description: SILTY CLAY
Overburden Pressure P_o ____ kPa
Swelling Pressure P_s 8.8 kPa
Pre-Consolidation Pressure P_c ____ kPa
Compression Index C_c ____

Note: 1 kPa = 1.044 x 10⁻² T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted.

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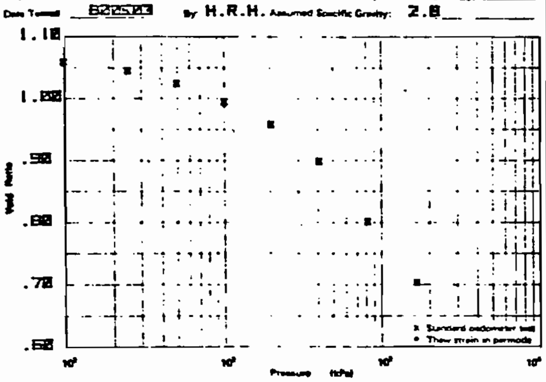
CONSOLIDATION TEST RESULTS

Project:
Address:
Test No.: 27
Borehole No.: 5-46 KEN.
Depth (m): 25.60
Project No.: 101-2581
Date Tested: 800508 By H.R.H.
Diameter (mm): 49.9
Assumed Specific Gravity: 2.8



	INITIAL	FINAL
Height (mm):	25.6	24.2
Water Content (%):	37.2	31.6
Wet Density (Mg/m³):	1.88	1.98
Dry Density (Mg/m³):	1.37	1.50
Void Ratio	1.0650	.8817
Saturation (%):	97.7	100.0

Sample Description: SILTY CLAY
Overburden Pressure P_o ____ kPa
Swelling Pressure P_s 2.5 kPa
Pre-Consolidation Pressure P_c ____ kPa
Compression Index C_c ____

Note: 1 kPa = 1.044 x 10⁻² T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted.

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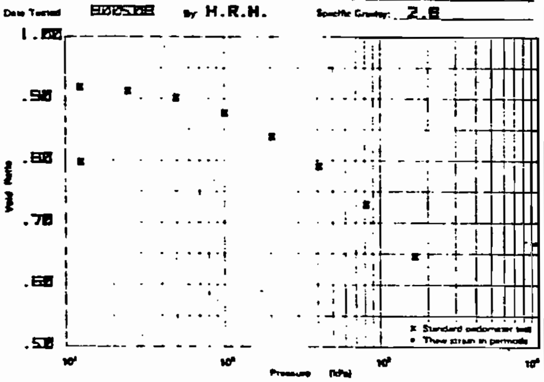
CONSOLIDATION TEST RESULTS

Project:
Address:
Test No.: 28
Borehole No.: 5-51 KEN.
Depth (m): 28.50
Project No.: 101-2581
Date Tested: 800508 By H.R.H.
Diameter (mm): 49.9
Specific Gravity: 2.8



	INITIAL	FINAL
Height (mm):	25.6	24.9
Water Content (%):	31.4	28.7
Wet Density (Mg/m³):	1.95	2.04
Dry Density (Mg/m³):	1.48	1.58
Void Ratio	.9184	.7979
Saturation (%):	95.3	100.0

Sample Description: CLAYEY SILT
Overburden Pressure P_o ____ kPa
Swelling Pressure P_s 12.5 kPa
Pre-Consolidation Pressure P_c ____ kPa
Compression Index C_c ____

Note: 1 kPa = 1.044 x 10⁻² T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted.

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PLATE A-32

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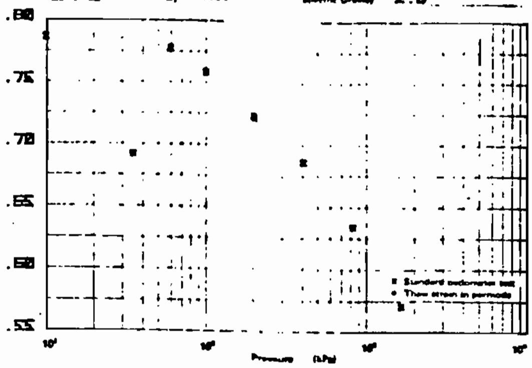
CONSOLIDATION TEST RESULTS

Project	
Address	
Project No.	101-2581
Date Tested	800407 by S.K.
Test No.	8
Borehole No.	5-62 KEN.
Depth (m)	36.27
Diameter (mm)	49.8
Specific Gravity	2.8



	INITIAL	FINAL
Height (mm)	25.6	25.0
Water Content (%)	25.3	24.8
Wet Density (Mg/m³)	1.97	2.07
Dry Density (Mg/m³)	1.56	1.66
Void Ratio	.8003	.6915
Saturation (%)	91.7	100.0

Sample Description: SILTY CLAY

Overburden Pressure P_o ____ kPa
Swelling Pressure P_s ____ kPa
Pre-Consolidation Pressure P_c ____ kPa
Compression Index C_c ____

Note: 1 kPa = 1.044 x 10⁻² T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted.

3072

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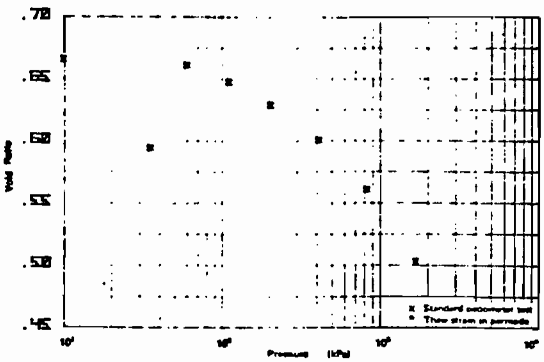
14535 - 118m AVENUE, EDMONTON, ALBERTA, Phone (403) 451-2121 — 5664 BURLEIGH CRES. S.E., CALGARY, ALBERTA, Phone (403) 253-7121

CONSOLIDATION TEST RESULTS

Project	
Address	
Project No.	101-2581
Date Tested	800414 by S.K.
Test No.	12
Borehole No.	5-67 KEN
Depth (m)	45.42
Diameter (mm)	49.9
Specific Gravity	2.8



	INITIAL	FINAL
Height (mm)	25.6	25.4
Water Content (%)	24.6	21.4
Wet Density (Mg/m³)	2.03	2.07
Dry Density (Mg/m³)	1.63	1.70
Void Ratio	.6734	.5348
Saturation (%)	101.7	100.0

Sample Description: OLIVE GRAY CLAY SILT

Overburden Pressure P_o ____ kPa
Swelling Pressure P_s 35 kPa
Pre-Consolidation Pressure P_c ____ kPa
Compression Index C_c ____

Note: 1 kPa = 1.044 x 10⁻² T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted.

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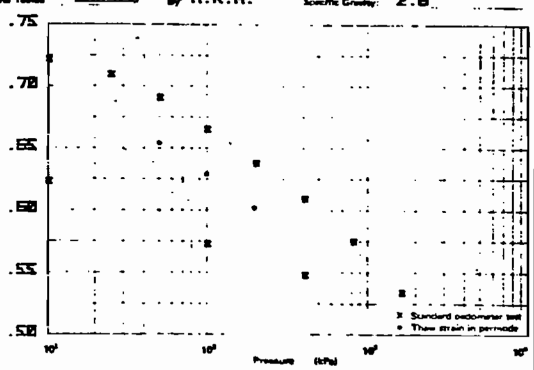
CONSOLIDATION TEST RESULTS

Project	
Address	
Project No.	101-2581
Date Tested	800320 by H.R.H.
Test No.	1
Borehole No.	5-74
Depth (m)	57.91
Diameter (mm)	59.9
Specific Gravity	2.8



	INITIAL	FINAL
Height (mm)	25.6	24.3
Water Content (%)	25.2	22.4
Wet Density (Mg/m³)	2.00	2.14
Dry Density (Mg/m³)	1.60	1.75
Void Ratio	.7733	.6229
Saturation (%)	90.6	100.0

Sample Description: SILT/SOME CLAY TRACE SAND

Overburden Pressure P_o ____ kPa
Swelling Pressure P_s 0.0 kPa
Pre-Consolidation Pressure P_c ____ kPa
Compression Index C_c ____

Note: 1 kPa = 1.044 x 10⁻² T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted.

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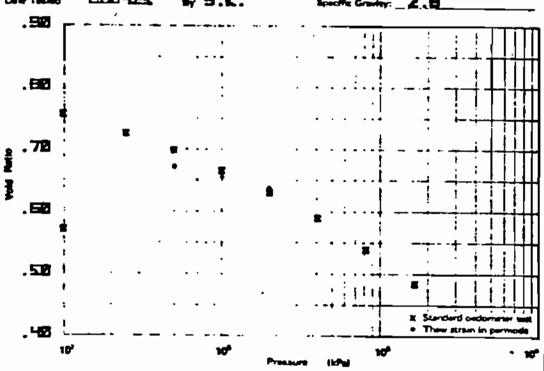
CONSOLIDATION TEST RESULTS

Project	
Address	
Project No.	101-2581
Date Tested	800425 by S.K.
Test No.	16
Borehole No.	5-79 KEN
Depth (m)	69.80
Diameter (mm)	45.6
Specific Gravity	2.8



	INITIAL	FINAL
Height (mm)	25.7	22.3
Water Content (%)	29.1	20.6
Wet Density (Mg/m³)	1.95	2.19
Dry Density (Mg/m³)	1.51	1.81
Void Ratio	.8760	.5717
Saturation (%)	92.3	100.0

Sample Description: CLAY-SILT

Overburden Pressure P_o ____ kPa
Swelling Pressure P_s 0.0 kPa
Pre-Consolidation Pressure P_c ____ kPa
Compression Index C_c ____

Note: 1 kPa = 1.044 x 10⁻² T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted.

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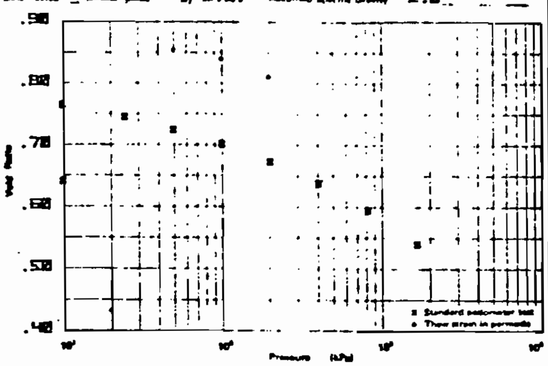
Phone (403) 253-7121

CONSOLIDATION TEST RESULTS

Project:
Address:
Project No.: 101-2581
Date Tested: 800425 By: S.K.
Test No.: 18
Borehole No.: 5-81 KEN
Depth (m): 75.90
Diameter (mm): 49.9
Assumed Specific Gravity: 2.8



	INITIAL	FINAL
Height (mm):	26.6	24.4
Water Content (%):	27.2	23.2
Wet Density (Mg/m³):	2.00	2.10
Dry Density (Mg/m³):	1.57	1.71
Void Ratio	.7852	.6416
Saturation (%):	96.0	100.0

Sample Description: SILTY CLAY

Overburden Pressure P_o kPa
Swelling Pressure P_s 0.0 kPa
Pre-Consolidation Pressure P_c kPa
Compression Index C_c

Note: 1 kPa = 1.044 x 10^{-2} T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted

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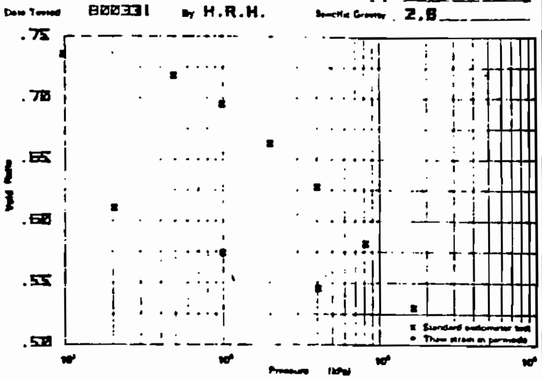
CONSOLIDATION TEST RESULTS

Project:
Address:
Project No.: 101-2581
Date Tested: 800331 By: H.R.H.
Test No.: 6
Borehole No.: 5-97
Depth (m): 103.63
Diameter (mm): 59.9
Specific Gravity: 2.8



	INITIAL	FINAL
Height (mm):	26.6	24.5
Water Content (%):	24.7	21.9
Wet Density (Mg/m³):	1.99	2.12
Dry Density (Mg/m³):	1.60	1.74
Void Ratio	.7523	.6112
Saturation (%):	91.6	100.0

Sample Description: SILTY CLAY

Overburden Pressure P_o kPa
Swelling Pressure P_s kPa
Pre-Consolidation Pressure P_c kPa
Compression Index C_c

Note: 1 kPa = 1.044 x 10^{-2} T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted

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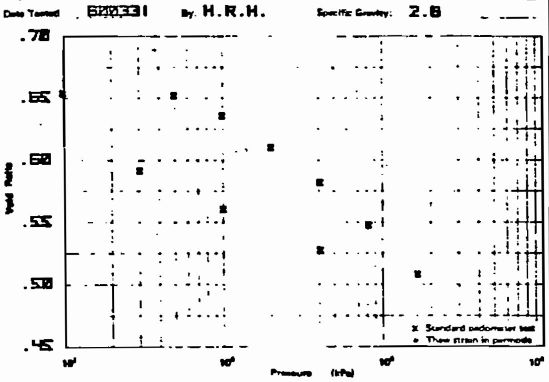
CALGARY, ALBERTA
Phone (403) 253-7121

CONSOLIDATION TEST RESULTS

Project:
Address:
Project No.: 101-2581
Date Tested: 800331 By: H.R.H.
Test No.: 7
Borehole No.: 5-111
Depth (m): 121.92
Diameter (mm): 59.9
Specific Gravity: 2.8



	INITIAL	FINAL
Height (mm):	26.6	25.2
Water Content (%):	22.2	21.2
Wet Density (Mg/m³):	2.04	2.13
Dry Density (Mg/m³):	1.67	1.76
Void Ratio	.6804	.5920
Saturation (%):	91.1	100.0

Sample Description: CLAY, SOME SILT

Overburden Pressure P_o kPa
Swelling Pressure P_s kPa
Pre-Consolidation Pressure P_c kPa
Compression Index C_c

Note: 1 kPa = 1.044 x 10^{-2} T/ft²

Tested in accordance with ASTM standard D2435 unless otherwise noted

3072

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CONSOLIOATION TESTS

SAMPLE NUMBER	DEPTH (metres)	USC	MOISTURE CONTENT INITIAL (%)	MOISTURE CONTENT FINAL (%)	DENSITY INITIAL BULK (Mg/m^3)	DENSITY INITIAL DRY (Mg/m^3)	EFFECTIVE OVERBURDEN PRESSURE (P'_o) (kPa)	PRECONSOLIDATION PRESSURE (P'_c) (kPa)	c_c	EFFECTIVE NORMAL STRESS (σ'_v) (kPa)	c_v (m^2/yr)	m_v (1/MPa)	k (m/s)
S-15	7.6	CL	51.9	34.1	1.72	1.13	72	38	0.39	10			
											1.2	2.1	7.9E-10
										25			
											1.6	2.0	9.7E-10
										50			
											1.6	1.5	7.3E-10
										100			
											1.5	0.90	4.2E-10
										200			
											1.8	0.57	3.1E-10
										400			
											2.3	0.28	2.0E-10
										800			
											2.6	0.16	1.3E-10
										1600			
S-20	12.5	CL	46.7	33.3	1.77	1.21	96	100	0.41	25			
											14	0.26	1.2E-09
										50			
											4.5	0.52	7.2E-10
										100			
											2.1	0.68	4.5E-10
										200			
											1.5	0.59	2.7E-10
										400			
											1.7	0.32	1.7E-10
										800			
											1.9	0.17	1.0E-10
										1600			

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CONSOLIDATION TESTS

SAMPLE NUMBER	DEPTH (metres)	USC	MOISTURE CONTENT INITIAL (%)	MOISTURE CONTENT FINAL (%)	DENSITY INITIAL BULK (Mg/m^3)	DENSITY INITIAL DRY (Mg/m^3)	EFFECTIVE OVERBURDEN PRESSURE (P'_o) (kPa)	PRECONSOLIDATION PRESSURE (P'_c) (kPa)	C_c	EFFECTIVE NORMAL STRESS (σ'_v) (kPa)	c_v (m^2/yr)	m_v (1/MPa)	k (m/s)
S-46	25.6	ML	37.2	31.6	1.88	1.37	227	400	0.32	10			
											29	0.47	4.3E-09
										25			
											32	0.44	4.4E-09
										50			
											36	0.41	4.5E-09
										100			
											44	0.29	4.0E-09
										200			
											51	0.18	2.9E-09
										400			
											44	0.15	2.1E-09
										800			
											37	0.13	1.5E-09
										1600			
S-51	28.5	ML	31.4	28.7	1.95	1.48	250	176 - 275	0.22-0.27	25			
											14	0.23	9.9E-10
										50			
											11	0.22	7.8E-10
										100			
											9.4	0.25	7.3E-10
										200			
											10	0.20	6.4E-10
										400			
											12	0.13	4.9E-10
										800			
											11	0.08	3.0E-10
										1600			

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CONSOLIDATION TESTS

SAMPLE NUMBER	DEPTH (metres)	USC	MOISTURE CONTENT INITIAL (%)	MOISTURE CONTENT FINAL (%)	DENSITY INITIAL BULK (Mg/m^3)	DENSITY INITIAL (Mg/m^3)	EFFECTIVE OVERBURDEN PRESSURE (P'_o) (kPa)	PRECONSOLIDATION PRESSURE (P'_c) (kPa)	c_c	EFFECTIVE NORMAL STRESS (σ'_v) (kPa)	c_v (m^2/yr)	m_v (1/MPa)	k (m/s)
S-62	36.3	CH	26.3	24.8	1.97	1.56	311	120 - 190	0.16-0.22	10			
											1.9	0.09	5.5E-11
										60			
											3.1	0.26	2.5E-10
										100			
											3.0	0.20	1.8E-10
										200			
											3.8	0.11	1.3E-10
										400			
											3.2	0.08	7.7E-11
										800			
											3.1	0.05	4.7E-11
										1600			
S-67	45.4	CH	24.6	21.4	2.03	1.63	397	162 - 257	0.15-0.20	10			
											2.3	0.05	3.5E-11
										60			
											7.2	0.16	3.6E-10
										100			
											8.4	0.13	3.3E-10
										200			
											8.6	0.09	2.3E-10
										400			
											9.7	0.06	1.9E-10
										800			
											9.1	0.05	1.3E-10
										1600			

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CONSOLIDATION TESTS

SAMPLE NUMBER	DEPTH (metres)	USC	MOISTURE CONTENT INITIAL (%)	MOISTURE CONTENT FINAL (%)	DENSITY INITIAL BULK (Mg/m^3)	DENSITY INITIAL DRY (Mg/m^3)	EFFECTIVE OVERBURDEN PRESSURE (P'_o) (kPa)	PRECONSOLIDATION PRESSURE (P'_c) (kPa)	c_c	EFFECTIVE NORMAL STRESS (σ'_v) (kPa)	c_v (m^2/yr)	m_v (1/MPa)	k (m/s)
S-74	57.9	CL	29.2	22.4	2.00	1.60	512	68 - 105	0.12-0.14	10			
											14	0.46	2.0E-09
										25			
											13	0.44	1.8E-09
										50			
											11	0.29	1.0E-09
										100			
											9.6	0.16	4.9E-10
										200			
											14	0.09	3.8E-10
										400			
											9.2	0.05	1.5E-10
										800			
											15	0.03	1.6E-10
										1600			
S-79	69.8	ML	29.1	20.6	1.95	1.51	600	200 - 270	0.16-0.18	10			
											9.6	6.3	1.9E-08
										25			
											8.8	1.2	3.2E-09
										50			
											7.7	0.62	1.5E-09
										100			
											7.4	0.38	8.8E-10
										200			
											7.5	0.21	4.8E-10
										400			
											8.7	0.13	3.4E-10
										800			
											9.8	0.08	2.4E-10
										1600			

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CONSOLIDATION TESTS

SAMPLE NUMBER	DEPTH (metres)	USC	MOISTURE CONTENT INITIAL (%)	MOISTURE CONTENT FINAL (%)	DENSITY INITIAL BULK (Mg/m^3)	DENSITY INITIAL DRY (Mg/m^3)	EFFECTIVE OVERBURDEN PRESSURE (P'_o) (kPa)	PRECONSOLIDATION PRESSURE (P'_c) (kPa)	C_c	EFFECTIVE NORMAL STRESS (σ'_v) (kPa)	c_v (m^2/yr)	m_v (1/MPa)	k (m/s)
S-81	75.9	ML	27.2	23.2	2.00	1.57	670	800	0.18	10			
											7.9	1.2	2.9E-09
										25			
											7.7	0.76	1.8E-09
										50			
											7.2	0.44	9.9E-10
										100			
											9.6	0.27	8.2E-10
										200			
											12	0.17	6.0E-10
										400			
											14	0.10	4.4E-10
										800			
											18	0.07	3.8E-10
										1600			
S-97	103.6	CL	24.7	21.9	1.99	1.60	898	100 - 151	0.15-0.17	10			
											8.5	0.25	6.5E-10
										50			
											7.9	0.27	6.5E-10
										100			
											8.9	0.19	5.2E-10
										200			
											11	0.11	3.6E-10
										400			
											11	0.07	2.4E-10
										800			
											11	0.04	1.4E-10
										1600			

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CONSOLIDATION TESTS

SAMPLE NUMBER	DEPTH (metres)	USC	MOISTURE CONTENT INITIAL (%)	MOISTURE CONTENT FINAL (%)	DENSITY INITIAL BULK (Mg/m^3)	DENSITY INITIAL DRY (Mg/m^3)	EFFECTIVE OVERBURDEN PRESSURE (P'_o) (kPa)	PRECONSOLIDATION PRESSURE (P'_c) (kPa)	c_c	EFFECTIVE NORMAL STRESS (σ'_v) (kPa)	c_v (m^2/yr)	m_v (1/MPa)	k (m/s)
S-111	121.9	CL	22.2	21.2	2.04	1.67	1053	103 - 148	0.12-0.13	10			
											3.1	0.02	2.2E-11
										50			
											2.0	0.20	1.2E-10
										100			
											2.1	0.16	1.0E-10
										200			
											3.9	0.09	9.4E-11
										400			
											4.2	0.50	7.0E-11
										800			
											4.3	0.03	4.3E-11
										1600			

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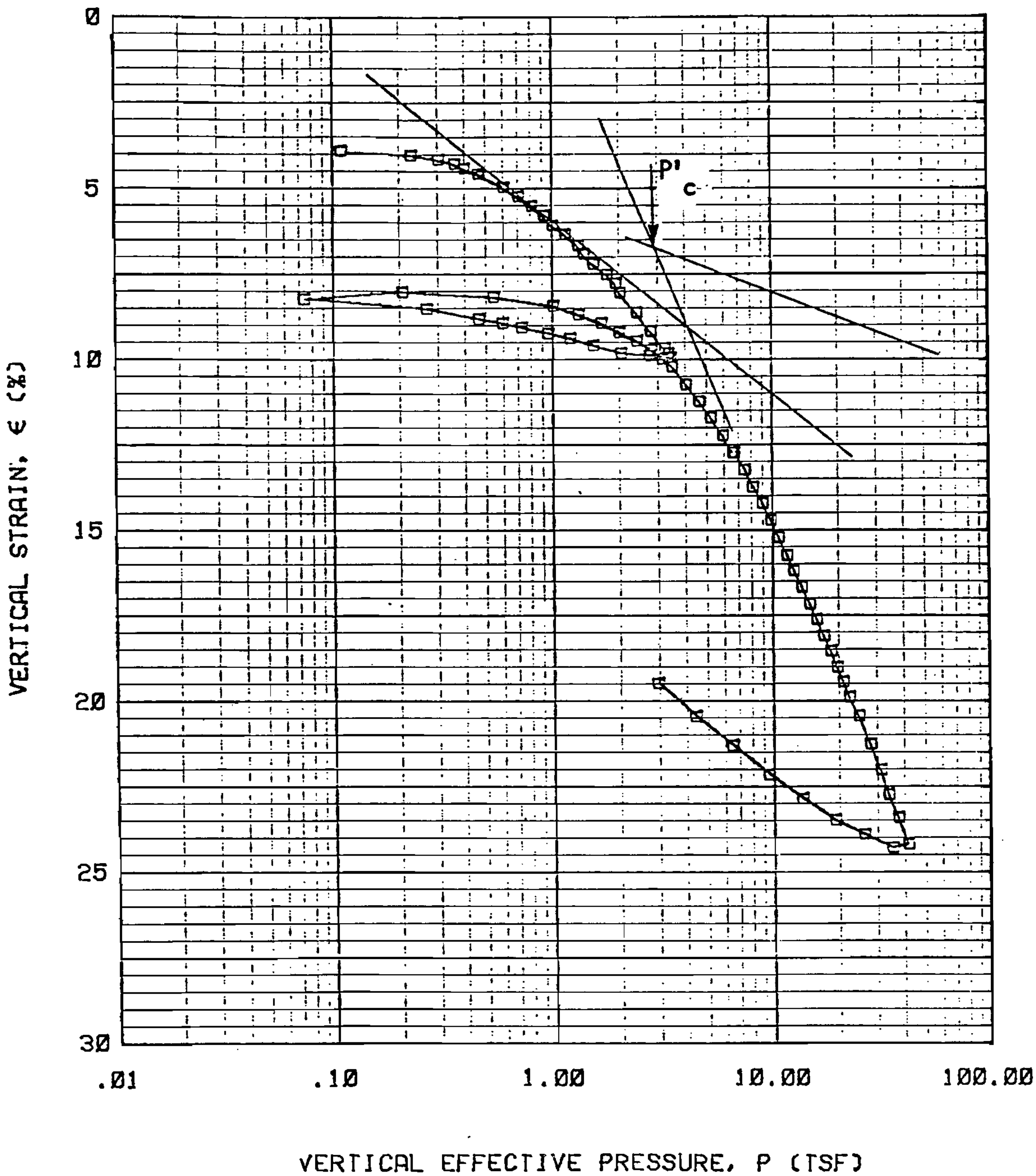
0179-0128

BORING: KEN PENETRATION: 149.0 FT.
MATERIAL: Very stiff dark olive gray clay
$P'_c = 2.9$ tsf
$c_c = 0.277$

DRY MASS DENSITY: 89 PCF
WATER CONTENT: 35 %
LIQUID LIMIT: 63
PLASTIC LIMIT: 27
SPECIFIC GRAVITY: 2.72 (ASSUMED)
INITIAL VOID RATIO: 0.913



CRS THAW - CONSOLIDATION TEST RESULTS

Elapsed Time (min)	Effective Pressure (kPa)	Excess Pore Press. (kPa)	Vertical Strain (%)	Void Ratio	c_v (cm2/day)	Effective Pressure (tsf)
0.0	0.0	0.00	3.856	0.8357	0.0	0.00
5.0	10.2	12.70	3.950	0.8277	1520.0	0.11
10.0	21.4	14.70	4.089	0.8353	474.7	0.22
15.0	28.5	19.00	4.218	0.8328	308.0	0.30
20.0	34.1	21.90	4.348	0.8302	189.8	0.36
25.0	37.8	23.90	4.503	0.8273	112.7	0.39
30.0	43.8	26.10	4.632	0.8245	154.4	0.46
45.0	57.7	29.60	5.046	0.8160	99.6	0.60
55.0	66.9	30.80	5.205	0.8120	84.2	0.70
65.0	76.9	32.60	5.590	0.8056	89.7	0.80
75.0	88.1	34.60	5.875	0.8011	93.5	0.92
85.0	96.5	35.10	6.159	0.7956	62.7	1.01
95.0	109.0	36.90	6.418	0.7907	94.0	1.14
105.0	125.2	38.80	6.729	0.7848	114.2	1.31
115.0	132.6	42.70	6.987	0.7798	60.2	1.38
125.0	146.8	42.10	7.272	0.7744	70.2	1.53
135.0	169.1	42.30	7.583	0.7684	129.4	1.77
145.0	184.7	43.20	7.841	0.7635	91.6	1.93
155.0	194.3	43.90	8.100	0.7585	55.0	2.03
175.0	231.2	48.40	8.670	0.7476	103.1	2.41
195.0	267.8	46.00	9.213	0.7372	87.2	2.80
215.0	309.2	49.50	9.731	0.7273	106.5	3.23
220.0	320.6	49.20	9.860	0.7248	105.3	3.35
225.0	331.0	48.60	9.989	0.7224	94.2	3.46
230.0	319.2	8.20	9.989	0.7224	0.0	3.33
235.0	263.6	-24.50	9.928	0.7234	4276.5	2.75
245.0	194.0	-43.70	9.860	0.7249	557.5	2.03
255.0	143.9	-47.20	9.653	0.7288	207.0	1.50
265.0	113.4	-53.90	9.446	0.7328	160.7	1.18
270.0	90.6	-53.50	9.291	0.7357	105.3	0.95
275.0	68.7	-54.40	9.135	0.7387	135.0	0.72
280.0	55.8	-51.80	9.006	0.7412	96.4	0.58
285.0	43.5	-54.00	8.877	0.7437	132.2	0.45
295.0	25.0	-50.60	8.566	0.7496	73.4	0.26
305.0	7.0	-47.90	8.281	0.7550	78.4	0.07
325.0	19.5	6.30	8.074	0.7590	0.0	0.20
335.0	51.5	23.90	8.230	0.7560	692.7	0.54
345.0	95.5	31.00	8.514	0.7506	422.7	1.00
355.0	124.7	34.00	8.747	0.7461	227.6	1.30
365.0	158.8	35.30	9.006	0.7412	227.9	1.66
375.0	191.0	35.70	9.265	0.7362	214.4	1.99
385.0	220.2	38.20	9.524	0.7313	252.1	2.30
395.0	266.2	40.90	9.782	0.7263	227.0	2.78
405.0	301.0	41.10	10.041	0.7214	196.7	3.14
415.0	332.6	48.00	10.274	0.7169	186.5	3.47
435.0	387.7	46.00	10.792	0.7070	130.0	4.05
455.0	444.9	53.00	11.283	0.6976	140.6	4.65
475.0	502.2	53.10	11.775	0.6882	132.9	5.24
495.0	566.5	56.80	12.203	0.6783	136.4	5.92
515.0	630.1	56.90	12.784	0.6689	132.0	6.58

RESULTS OF CRS THAW-CONSOLIDATION TEST

Boring 6, Kenalooak J-94
149-ft Penetration
Beaufort Sea

535.0	700.9	61.20	13.276	0.6595	162.1	7.22
555.0	772.6	60.50	13.768	0.6501	150.5	8.07
575.0	850.5	63.50	14.256	0.6407	164.3	8.88
595.0	933.6	68.10	14.751	0.6312	165.2	9.75
615.0	1017.6	71.40	15.243	0.6218	154.3	10.63
635.0	1113.3	79.60	15.760	0.6119	165.0	11.63
655.0	1204.4	75.90	16.220	0.6030	139.6	12.58
675.0	1304.2	79.80	16.692	0.5941	159.4	13.62
695.0	1413.5	88.20	17.184	0.5847	164.1	14.76
715.0	1524.0	89.10	17.650	0.5758	149.0	15.91
735.0	1639.1	90.70	18.115	0.5669	152.0	17.13
755.0	1759.4	97.50	18.555	0.5585	154.7	18.37
775.0	1883.7	94.60	19.021	0.5495	147.1	19.67
795.0	2016.5	99.60	19.461	0.5411	160.2	21.06
815.0	2151.8	100.80	19.901	0.5327	153.8	22.47
835.0	2376.7	203.60	20.445	0.5223	216.0	24.82
855.0	2673.9	259.90	21.273	0.5065	159.4	27.92
915.0	2973.7	218.50	22.023	0.4921	41.2	31.05
935.0	3239.1	181.90	22.748	0.4782	128.5	33.82
955.0	3598.3	287.50	23.421	0.4654	192.4	37.58
975.0	3977.7	297.20	24.197	0.4505	136.5	41.54
995.0	3324.6	-152.20	24.275	0.4490	0.0	34.72
1015.0	2466.6	-287.10	23.912	0.4560	442.2	25.76
1035.0	1793.6	-365.90	23.498	0.4639	230.4	18.73
1055.0	1271.6	-423.90	22.877	0.4758	148.9	13.28
1075.0	882.2	-448.50	22.179	0.4891	99.0	9.21
1095.0	603.2	-449.70	21.325	0.5055	67.4	6.30
1115.0	409.3	-424.80	20.470	0.5218	44.7	4.27
1135.0	274.3	-389.00	19.513	0.5401	30.7	2.86

RESULTS OF CRS THAW-CONSOLIDATION TEST
Boring 6, Kenalooak J-94
149-ft Penetration
Beaufort Sea

0179-0129

BORING: KEN   PENETRATION: 190.0 FT.

MATERIAL: Very stiff brown silty clay

$P'_c = 9.1$ tsf

$c_c = 0.218$

DRY MASS DENSITY: 101 PCF

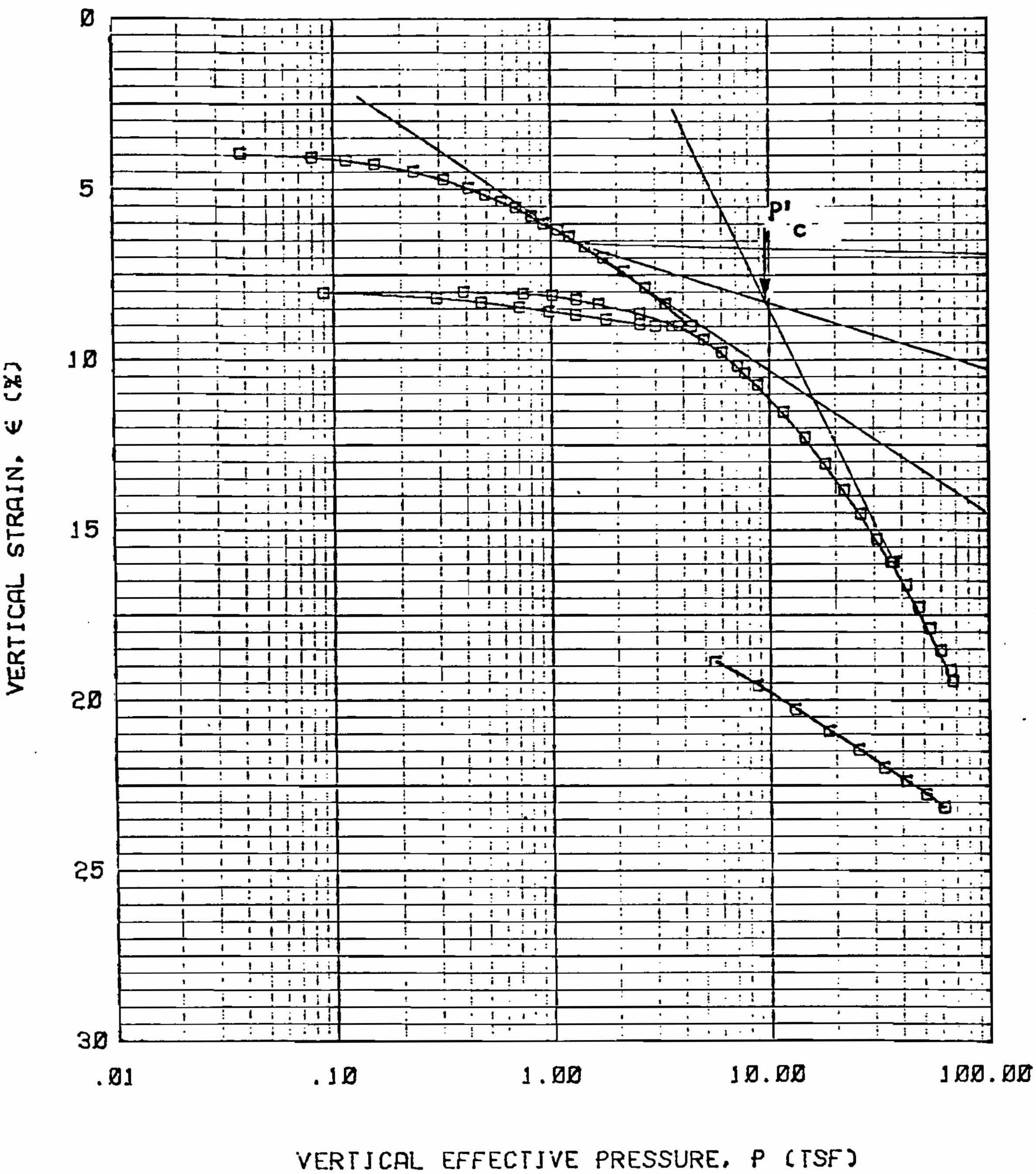
WATER CONTENT: 27 %

LIQUID LIMIT: 37

PLASTIC LIMIT: 21

SPECIFIC GRAVITY: 2.72 (ASSUMED)

INITIAL VOID RATIO: 0.674



CRS THAW - CONSOLIDATION TEST RESULTS

Elapsed Time (min)	Effective Pressure (kPa)	Excess Pore Press. (kPa)	Vertical Strain (%)	Void Ratio	c_v (cm2/day)	Effective Pressure (tsf)
0.0	0.0	0.00	3.893	0.6086	0.0	0.00
5.0	3.7	1.20	3.995	0.6060	0.0	0.04
10.0	7.7	1.90	4.122	0.6048	0.0	0.08
15.0	10.9	1.60	4.224	0.6031	0.0	0.11
20.0	14.6	2.50	4.351	0.6010	1015.6	0.15
30.0	22.3	4.80	4.580	0.5971	528.5	0.23
40.0	30.6	5.10	4.793	0.5937	441.2	0.32
50.0	40.2	6.40	5.038	0.5895	467.0	0.42
60.0	48.0	6.60	5.241	0.5861	313.3	0.50
70.0	57.6	8.00	5.445	0.5827	364.6	0.60
80.0	67.1	8.70	5.623	0.5797	302.4	0.70
90.0	79.1	11.70	5.852	0.5759	345.3	0.83
100.0	90.0	11.80	6.081	0.5720	334.3	0.94
110.0	105.1	12.50	6.259	0.5690	318.0	1.10
120.0	119.0	12.50	6.463	0.5656	278.1	1.24
130.0	140.8	15.10	6.758	0.5605	420.6	1.47
150.0	170.0	14.00	7.073	0.5554	340.4	1.78
170.0	211.5	18.40	7.480	0.5486	334.7	2.21
190.0	264.1	24.20	7.964	0.5405	320.9	2.76
210.0	327.1	26.30	8.422	0.5328	305.3	3.42
250.0	432.6	26.90	9.083	0.5218	335.4	4.52
260.0	348.3	-18.20	9.083	0.5218	0.0	3.64
270.0	294.1	-22.30	9.083	0.5218	650.5	3.07
275.0	249.4	-26.40	9.032	0.5226	615.9	2.60
280.0	174.1	-29.70	8.880	0.5252	1300.0	1.82
290.0	125.4	-27.00	8.753	0.5273	391.2	1.31
300.0	94.5	-25.60	8.651	0.5290	369.8	0.99
310.0	68.1	-27.30	8.524	0.5311	346.5	0.71
320.0	45.4	-27.60	8.371	0.5337	199.3	0.47
330.0	27.9	-27.60	8.244	0.5358	151.2	0.29
340.0	8.6	-30.10	8.091	0.5384	174.1	0.09
350.0	37.5	-10.50	8.091	0.5384	0.0	0.39
360.0	71.4	0.80	8.116	0.5380	0.0	0.75
370.0	98.3	3.10	8.167	0.5371	0.0	1.03
380.0	126.5	5.00	8.295	0.5350	1739.9	1.32
390.0	161.7	7.00	8.422	0.5328	1449.3	1.69
410.0	249.4	11.00	8.702	0.5282	1192.7	2.60
430.0	374.3	17.00	9.083	0.5218	1085.4	3.91
450.0	491.2	24.50	9.455	0.5154	687.0	5.13
470.0	594.1	27.50	9.847	0.5090	469.1	6.20
490.0	702.4	29.10	10.254	0.5022	444.7	7.33
500.0	758.4	29.80	10.457	0.4988	438.2	7.92
510.0	871.6	66.70	10.814	0.4928	649.3	9.10
530.0	1132.8	78.80	11.602	0.4795	415.3	11.83
550.0	1430.7	85.90	12.366	0.4668	405.7	14.94
570.0	1768.2	90.00	13.129	0.4541	419.8	18.46
590.0	2148.0	101.70	13.892	0.4413	421.6	22.43
610.0	2560.2	108.90	14.605	0.4294	415.8	26.74
630.0	3012.3	113.40	15.242	0.4170	422.9	31.46
650.0	3498.4	120.70	16.029	0.4055	426.4	36.53

RESULTS OF CRS THAW-CONSOLIDATION TEST

Boring 6, Kenalooak J-94

190-ft Penetration

Beaufort Sea

McCLELLAND ENGINEERS

670.0	4026.5	125.70	16.691	0.2944	556.9	42.95
690.0	4584.6	134.50	17.253	0.2824	555.1	47.88
710.0	5175.3	138.60	17.963	0.2722	546.4	54.04
730.0	5790.1	138.80	18.599	0.2625	552.4	60.46
750.0	6407.7	142.20	19.184	0.2527	542.9	[illegible]
770.0	6550.5	142.20	19.515	0.2472	122.7	68.40
1050.0	5911.6	-42.70	23.179	0.2859	0.0	61.72
1070.0	4883.5	-52.30	22.822	0.2918	2462.2	51.00
1090.0	3954.8	-62.60	22.441	0.2982	1855.9	41.30
1110.0	3124.0	-76.50	22.024	0.3050	1387.2	32.62
1130.0	2393.9	-91.80	21.525	0.3125	1020.2	25.00
1150.0	1748.1	-112.10	20.966	0.3220	750.1	18.25
1170.0	1226.8	-126.70	20.329	0.3325	528.6	12.81
1190.0	821.8	-151.60	19.642	0.3450	365.1	8.59
1210.0	525.8	-163.90	18.930	0.3570	235.8	5.40

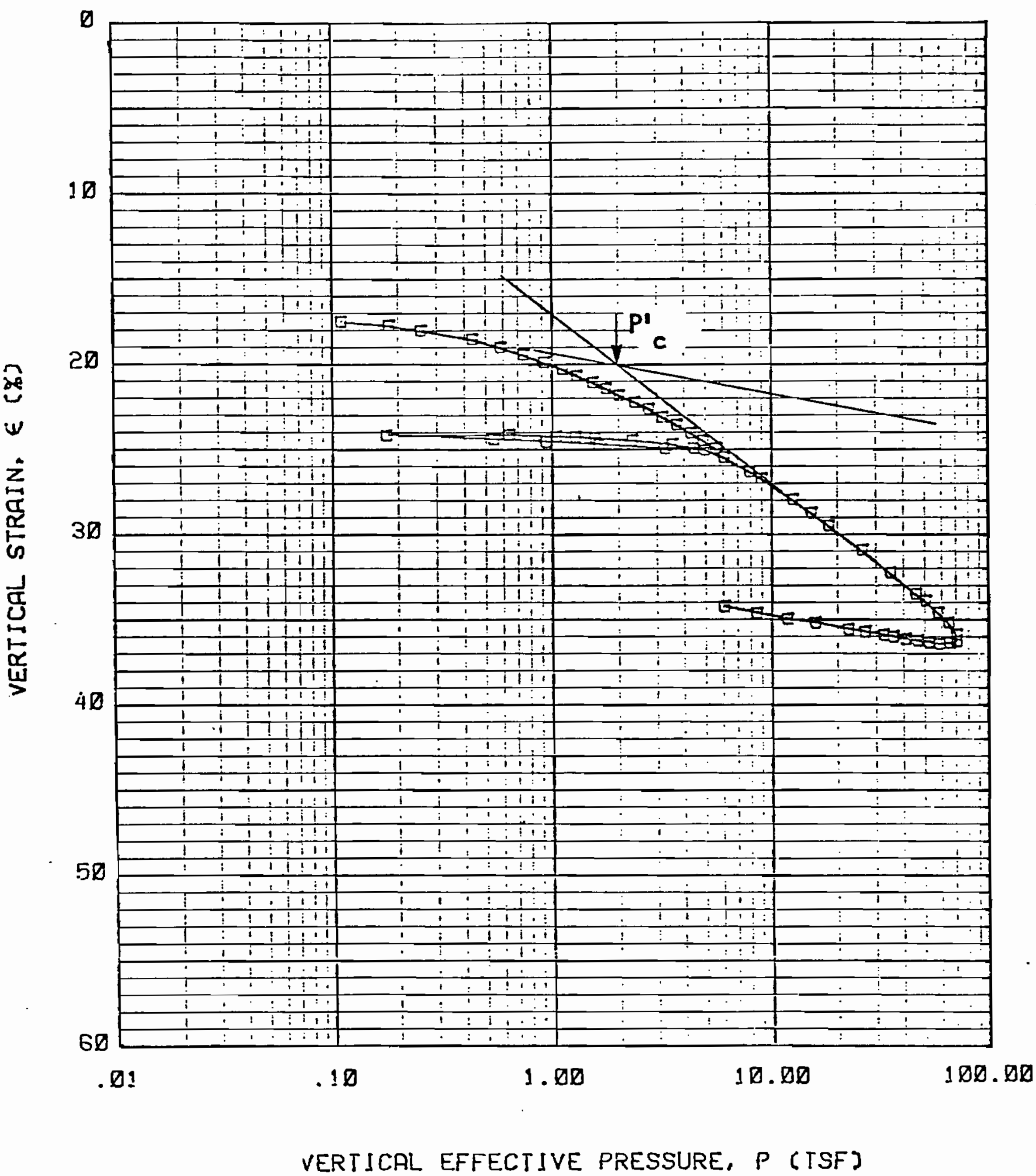
RESULTS OF CRS THAW-CONSOLIDATION TEST

Boring 6, Kenalooak J-94
190-ft Penetration
Beaufort Sea

0179-0128

BORING: KEN PENETRATION: 229.0 FT.
MATERIAL: Grayish brown silt
$P'_c = 2.0$ tsf
$c_c = 0.107$

DRY MASS DENSITY: 91 PCF
WATER CONTENT: 31 %
LIQUID LIMIT: 36
PLASTIC LIMIT: 22
SPECIFIC GRAVITY: 2.72 (ASSUMED)
INITIAL VOID RATIO: 0.980



CRS THAW - CONSOLIDATION TEST RESULTS

McCLELLAND ENGINEERS

PLATE A-4

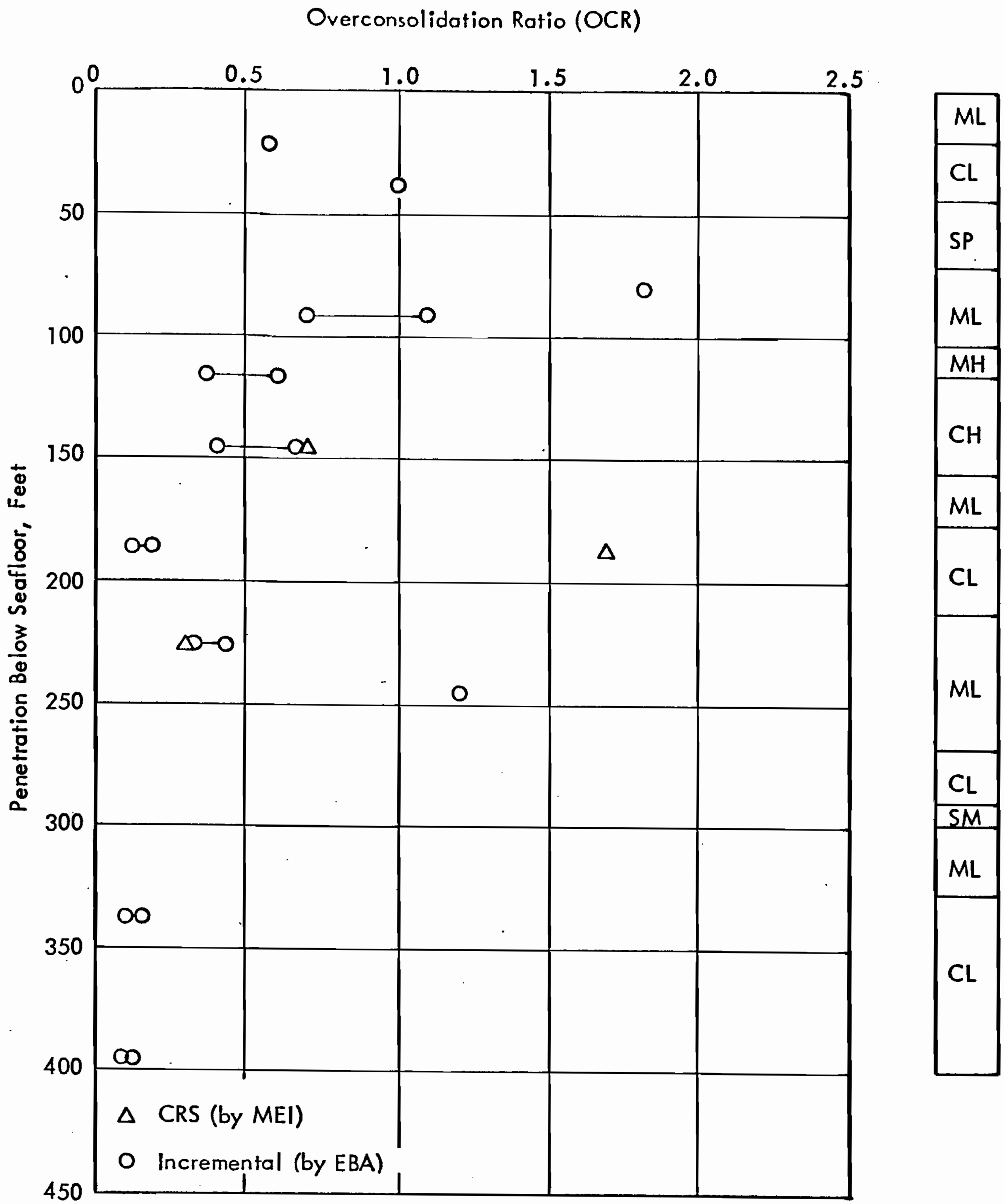
Elapsed Time (min)	Effective Pressure (kPa)	Excess Pore Press. (kPa)	Vertical Strain (%)	Void Ratio	c_v (cm²/day)	Effective Pressure (tsf)
0.0	0.0	0.00	17.582	0.5491	0.0	0.00
10.0	10.5	0.00	17.810	0.5448	0.0	0.11
20.0	17.3	0.20	18.065	0.5400	0.0	0.18
30.0	24.5	0.00	18.345	0.5348	0.0	0.26
40.0	41.5	0.00	18.854	0.5252	0.0	0.43
50.0	56.0	-0.10	19.327	0.5161	0.0	0.58
60.0	71.0	0.70	19.795	0.5075	0.0	0.74
70.0	87.6	0.40	20.228	0.4994	0.0	0.91
80.0	106.1	0.40	20.660	0.4912	0.0	1.11
90.0	123.4	0.70	21.016	0.4846	0.0	1.29
100.0	145.0	0.60	21.423	0.4769	0.0	1.51
110.0	167.0	0.50	21.805	0.4697	0.0	1.74
120.0	192.5	0.50	22.161	0.4630	0.0	2.01
130.0	225.9	0.50	22.594	0.4549	0.0	2.36
140.0	261.5	0.30	23.001	0.4472	0.0	2.73
150.0	305.3	0.50	23.484	0.4382	0.0	3.19
160.0	352.7	0.60	23.942	0.4296	0.0	3.68
170.0	406.8	0.40	24.400	0.4210	0.0	4.25
180.0	466.2	0.50	24.858	0.4124	0.0	4.87
190.0	533.1	0.60	25.316	0.4037	0.0	5.57
195.0	421.6	-0.80	25.316	0.4037	0.0	4.40
200.0	309.1	-0.60	25.316	0.4037	0.0	3.23
210.0	154.2	-0.50	25.087	0.4081	0.0	1.61
220.0	88.4	-0.80	24.884	0.4119	0.0	0.92
230.0	51.4	-0.70	24.706	0.4152	0.0	0.54
240.0	16.7	-1.00	24.477	0.4195	0.0	0.17
245.0	60.7	0.20	24.477	0.4195	0.0	0.63
250.0	98.9	0.00	24.553	0.4181	0.0	1.03
255.0	120.8	-0.60	24.604	0.4171	0.0	1.26
260.0	148.7	-0.50	24.680	0.4157	0.0	1.55
270.0	219.5	-1.10	24.833	0.4128	0.0	2.29
280.0	335.3	-0.30	25.087	0.4081	0.0	3.50
290.0	475.0	0.00	25.444	0.4014	0.0	4.96
300.0	578.9	0.00	25.851	0.3937	0.0	6.05
320.0	757.1	0.20	26.716	0.3774	0.0	7.91
340.0	855.0	0.20	27.123	0.3698	0.0	8.93
360.0	1183.6	1.80	28.344	0.3468	0.0	12.36
380.0	1443.5	2.10	29.133	0.3320	0.0	15.07
400.0	1737.6	4.00	29.896	0.3177	8411.6	18.15
440.0	2453.6	5.90	31.346	0.2904	7158.0	25.62
480.0	3306.0	10.00	32.695	0.2651	5142.8	34.52
520.0	4316.8	14.80	33.942	0.2416	2792.7	45.08
530.0	4756.5	22.60	34.323	0.2344	4222.5	49.67
540.0	5485.7	41.60	35.036	0.2211	4083.3	57.29
550.0	6152.1	44.50	35.646	0.2096	2743.6	64.24
560.0	6649.8	56.20	36.308	0.1971	1745.2	69.44
575.0	6672.2	21.10	36.715	0.1895	0.0	69.68
585.0	6159.8	5.20	36.817	0.1876	0.0	64.32
595.0	5518.2	-1.30	36.893	0.1861	0.0	57.62
605.0	4941.0	-4.40	36.791	0.1881	34629.2	51.60
615.0	4372.9	-6.40	36.690	0.1900	27581.2	45.66
625.0	3847.5	-9.60	36.588	0.1919	17280.7	40.18
635.0	3362.4	-9.40	36.435	0.1948	13474.8	35.11
645.0	3041.3	-8.00	36.333	0.1967	9594.6	31.76
660.0	2505.6	-7.70	36.130	0.2005	12015.6	26.17
670.0	2098.3	-12.60	35.952	0.2038	10729.1	21.91
680.0	1480.4	-26.80	35.596	0.2105	8491.4	15.46
685.0	1106.4	-36.70	35.316	0.2158	6433.2	11.55
690.0	803.3	-42.90	34.985	0.2280	4167.0	8.39
695.0	566.9	-45.70	34.603	0.2292	3056.8	5.92

RESULTS OF CRS THAW-CONSOLIDATION TEST

Boring 6, Kenalooak J-94

229-ft Penetration

Beaufort Sea



OVERCONSOLIDATION RATIO vs PENETRATION
Boring 6, Kenalooak J-94
Beaufort Sea

APPENDIX B

ILLUSTRATIONS

LOG OF SEDIMENTARY STRUCTURE
BORING 6, KENALOOAK
BEAUFORT SEA

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF BEDDING
			Even, parallel
			Even, parallel
			Even, parallel
10			Even, parallel
			Even, parallel
			Even, parallel
20			Even, parallel
			Even, parallel
			Even, nonparallel, discontinuous
			Even, parallel
30			Even, parallel
			Even, parallel and homogeneous
			Even, parallel
40			Even, parallel and homogeneous
			Homogeneous
50			
60			
70			
			Homogeneous
80			Homogeneous
90			
			Homogeneous
			Homogeneous
100			

(Continued on Plate B-1b)

LOG OF SEDIMENTARY STRUCTURE
BORING 6, KENALOOAK
BEAUFORT SEA

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF BEDDING
			Homogeneous
			Homogeneous
110			
			Even, parallel
120			
130			
140			
150			
160			
170			Homogeneous
			Homogeneous
			Curved, parallel, discontinuous
180			
			Curved, nonparallel
190			
			Curved, nonparallel
200			

(Continued on Plate B-1c)

LOG OF SEDIMENTARY STRUCTURE
BORING 6, KENALOOAK
BEAUFORT SEA

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF BEDDING
210			
220			Curved, nonparallel
230			Curved, nonparallel, discontinuous
240			Even, parallel
250			
260			
			Even, parallel
270			
			Even, parallel
280			
290			
			Even, parallel
300			

(Continued on Plate B-1d)