Public Works Canada

Travaux Publics Canada

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Western Region

Region de l'Ouest

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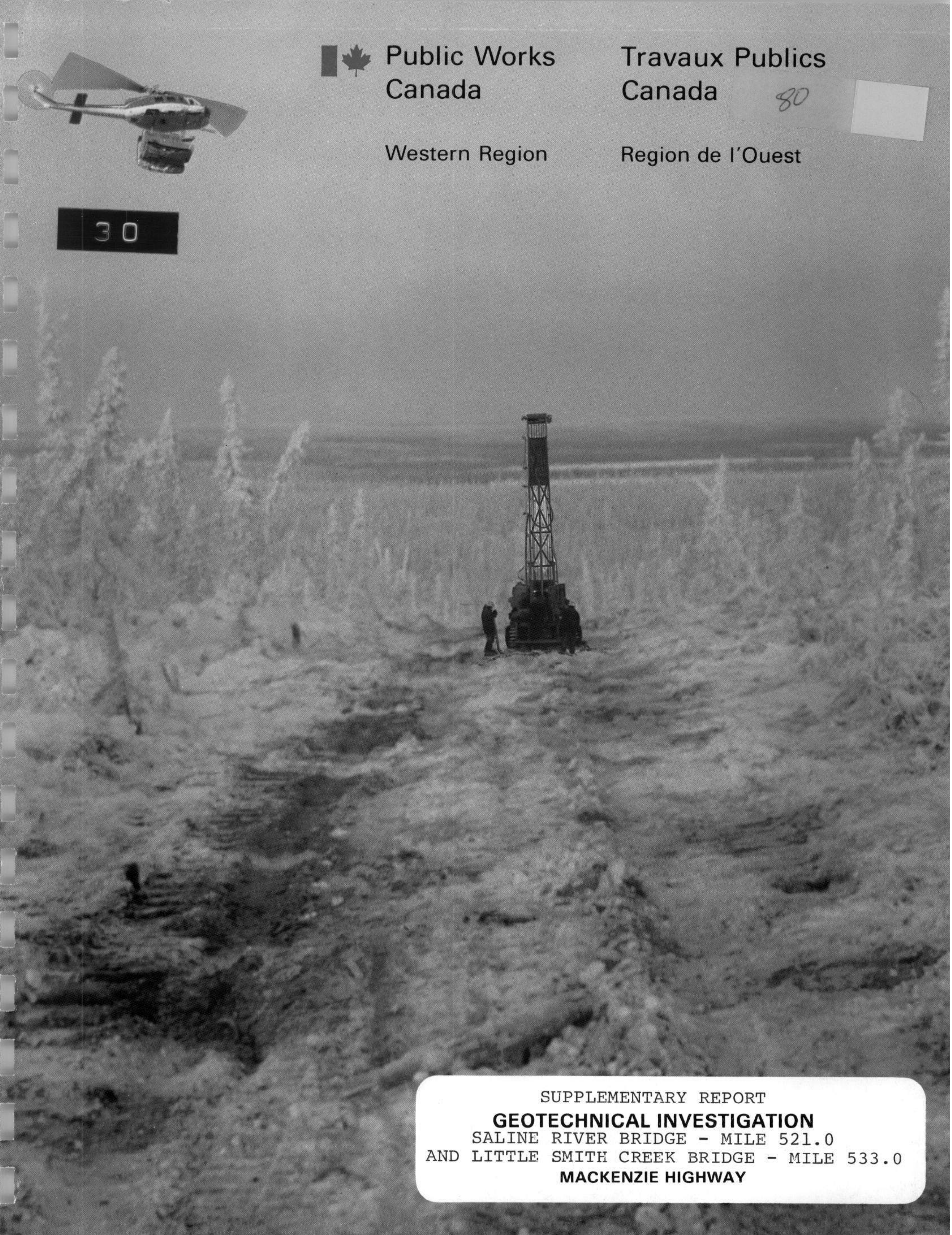

SUPPLEMENTARY REPORT

GEOTECHNICAL INVESTIGATION

SALINE RIVER BRIDGE - MILE 521.0
AND LITTLE SMITH CREEK BRIDGE - MILE 533.0

MACKENZIE HIGHWAY

PUBLIC WORKS CANADA

WESTERN REGION

SUPPLEMENTARY REPORT
FOUNDATION INVESTIGATIONS
PROPOSED BRIDGES
AT
SALINE RIVER, MILE 521.0
AND
LITTLE SMITH CREEK, MILE 533.0

MACKENZIE HIGHWAY

Submitted by

R. D. Cook
Soils Engineer
Special Services
Western Region

February 5, 1976

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APPENDICES

I PRELIMINARY

The initial subsurface investigations at the Saline River (Mile 521.0) and Little Smith Creek (Mile 533.0) were carried out by Underwood, McLellan & Associates during February of 1973, and summarized in foundation reports dated April, 1973. Pertinent excerpts from both reports have been included herein, however this report should be considered as an addendum to the original reports by Underwood, McLellan (UMA) and reviewed in conjunction therewith.

Foundation conditions at both bridge sites are very similar. The streams are incised into dense basal till with bedrock at depths of less than 100 feet. There are relatively thin, (10-20') fluvial, sands and gravels overlying the glacial till in the stream beds. The till is an excellent foundation stratum and UMA have recommended steel piling for the foundation elements at both sites.

Additional test holes were drilled by Public Works Canada at the crossing sites in February, 1975, to augment the initial subsoil data. This additional data and any changes in the foundation recommendations resulting from the additional data are presented on the following pages.

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II SALINE RIVER - MILE 521.0

Two holes were drilled at Saline River - one on either side of the stream channel. A profile of the site showing boreholes from both drilling programmes, and the inferred subsoil stratigraphy has been included on Drawing No. A-1 in Appendix A, along with the borehole logs. The proposed gradeline, and the locations of piers and abutments as recommended by the bridge consultants (Reid, Crowther & Partners Ltd.) have also been shown on the profile.

A. SUBSOIL CONDITIONS AND FOUNDATION RECOMMENDATIONS - UNDERWOOD, McLELLAN & ASSOCIATES

The following 10 pages are taken from the foundation report by Underwood, McLellan & Associates and refer only to the subsoil conditions below the structure and the recommendations for foundation design.

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1. SITE AND SUBSOIL CONDITIONS

The Saline bridge site is located at mile 521 (chainage 1190 + 00) of the Mackenzie Highway in the Northwest Territories. The Saline River is a major river flowing into the Mackenzie River and has a drainage basin which extends to Mt. Clark in the Franklin mountains.

The main Saline River channel is 140 feet deep with stable slopes of 5:1. The main channel elevation at the bridge crossing is 335 (D.P.W. datum) with an estimated high water level during flood stages of 343.

Four test holes were drilled at the stream elevation in the vicinity of the proposed piers and abutments, although three of these holes were drilled from 20 to 265 feet east of the centreline. The three holes drilled adjacent to the centreline disclosed 5 to 15 feet of surface gravel underlain by a very firm grey glacial clay till. The glacial till consisted of a high percentage of large stones and

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its very firm to stiff consistency was indicated by the solid penetrometer blow counts from 26 to 83. Some difficulty was experienced in identifying the soil strata in test hole 355A as a result of caving but it has been classified as gravel with some clay. Large boulders exist at the surface but at greater depths large boulders were not encountered. This test hole, 355A was drilled at the north abutment location and the solid penetrometer blow counts of 42 to 53 indicate the dense nature of the strata. The moisture content of both the glacial boulder clay till and gravel was low averaging 7% to 8% throughout the depth of the test holes. All four test holes indicated non-permafrost conditions to the lowest elevation of 310 which was attained during drilling operations. The glacial till is likely to be of Pleistocene origin with the stream gravel having been deposited since glaciation.

Test holes 356A and 357A drilled on the north approach slopes indicated dry frozen gravel and dense glacial clay till with moisture contents of

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4% and 8% respectively. Test hole 469B near the top of the slope disclosed unfrozen glacial till with an average moisture content of 10 percent.

Two river terraces on the south approach, investigated with test holes 341A and 350A indicated 4 to 7 feet of gravel over clay till. The moisture content of the clay till was approximately 20% and exhibited a dry unit weight of 106.3 pcf. All materials on the south slope were frozen with some random ice. Although the moisture contents of the frozen till is relatively low, it is 10% greater than the unfrozen till found in test hole 469B on the north slope. On the valley slopes the maximum peat depth was four inches.

Free water was not encountered in the test holes and no significant sloughing occurred during air-drilling.

Bedrock materials were not encountered during drilling at the Saline River crossing although Cretaceous sandstones and shales with interbedded coal would be expected.

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2. CONCLUSIONS AND RECOMMENDATIONS

On the basis of the present investigation, we wish to offer the following generalized conclusions and recommendations relative to the design and construction of the proposed Saline River bridge foundation and approaches.

1. Pier and Abutment Foundation Design

The gravel and glacial till deposits which were described in the previous section will form the bearing strata for the proposed bridge piers and abutments. The two foundation types which may be considered include a spread footing or pile system.

Pier bases should be placed at approximately elevation 325 in the dense glacial clay till and the allowable footing bearing capacity would be 6000 psf in this material. In order to attain the elevation of the glacial till approximately 10 feet of overburden gravel must be removed.

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Although a footing type foundation may be employed at this site, driven piles are preferred. The pile types available include timber, pre-cast concrete, steel H-piles and pipe piles. Timber piles may be utilized but tip protection would be necessary to drive through the gravel strata into the glacial till. Timber piles driven to "refusal" would attain allowable load bearing capacities near 40 tons but this would depend upon the pile size and energy rating of the pile driving hammer. The timber piles would be expected to attain "refusal" at a depth of approximately 15 feet from the surface.

The pre-cast concrete piles may be utilized but splicing is difficult although at this site the "refusal" depth will be in the range of 15 to 20 feet below existing channel grades, unless significant variations in the soil profile exist across the stream channel.

Steel H-piles or pipe piles are recommended

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for consideration based upon high driving strength, high load capacity and ease in splicing and cutting. At the Saline River site, the steel pipe or H-piles will meet refusal in the dense till at approximately the 30 foot level and the allowable capacities will be in the range of 80 tons. The capacity will depend upon the structural shape and material as well as the tip protector utilized. Wherever "refusal" is not attained in the glacial till or gravel deposits, the capacity of the pile may be estimated on the basis of the skin friction between the soil strata and pile. Approximate allowable capacities for long steel friction piles may be determined by using allowable unit adhesion of 600 psf in the dense glacial clay till and 800 psf in the gravel strata. In order to attain penetration of the steel piles around large boulders, it may be necessary to utilize vibration techniques.

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It is recommended that all piles be driven to "refusal". "Refusal" will depend upon the energy rating of the hammer but is commonly 15,000 ft. lb. As an initial guide, piles should be driven to blow counts of 180 blows/ft. or 15 blows/inch. The pile capacity is largely a function of the amount of energy expended in installing the pile and not just of the recorded resistances. The pile which is driven to a sustained resistance will perform better than one which is terminated the instant a given resistance is attained. Of course, the pile must not be driven until damage occurs and whenever resistance increases greatly, the driving should be terminated.

The capacity of a pile should be established in the field by several pile-driving tests. The ultimate capacity would be calculated on the basis of a dynamic pile driving formulae such as the Hiley, Danish or Weisbach. It is recommended that the Engineering News formula

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not be utilized as a result of its extreme variation in factor of safety.

It is further recommended that static load tests be performed to establish more accurately the bearing capacity of a typical pile and the applicability of dynamic pile formulae for predicting pile capacity. It is emphasized that load tests at this site are important to establish pile capacity as driving formulae can only be considered a rough guide for piles in cohesive till soils.

The lateral resistance of piles can be established by recently developed methods based on beam on elastic foundation methods but the simple and more conventional approach is based on calculation of the Rankine passive earth pressure against the pile. Using Rankine theory the ultimate passive resistance force/ft.^{2} of pile can be obtained by $P = \frac{1}{2} \gamma_b K_p H^2$. The submerged unit weight of the gravel and till material may be taken as 60 pcf and the passive pressure coefficient K_p as 3.0. In order to establish the allowable lateral load a factor of safety of 2.5 should be applied to the above ultimate load. Generally, the lateral load on deep piles tends to exceed the passive resistance, consequently, the above simplified approach is on the conservative side.

Whenever the deep embankments are constructed of compacted granular fill as outlined and recommended in section D3, the abutments may be placed on spread footings in the fill. The allowable bearing pressure for the abutments will depend upon the characteristics of the fill material utilized but for initial design, 4000 psf may be used assuming a coarse compacted gravel is used. If inferior material or compaction techniques are utilized in the embankment, the abutment must be carried on piles into the insitu granular subsoils below the creek channel. Piles must be designed to

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carry a negative skin friction load whenever embankment fills are subject to settlement.

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B. EVALUATION OF ADDITIONAL BOREHOLE DATA

With reference to Drawing No. A-1 in Appendix A, the 1975 boreholes confirm the subsoil stratigraphy as inferred from the 1973 boreholes. (Note - several of the 1973 holes by UMA were upstream of the crossing site)

Hole #1 (1975) near the south abutment was extended to a depth of 60' where bedrock (shale) was encountered. Hole #2 (1975) near the north abutment was terminated at a depth of 30 feet in the dense glacial till. Permafrost was reported in hole #1 to a depth of 18' - none of the holes by UMA reported permafrost.

C. FOUNDATION RECOMMENDATIONS

Underwood, McLellan & Associates have recommended steel piling and this is concurred with. The base of the piers and abutments will be relatively near the surface of the glacial till thus there should be little problem in penetrating the coarse fluvial deposits with piling - hence either steel H-piles or steel pipe piles could be employed.

UMA have indicated that either steel pipe or steel H-piles would meet refusal in the dense till at a penetration of 30 feet, and the allowable pile capacity will be in the

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range of 80 tons. This is considered slightly optimistic, especially for steel H-piles, and it is recommended the piles be designed on the basis of an allowable unit adhesion value of 450 p.s.f. in the till (vs. 600 psf. by U.M.A.), plus an allowable end-bearing of 15 kips per square foot.

The pile section used should be moderately heavy to ensure penetration of the upper granular deposits - 12BP53 H-piles or 10.75" pipe piles @ 40 lbs./ft. may be employed. The pile hammer should have a ram at least as heavy as the piling in order to impart relatively high energy - low velocity blows to the piling - an energy of roughly 20,000 ft. lbs. per blow is recommended.

Underwood, McLellan have recommended a pile load test which would have merit as the results could be applied to both the Saline crossing and the Little Smith crossing. However it is considered the adhesion values recommended will provide a safe rational pile loading, and, because of the isolated location, there would be little if any net saving resulting from a load test.

Detailed driving records should be obtained for all piles with particular attention to pile lengths, penetrations, marked changes in blow counts and the number of blows for the last 10-15 feet.

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On pages 15 and 16 of the original report by U.M.A. are recommendations for lateral load capacity for foundation piles (those pages not included herein). The equation proposed for calculating lateral resistance should not be applied for pile penetrations in excess of approximately 15 feet. The allowable lateral load per pile may be assumed at 6 kips for a lateral movement in the order of 1/2 inch.

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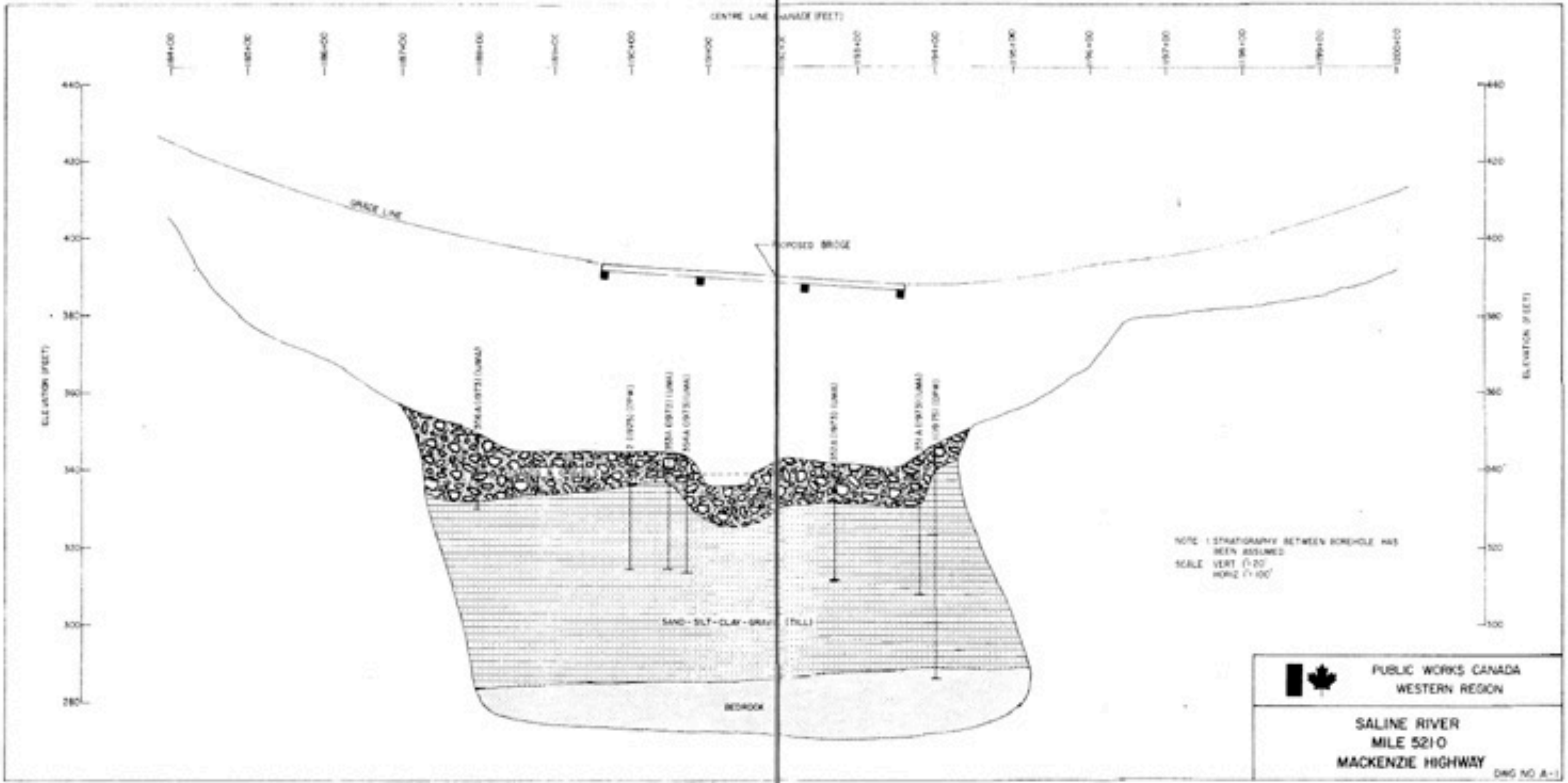

NOTE 1 STRATIGRAPHY BETWEEN BOREHOLE HAS BEEN ASSUMED

SCALE VERT 1"-20'
HORIZ 1"-100'

PUBLIC WORKS CANADA
WESTERN REGION

SALINE RIVER
MILE 521·0
MACKENZIE HIGHWAY

DWG NO A-1

III LITTLE SMITH CREEK - MILE 533.0

Two holes were drilled at Little Smith Creek - one on the south side of the stream channel and one on an island within the channel. A profile of the site showing boreholes from both drilling programmes, and the inferred subsoil stratigraphy has been included on Drawing No. B-1 in Appendix B, along with the borehole logs. The proposed gradeline, and the locations of a single pier and the abutments, as recommended by the bridge consultants (Reid, Crowther & Partners Ltd.) have also been shown on the profile.

A. SUBSOIL CONDITIONS AND FOUNDATION RECOMMENDATIONS - UNDERWOOD, McLELLAN & ASSOCIATES

The following 11 pages are taken from the foundation report by Underwood, McLellan & Associates and refer only to the subsoil conditions below the structure and the recommendations for foundation design.

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1. SITE AND SUBSOIL CONDITIONS

The Little Smith Creek is located at mile 533.7 (chainage 520 + 00) of the Mackenzie Highway in the Northwest Territories. The Little Smith Creek flows into the Mackenzie River from the east having a drainage basin whose origin is in the Franklin Mountains. This stream exhibits a typical sand-gravel bed which is depositing floodplain sediments at the entrance to the Mackenzie River. The main valley is pre-glacial and valley fillings in the form of glacial till were deposited during the Pleistocene Epoch. Subsequently, various levels of the creek have allowed deposition of sands, silty sands and gravels on the valley slopes and in the present main flow channel.

The Little Smith Creek bed elevation is 215 (D.P.W. plan-profile datum) with the recently recorded high water mark at elevation 223.

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The main stream channel across which the creek has flowed in recent times is approximately 700 feet wide and 20 feet deep but at the present time the flow is near the south bank. Gravel and sand terraces occur immediately above the south bank and extend 1000 feet, whereas, on the north a terrace 600 feet long is present approximately 20 feet above the channel level.

Four test holes, 401A 402A, 403A and 404A were drilled across the 700 foot wide channel. All four test holes disclosed very stiff glacial clay till underlying surface deposited sands and gravels. Test holes 401A and 402A were drilled at the south and north abutment locations, respectively, and indicated 20 feet of coarse gravel, boulders and sand. These two test holes confirmed a consistent glacial till elevation of 195 at the proposed bridge location. Test hole 401A which was drilled to a depth of 60 feet did not encounter permafrost conditions, nor did the other three drill holes. Solid standard penetration blow counts from 18 to 91 were obtained in the stiff glacial till with an average count of 62 blows. Sloughing of the surface sands and gravels allowed only disturbed air returned samples to be recovered, consequently, approximate average moisture contents in the glacial till of 16% were obtained. This moisture content is near the plastic limit for the glacial till. The free water table was present at 10 feet in the gravel strata and presented severe caving conditions. Solid penetrometer blow counts in the gravel varied from 23 to 112. This range in blow count may be indicative of material variations in the surface stream deposited soils.

Test holes 399A and 400A were drilled on the south terrace at stations 531 + 00 and 525 + 00, respectively. Test hole 399A encountered 3 feet of silty clay underlain by frozen gravel with moisture contents of 5%. This test hole exhibited a similar stratigraphic sequence to that encountered across the main creek channel except 6 feet of silty clay overlies the clay till.

Near the south uplands, surface gravel and sands were found above silty clay. Permafrost was found throughout test hole 397A but test hole 398A had seasonal frost to 7 feet and then was unfrozen to the 22 foot depth where permafrost was again encountered.

On the north slopes dry sand and gravel was indicated by test borings 405A and 406A. The dry nature of the sand in test hole 405A produced extreme caving conditions.

A soil profile of the bridge valley site is included in the appendix on Plate 2.

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2. CONCLUSIONS AND RECOMMENDATIONS

On the basis of the present investigations, we wish to offer the following generalized conclusions and recommendations relative to the design and construction of the proposed Little Smith Creek bridge foundation and approaches.

1. Pier and Abutment Foundation Design

The gravel and glacial clay deposits which were described in the previous section will form the bearing strata for the proposed bridge piers and abutments. Consideration may be given for either a spread footing pier foundation or driven piles. If a pier-footing type foundation is selected, the base must be placed below the potential scour depth. It is recommended that the piers be founded in the glacial till at elevation 195 approximately 20 feet below the average existing site grades. The allowable bearing capacity for footings in undisturbed stiff till would be 6000 psf. Excavation for

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piers and footings will take place under caving conditions in the lower saturated gravels and will necessitate cofferdams and pumping.

Driven piles are recommended in preference to the footing type foundation, primarily based upon excavation difficulties in the gravel strata. The pile types available include timber, precast concrete, steel H-piles and pipe piles. The timber piles are not recommended as a result of their low capacity and possible damage when driving through gravel strata. The pre-cast concrete piles are also not recommended primarily due to the difficulty in establishing ultimate pile lengths and unless the lengths can be predetermined considerable difficulty in splicing results.

Steel H-piles or pipe piles are, therefore, recommended for consideration primarily based on high driving strength, high load capacity and relative ease in splicing. The granular strata

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and glacial till at this site will provide adequate lateral support such that instability in the form of buckling of the piling will not be a problem.

It is recommended that all piles be driven to "refusal". "Refusal" will depend upon the energy rating of the hammer but is commonly 15,000 ft. lb. As an initial guide, piles should be driven to blow counts of 180 blows/ft. or 15 blows/inch. The pile capacity is largely a function of the amount of energy expended in installing the pile and not just of the recorded resistances. The pile which is driven to a sustained resistance will perform better than one which is terminated the instant a given resistance is attained. Of course, the pile must not be driven until damage occurs and whenever resistance increases greatly, the driving should be terminated.

Steel piles driven to "refusal" can be expected to attain allowable load capacities in the range of 80 tons depending upon the cross-sectional area. False "refusal" may occur whenever extremely large boulders cannot be penetrated during driving. Although "refusal" may be attained while driving in boulders, adequate bearing capacity may not exist below the pile tip. Load tests would be required to establish allowable loads if this situation is recognized. In order to attain penetration of the steel piles around large boulders, it may be necessary to utilize vibration techniques or large diameter open pipe piles may be employed with churn-drill crushing of large boulders inside the bottom of the pipe. It is anticipated that penetration of the gravel overburden will be achieved with final refusal occurring at a depth of 30 to 40 feet below existing site grades.

Difficulty exists in establishing the depth at which refusal will be realized as no significant

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increasing blow count with depth was noted in the glacial till.

Conventional pile-driving tests on piles driven into the glacial clay till can only be utilized as an approximate guide in establishing load capacity. In primarily cohesive soils, the penetration resistance does not increase with depth, although load-carrying capacity of the pile is constantly increasing with depth. The common Engineering News formula should not be utilized as recent studies have indicated extreme variations in factor of safety.

Whenever piles are driven to great depths, the allowable load may be approximately determined by the adhesion between the soil and pile based on 600 psf. When "refusal" is attained, the allowable load should be based upon the pile material strength.

It is recommended that static load tests be performed to establish more accurately the

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bearing capacity of a typical pile.

The lateral resistance of piles can be established by recently developed methods based on the beam on elastic foundation method but the simple and more conventional approach is based on calculation of the Rankine passive earth pressure against the pile. Using Rankine theory the ultimate passive resistance force/ft.^{2} of pile can be obtained by $P = \frac{1}{2} \gamma_b K_p H^2$. The submerged unit weight of the granular material and glacial till may be taken as 60 pcf and the passive pressure coefficient K_p as 3.0. In order to establish the allowable lateral load a factor of safety of 2.5 should be applied to the above load.

Whenever the embankments are constructed of compacted granular fill or unfrozen glacial till as outlined and recommended in Section D3, the abutments may be placed on spread footings in the fill. The allowable bearing stress for the

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abutments should be established when the characteristics of the fill are known. If inferior materials or compaction techniques are utilized in the embankment, the abutment must be carried on piles into the insitu glacial till subsoils below the creek channel. Piles must be designed to carry a negative skin friction load whenever embankment settlement is anticipated.

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B. EVALUATION OF ADDITIONAL BOREHOLE DATA

With reference to Drawing No. B-1 in Appendix B, the 1975 boreholes confirm the subsoil stratigraphy as inferred from the 1973 boreholes. However there may be some discrepancy regarding the presence of permafrost.

None of the holes by U.M.A. within the present channel, or on the adjacent flood plain, reported permafrost. However hole #1 by D.P.W. on the south channel bank, and 25'-30' south of the proposed abutment location, indicated frozen ground to the drilling depth of 43'. It is entirely possible that there is a thaw zone only below the channel, as investigations at several stream crossings in permafrost terrain have revealed such limited thaw areas which are confined directly below the width of flow. Thus it is possible that Holes 401A, 402A, 403A and 404A (UMA) within the channel were in a thaw zone, and Hole #1 (DPW) on the channel bank was in permafrost. However it is believed that there is permafrost in the area of Hole 400A (400' south of the channel), although none was reported by U.M.A. Permafrost was encountered throughout the south valley wall and adjacent to the stream channel by D.P.W. thus it is considered likely permafrost is present below the floodplain in the area of Hole 400A.

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The subsoil profile in the area of the proposed structure consists of 15-20 feet of sands and gravels over glacial till which extends to depths of at least 60 feet, and is likely underlain by bedrock at less than 100 feet. The glacial till is an excellent bearing stratum - it is very dense with moisture contents below the plastic limit even in the permafrost zone.

C. FOUNDATION RECOMMENDATIONS

Underwood, McLellan & Associates have recommended steel piling and this is concurred with. Open-ended steel pipe piles are considered the most practical for this site due to the presence of cobbles and boulders in the upper granular stratum, and to the possibility of encountering permafrost near the south abutment.

A heavy section - at least 10" diameter @ 40 lbs./ft., should be used to withstand hard driving which may be required to penetrate the granular stratum, and to gain adequate penetration into the till if permafrost should be encountered. The hammer used should have a ram at least as heavy as the piling in order to impart relatively high energy - low velocity blows to the piling - an energy of roughly 20,000 ft. lbs. per blow is recommended.

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It is considered that open-ended steel pipe piling will provide a safe foundation in the dense glacial till regardless of the thermal condition. Since the moisture (ice) content of the till is very low, post-construction thawing around piling driven into frozen till is not expected to produce bearing parameters that are significantly less than the bearing parameters for similar piling driven into unfrozen till. The problem if permafrost is encountered may be in obtaining sufficient pile penetration into the frozen soils to ensure an adequate safety factor if thawing does occur. There is a lack of construction information regarding pile driving into frozen soils, but any experiences reported with conventional driving equipment (i.e. drop hammers, diesel hammers) indicate extreme difficulty, pile damage and generally negative results. In addition,a vibratory hammer would most probably not perform well in this dense clay-till subsoil. The Bodine driving equipment based on resonance principle has apparently shown some success in permafrost soils,and by the time of highway construction in the Little Smith area may have proven effective in pile driving into permafrost.

Underwood, McLellan have recommended an allowable adhesion value of 600 p.s.f. for steel piling, but have also estimated a pile penetration of 30 to 40 feet to refusal in unfrozen till

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below existing grade, wherein the allowable pile load could be based upon the structural column strength. Since the possibility of some permafrost does exist at this site, and 'false' or 'short-term' refusal may result in the frozen ground, it is recommended the pile loadings and lengths be based only upon pile adhesion. The adhesion value of 600 p.s.f. recommended by U.M.A. is considered slightly high for an average value over the length of a straight sided pile in the very stiff till, as the effects of driving may create an annulus around the pile and the stiff till will not 'flow back' around the pile. Thus an average value of 450 p.s.f. is recommended. Should permafrost be encountered (and confirmed) during driving and it becomes practically impossible to conventionally drive the piles to the design penetration, then the piles may be advanced by a combination of pre-drilling a lead hole 1/2 to 1" in diameter less than the pile and driving, and/or drilling ahead of the pile through the central core. Pile installation should be under close inspection and driving trials may be required to ensure that pre-drilling does not exceed acceptable limits and pile bearing is effected. As an initial requirement it is recommended the lead hole be limited to 75% of the design pile penetration.

Detailed driving records should be obtained for all piles with particular attention to pile lengths, penetrations, marked changes in blow counts, and the number of blows for the last 10-15 feet.

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Underwood, McLellan have recommended pile load test, however this is considered unnecessary providing there is a competant inspector on site during pile installation.

On page 11 of this report are recommendations by U.M.A. regarding the lateral resistance of piles at this site. The equation proposed should not be applied to pile penetration in excess of roughly 15 feet. The allowable lateral load per pile may be assumed at 6 kips per pile for a lateral movement in the order of 1/2 inch.

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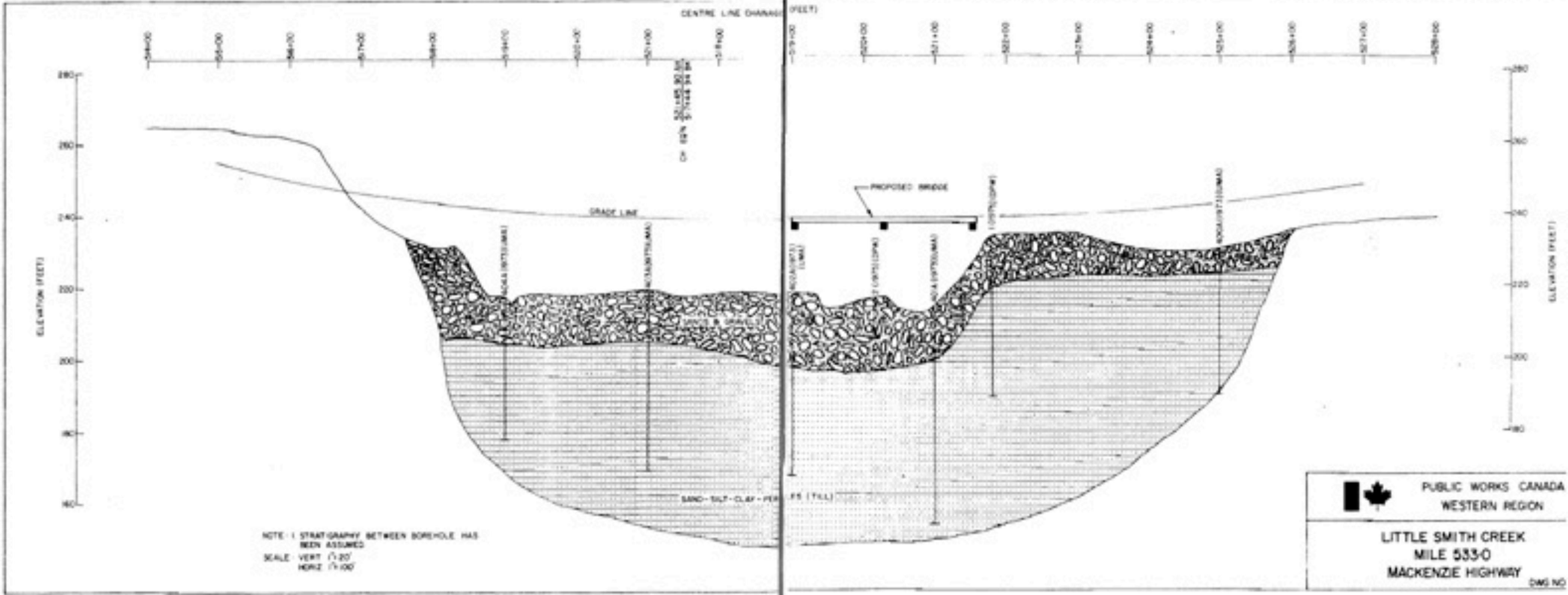


NOTE : I STRATIGRAPHY BETWEEN BOREHOLE HAS BEEN ASSUMED

SCALE : VERT 1"=20'
HORIZ 1"=100'

PUBLIC WORKS CANADA
WESTERN REGION

LITTLE SMITH CREEK
MILE 533·0
MACKENZIE HIGHWAY

DWG NO

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

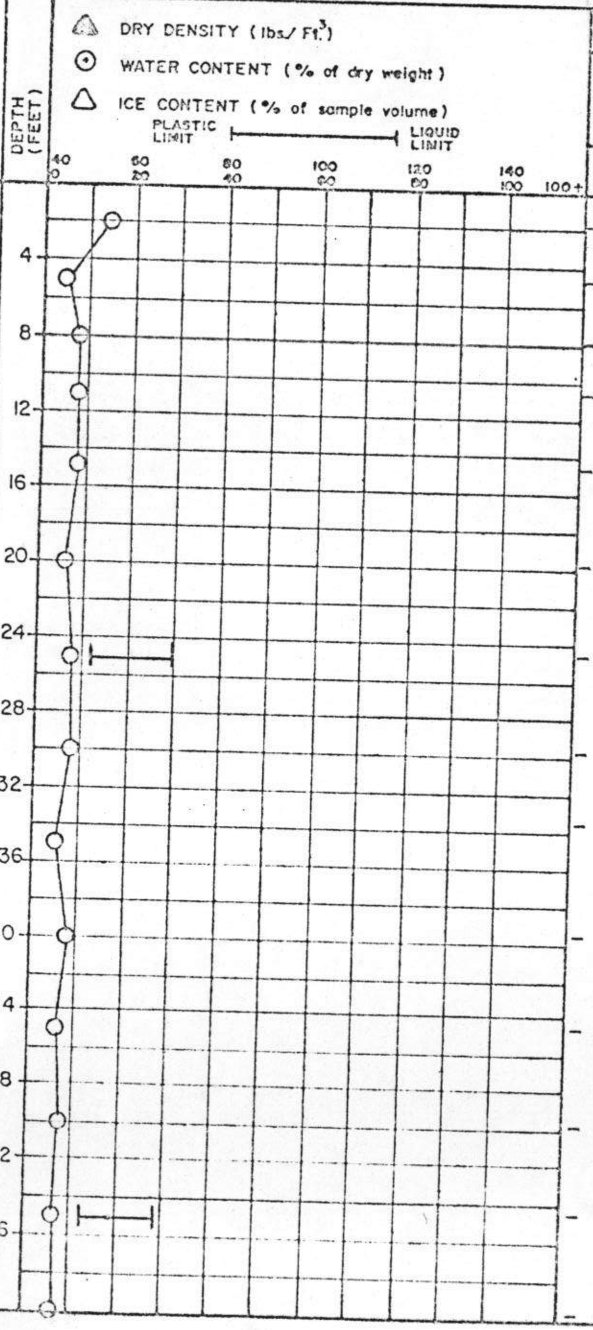
DRILL HOLE REPORT

SITE: SALINE RIVER

FIELD ENG.
DATE DRILLED. 13/3/75
AIRPHOTO NO.
CHAINAGE.
OFFSET. 30' LEFT
TECH. REYNOLDS
RIG. AIR
SURFACE DRAINAGE.
VEGETATION.
ELEV.
TEST HOLE #1 (1975)
MILE

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
4					SM	Sand-Silty Gravelly		
6'								
8						Clay-Silty Sandy Pebbles	F	Nbn V_x
12								
16					Cl	- TILL Medium Plastic Gravelly		
20								
24								
28					CL	Clay-Silty Sandy Gravelly Low Plastic		
32								
36								
40								
44								
48								
52						Clay-Silty Sandy Pebbles		
56					CL	Low Plastic Shale Bedrock @ 59'		

Bottom of Hole - 60'



GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	RELATIVE THAWED MOISTURE CONTENT	REMARKS
24	-	61	15	Wet	
12	-	61	27	Damp	
53	-	38	9	Damp	
57	-	36	7	Damp	
59	-	35	6	Damp	
50	-	32	18	Damp	
56	-	33	11	Damp	
58	-	32	15	Damp	
59	-	35	6	Humid	
60	-	31	9	Damp	
54	-	32	14	Damp	
59	-	29	12	Damp	
61	-	31	8	Damp	
60	-	93	7	Damp	

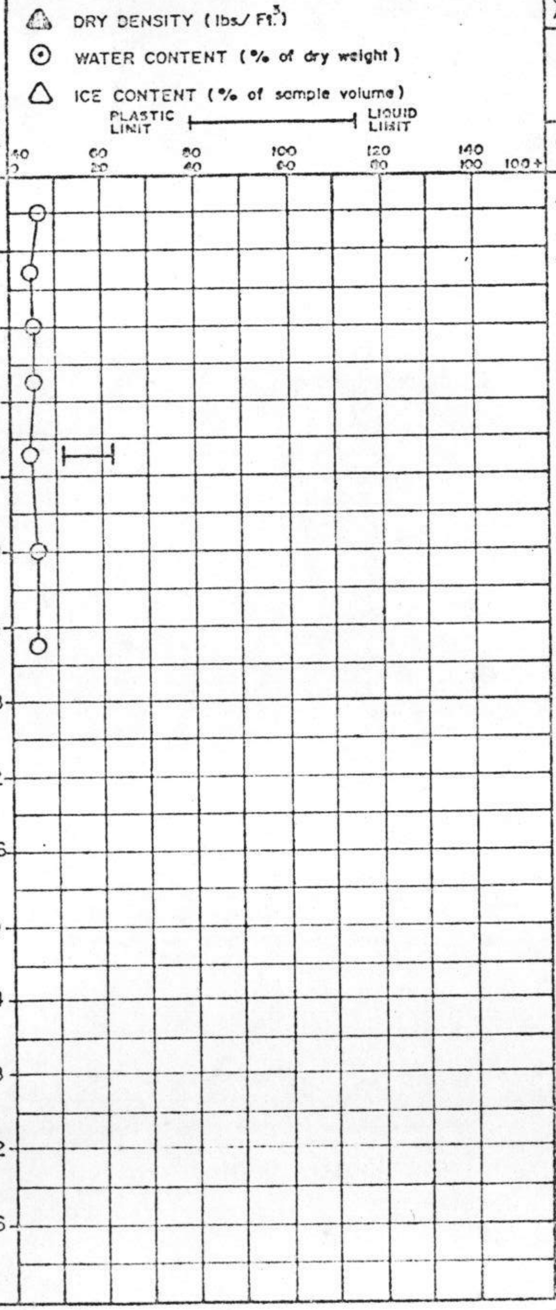
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DRILL HOLE REPORT

SITE: SALINE RIVER

FIELD ENG.
DATE DRILLED. 13/3/75
AIRPHOTO NO.
CHAINAGE.
OFFSET.
TECH. REYNOLDS
RIG. AIR
SURFACE DRAINAGE.
VEGETATION.
ELEV.
TEST HOLE #2 (1975)
MILE

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0–8					SM	Sand-Silty Gravelly Clayey 8'	F	V_x
8–30					CL	Clay-Silty Sandy Gravelly Low Plastic Pebbles -Till		
30						Bottom of Hole - 30'		



GRAIN-SIZE ANALYSIS

CLAY %	SILT %	SAND %	GRAVEL %	RELATIVE THAWED MOISTURE CONTENT	REMARKS
-	28	58	14	Damp	
-	14	61	25	Damp	
-	40	31	29	Damp	
-	34	25	41	Damp	
-	49	36	15	Damp	
-	58	34	7	Damp	
-	59	32	9	Damp	

D3·14·5·74 a

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN A EW
FIELD ENG NWC
DATE DRILLED 6/2/73
AIRPHOTO NO:
CHAINAGE: 1193+80
OFFSET: 20' E OF ℄
CKD DSP
TECH:
RIG: AIR
SURFACE DRAINAGE: GOOD
VEGETATION:
ELEV.

TEST HOLE

MILE	B,C,S	NUMBER
521	S	351A

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS
		⧄				GRAVEL	UF								
4		⧄			SW	SAND WITH COBBLES AND BOULDERS, DRY									
8															
12		⧄			GP	WET									WATER COMING OUT OF HOLE
16		⧄			GC	CLAY GLACIAL TILL									
20		⧄				DARK BROWN WITH SAND & GRAVEL									
24		⧄				BOULDERS									
28		⧄													
32		⧄			SC										
36						END OF HOLE									
40															
44															
48															

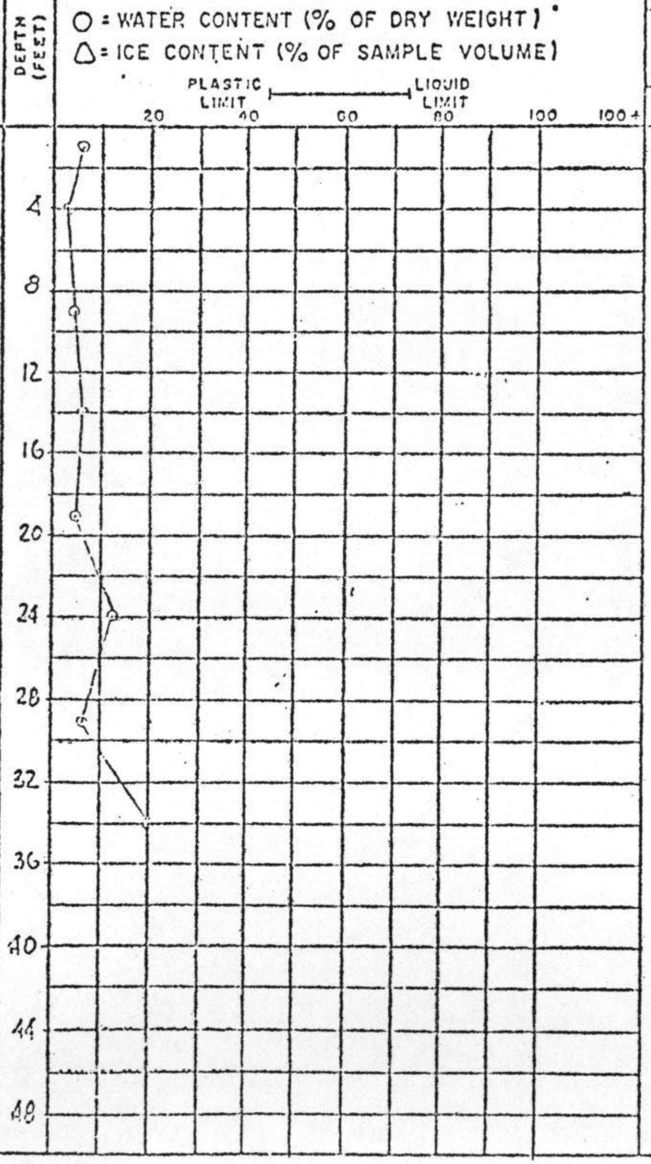


PLATE 6

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

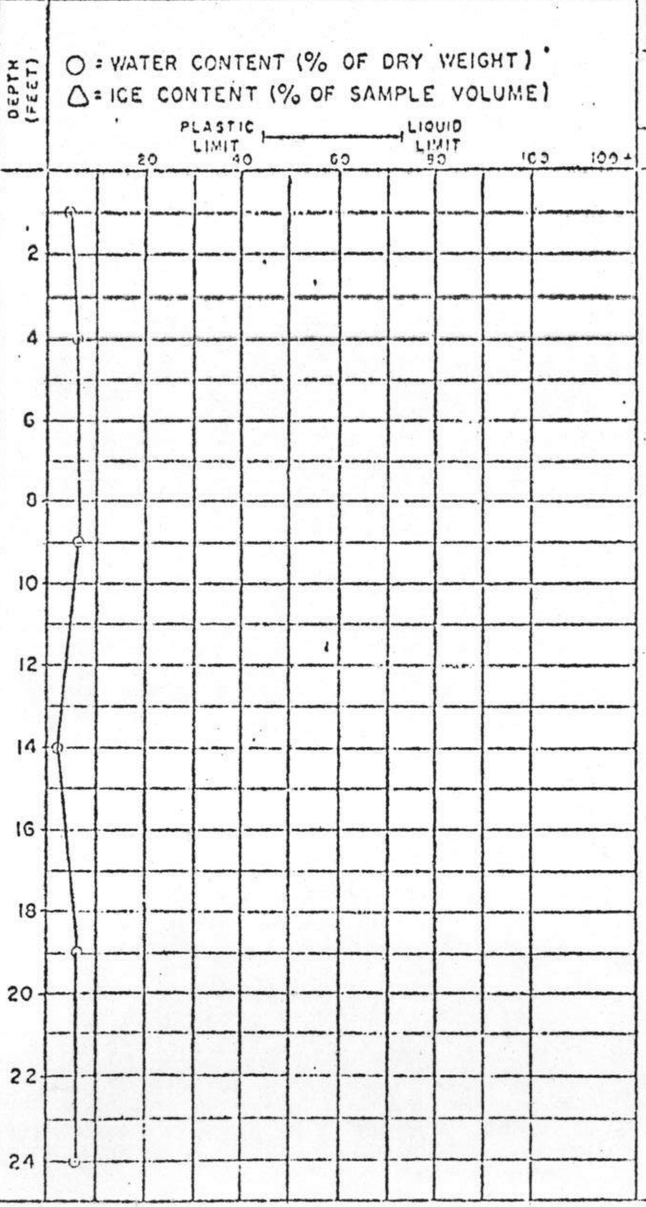
[illegible] | FIELD ENG AWC | DATE DRILLED 6/2/73 | AIRPHOTO NO: | CHAINAGE: 1192+70 | OFFSET: 130' E of ℄
[illegible] | TECH | RIG: AIR | SURFACE DRAINAGE: GOOD | VEGETATION: N/L | ELEV.

TEST HOLE

MILE	B,C,[illegible]	NUMBER
521	S	352

REMARKS

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
0						GRAVEL	UF	
2–8					GP	SAND FAIRLY DRY COBBLES AND BOULDERS		
9				26 BLS				
10					GC	CLAY GLACIAL TILL		
14–16						GRAVEL & SAND SOME MOISTURE COBBLES & BOULDERS		
19				44 BLS				
22					GC	WET		
24						END OF HOLE AT 30'±		



GRAIN-SIZE ANALYSIS				WET DENSITY (PCF)	DRY DENSITY (PCF)
CLAY	SILT	SAND	GRAVEL		
%	%	%	%		

PLATE 7

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

[illegible] FIELD ENG. /. NC DATE DRILLED 6/2/73 AIRPHOTO NO: CHAINAGE: 1190+50 OFFSET. 265' E OF ℄

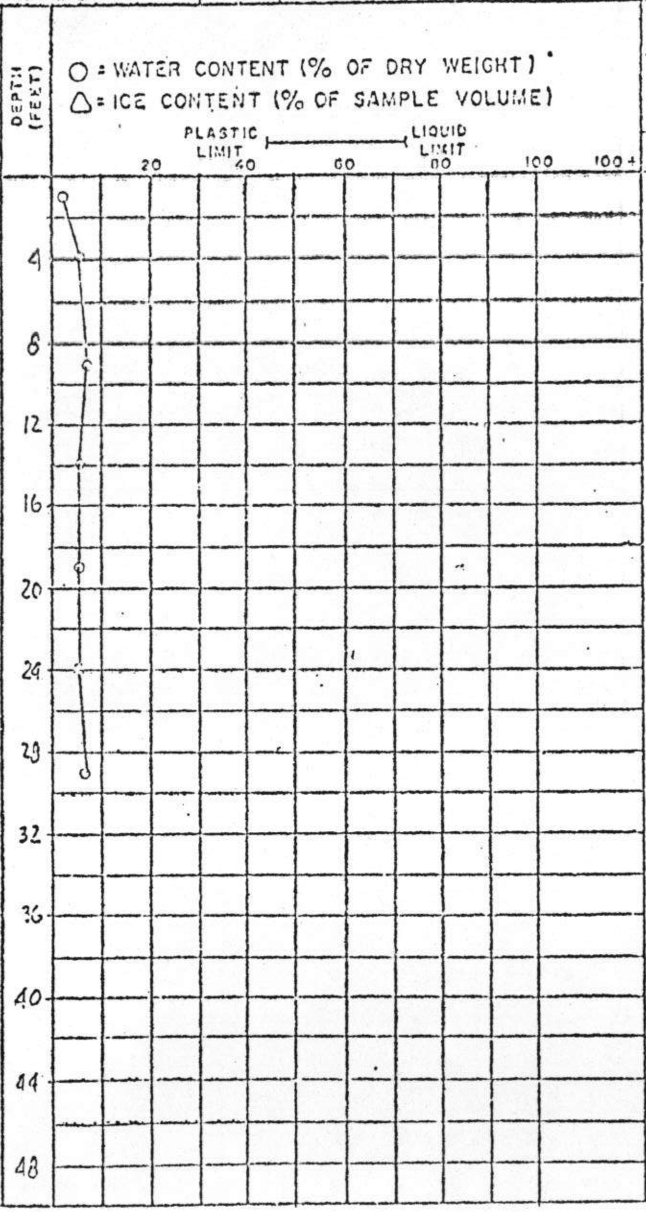
CKD TECH: RIG: AIR SURFACE DRAINAGE: GOOD VEGETATION: NIL ELEV.

TEST HOLE

MILE	B.C.S.	NUMBER
521	S	353A

REMARKS

DEPTH (FEET)	SAMPLE NO.	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
						GRAVEL	UF	
4					GP	SAND, BOULDERS. ROCK FRAGMENTS		
8					GC	CLAY TILL		
12				48 BLS		SAND, GRAVEL AND BOULDERS		
16						FINE, HARD, DRY		
20				83 BLS.	GC			
24								
28				80 BLS	SM	SILTY, SAND		
32						END OF HOLE		
36								
40								
44								
48								



GRAIN-SIZE ANALYSIS					
CLAY	SILT	SAND	GRAVEL	UNIT DENSITY (PCF)	DRY DENSITY (PCF)
%	%	%	%		

PLATE 8

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

A EW	FIELD ENG B M	DATE DRILLED 7/2/73	AIRPHOTO NO:	CHAINAGE: 1190+00	OFFSET: 300' E of ℄
D GGF	TECH	RIG AIR	SURFACE DRAINAGE: GOOD	VEGETATION: SPRUCE & BIRCH TO 40'	ELEV.

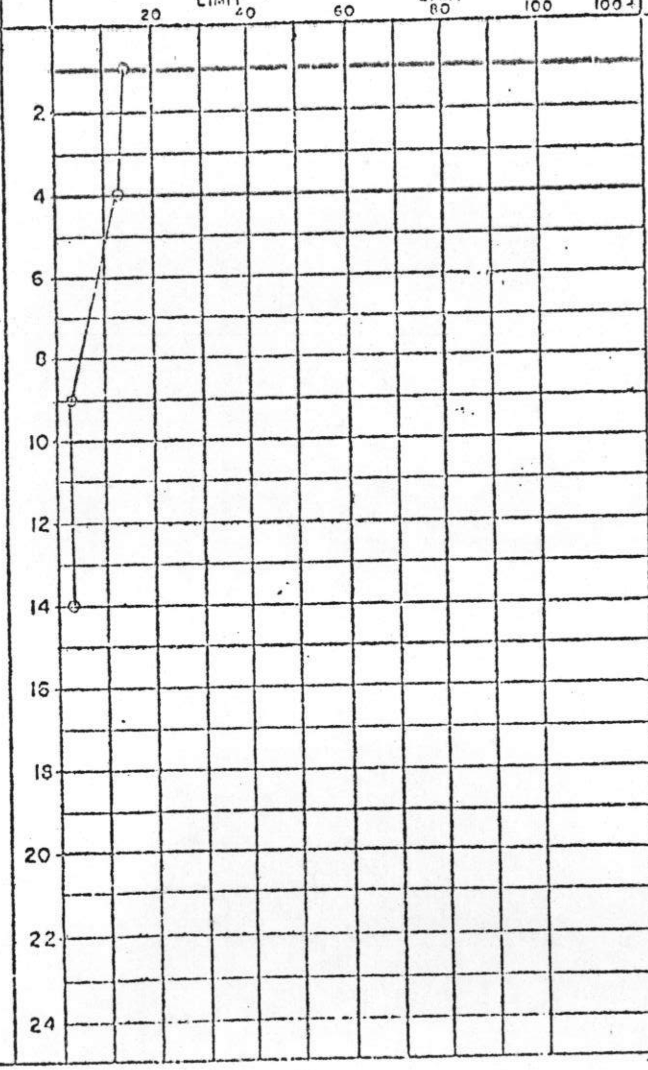
TEST HOLE

MILE	B,C,S	NUMBER
521	S	354A

REMARKS

(FEET)	SAMPLE NUMBER	SAMPLE-TYPE *	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
					GM	GRAVEL 2" ORGANIC	F	
2		/				SILT		
4		/			GC	SILTY CLAY	UF	
6								
3							F	
10		/				SAND		
12					GW			
14		/				SAND		
15		P				END OF HOLE		
18								
20								
22								
24								

○ = WATER CONTENT (% OF DRY WEIGHT) *
△ = ICE CONTENT (% OF SAMPLE VOLUME)



GRAIN-SIZE ANALYSIS				WET DENSITY (PCF)	DRY DENSITY (PCF)
CLAY	SILT	SAND	GRAVEL		
%	%	%	%		

PLATE 10

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

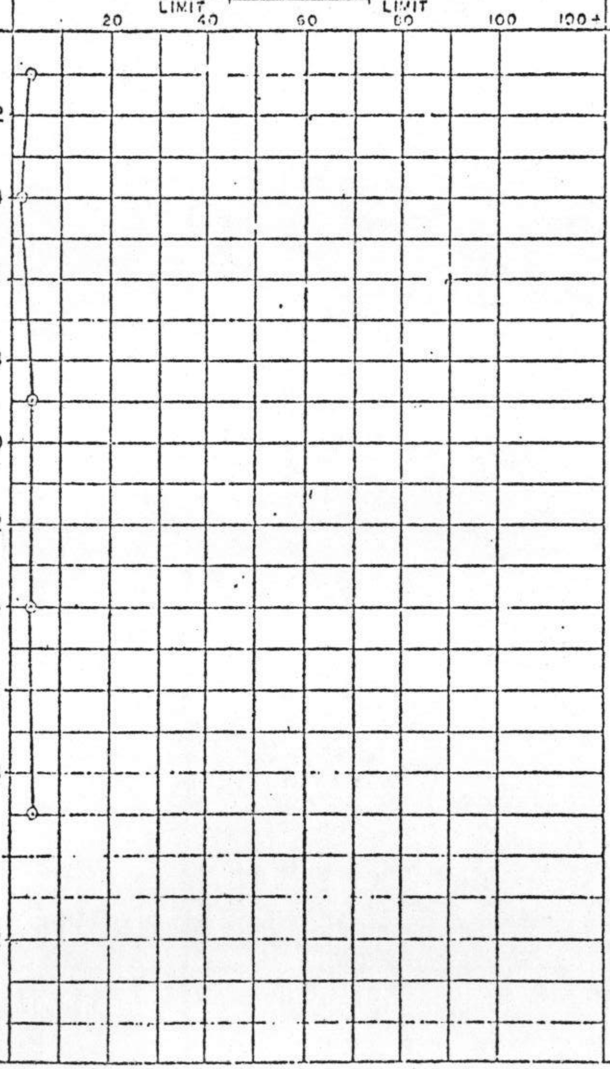
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

AN A.E.W. FIELD ENG B M
KD TECH
DATE DRILLED 7/2/73
RIG: AIR
AIRPHOTO NO:
SURFACE DRAINAGE: GOOD
CHAINAGE: 1188+00
VEGETATION: SPRUCE & BIRCH TO 35'
OFFSET: ℄
ELEV:

TEST HOLE		
MILE	B,C,S	NUMBER
521	S	356A
REMARK:		

(FEET)	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	DEPTH (FEET)
					SILT 3" ORGANIC	F		
2	╲			GC	GRAVEL		V	2
4	╲							4
6				GW	BOULDERS			6
8								8
10	╲				BOULDERS & SAND			10
12				GW				12
14	╲				SAND			14
16	P		23 BLS					16
18				GL	SANDY, POWDERY CLAY		V	18
20	╲				END OF HOLE			20
22								22
24								24

○ = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)
PLASTIC LIMIT —— LIQUID LIMIT



GRAIN-SIZE ANALYSIS				WET DENSITY (PCF)	DRY DENSITY (PCF)
CLAY	SILT	SAND	GRAVEL		
%	%	%	%		

PLATE 11

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

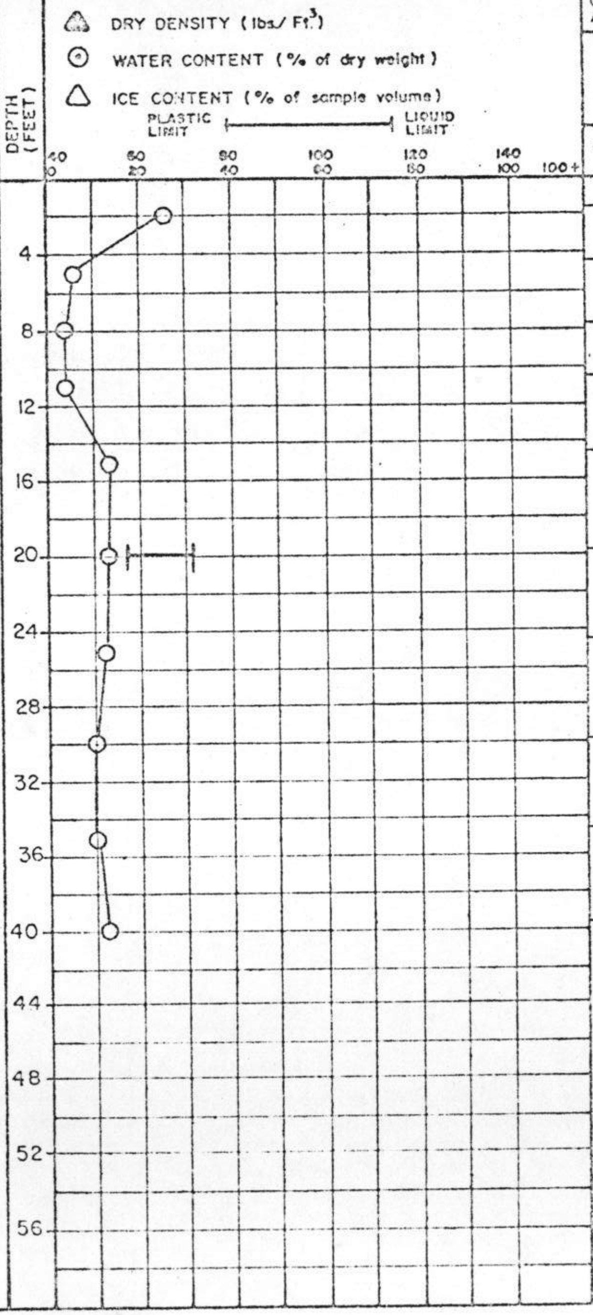
DRILL HOLE REPORT

SITE: LITTLE SMITH CREEK

FIELD ENG.
DATE DRILLED.
AIRPHOTO NO.
CHAINAGE.
OFFSET.
TECH. REYNOLDS
RIG. AIR
SURFACE DRAINAGE.
VEGETATION.
ELEV.
TEST HOLE #1 (1975)
MILE 534
REMARKS

UNIFIED SOIL SYMBOL	SOIL DESCRIPTION		LIMITS OF FROZEN GROUND	ICE DESCRIPTION
Pt	Peat	1'		
CL OL	Clay-Silt-Sand Organic	3'		
GW	Gravel-Sand Mixture	13'	F	$V_c - V_r$
Cl	Clay-Silty Sandy Pebbles Medium Plastic - Till			Nbn
Cl	Clay-Silty Sandy Pebbles Medium Plastic	43'	F	
	Bottom Of Hole - 43'			

DEPTH (FEET): 4, 8, 12, 16, 20, 24, 28, 32, 36, 40, 44, 48, 52, 56



GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	RELATIVE THAWED MOISTURE CONTENT
72	-	26	2	Damp
5	-	53	41	Sat.
12	-	52	36	Damp
6	-	44	50	Wet
48	-	43	9	Moist
65	-	31	4	Damp
65	-	28	7	Damp
62	-	29	9	Damp
59	-	28	13	Damp
63	-	30	7	Damp

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DRILL HOLE REPORT

SITE: LITTLE SMITH CREEK

FIELD ENG.

DATE DRILLED.

AIRPHOTO NO.

CHAINAGE.

OFFSET.

TECH. REYNOLDS

RIG. AIR

SURFACE DRAINAGE.

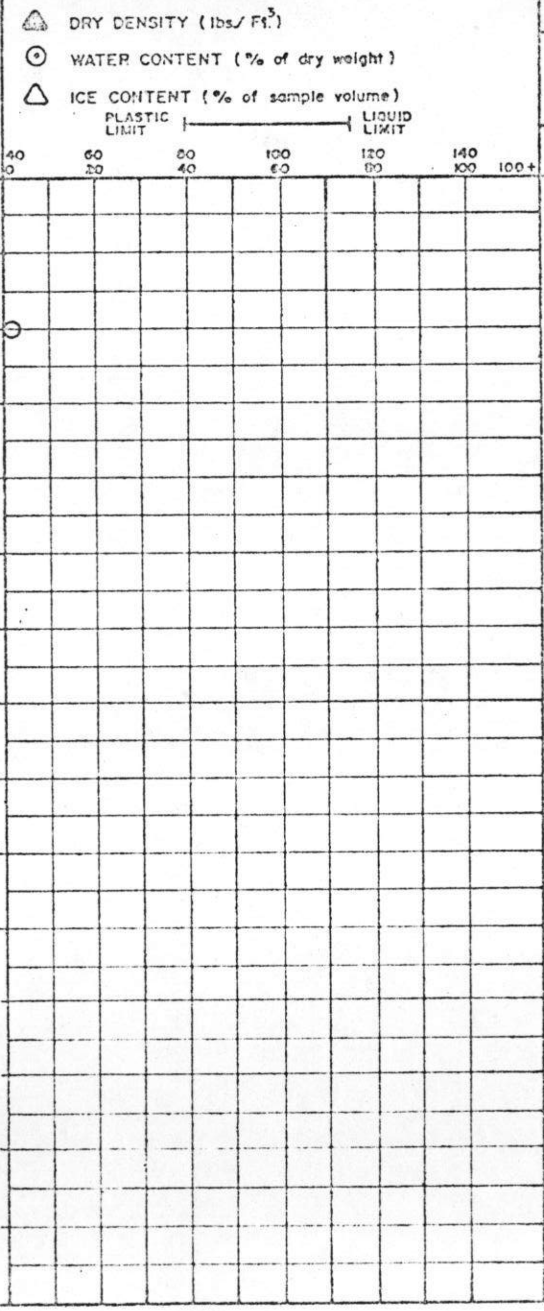
VEGETATION.

ELEV.

TEST HOLE #2 (1975)

MILE 534

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	DEPTH (FEET)	GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	RELATIVE THAWED MOISTURE CONTENT	REMARKS
						Gravel-Sandy				-	10 -	54	36		
4					GC	Silty	UF		4	-	7 -	43	50		
8						Damp to 8'			8	-	5 -	29	66	Moist	
						Sat. 8' - 11'				-	13 -	43	44		
12						Bottom of Hole - 11'			12						
16									16						
20									20						
24									24						
28									28						
32									32						
36									36						
40									40						
44									44						
48									48						
52									52						
56									56						



D3·14·5·74 a

NDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

N. R. S
FIELD ENG R. S
TECH:
DATE DRILLED 10/2/73
RIG: AIR
AIRPHOTO NO:
SURFACE DRAINAGE: GOOD TO RIVER
CHAINAGE: 525+00
VEGETATION: 50'-80' SPRUCE
OFFSET: ℄
ELEV:

TEST HOLE		
MILE	B,C,S	NUMBER
533	C	400A

SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P C F)	DRY DENSITY (P C F)	REMARKS
	/			GW	GRAVEL	F			5	25	70			
	/			GW										
	/			CL	CLAY TILL	UF								
	/		87 BLS	CL										
	/			CL										
	/		110 BLS	CL										
	/			CL										
	/			CL		UF								
	/		67 BLS	CL	END OF HOLE									

DEPTH (FEET)

○ = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)
PLASTIC LIMIT ⊢—⊣ LIQUID LIMIT

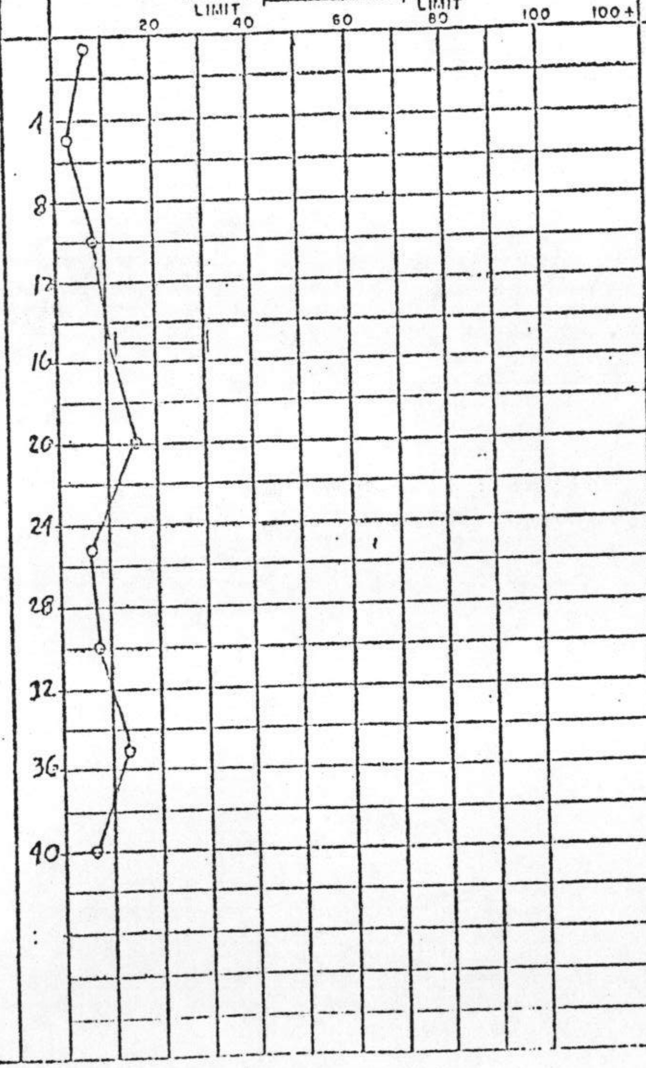


PLATE 7

UNDERWOOD McLELLAN & ASSOCIATES LIMITED | **DRILL HOLE REPORT** | **DEPARTMENT OF PUBLIC WORKS, CANADA**
MACKENZIE HIGHWAY

DWN A.C.W. | FIELD ENG. R.S. | DATE DRILLED 10/2/73 | AIRPHOTO NO: | CHAINAGE: 521+00 | OFFSET: ℄
CKD D.E.P. | TECH: | RIG: AIR | SURFACE DRAINAGE: GOOD | VEGETATION: SPRUCE TO 40' | ELEV:

TEST HOLE		
MILE	B.C.S	NUMBER
533	S	401A

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)	REMARKS
						[illegible] CLAY SAND & PEBBLES	F								
5						GRAVEL			12		29	59			
10				[illegible]		WATER @ 9' & 12' SAND, CLAY, COBBLES	UF								
15															
20				48 [illegible]		CLAY TILL									
25															
30				73 [illegible]											
35															
40				75 [illegible]											
45						CLAY TILL SOME BOULDERS									
50				73 CLS		CLAY									
55															
60				41 [illegible]		END OF HOLE									

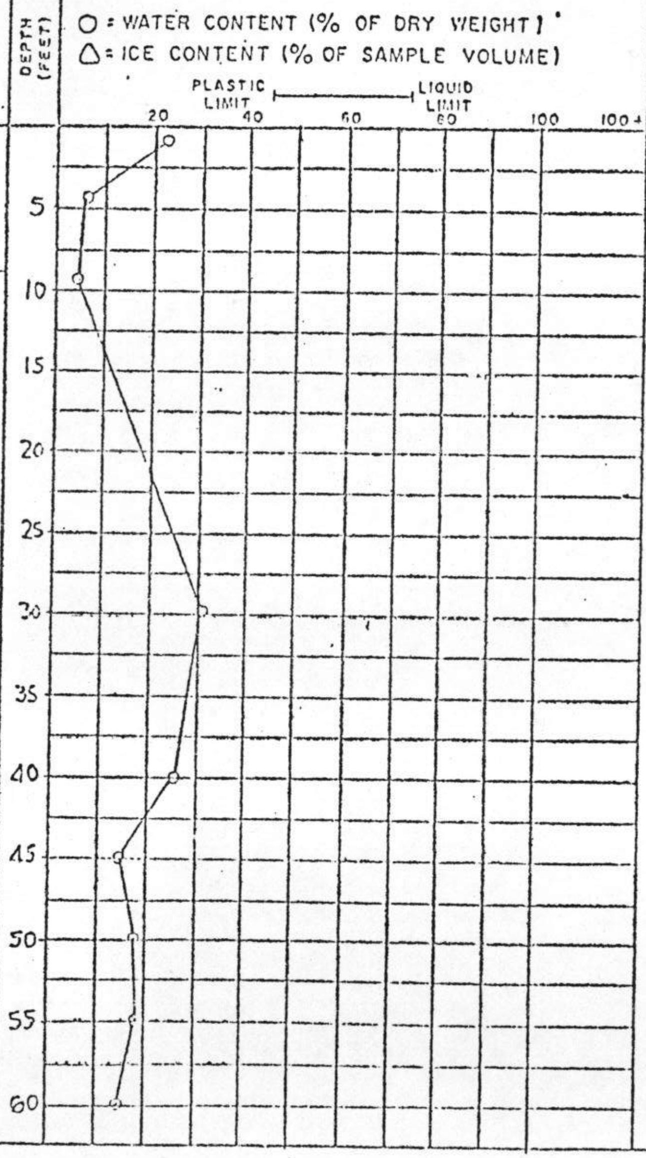


PLATE 8

NDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

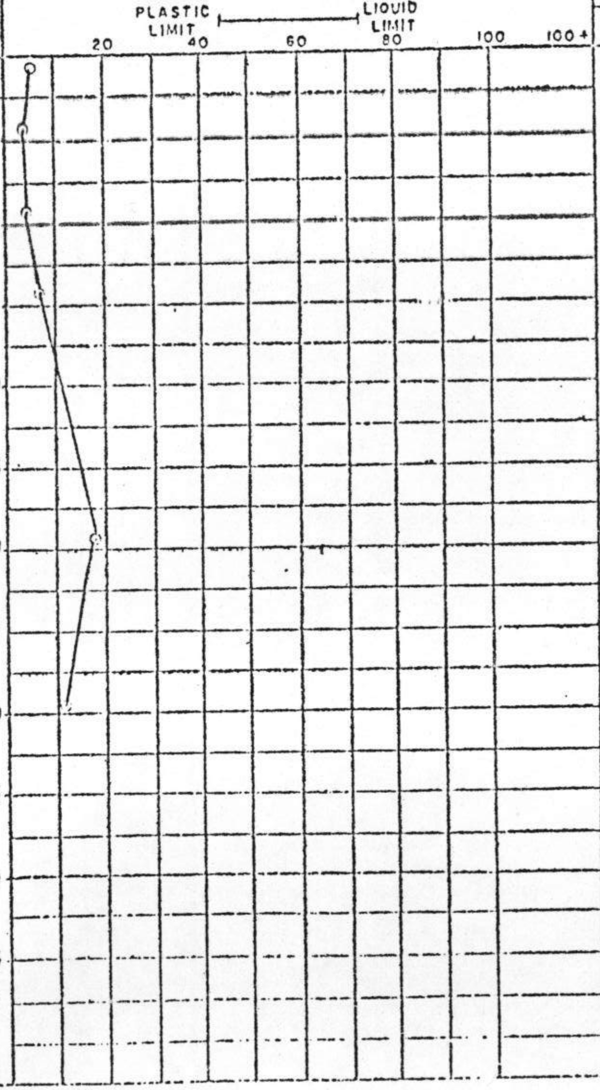
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

N A E W   FIELD ENG N.W.C   DATE DRILLED 11/2/73   AIRPHOTO NO: CHAINAGE: 519+00   OFFSET: ℄

KD DGP   TECH: RIG: AIR   SURFACE DRAINAGE: GOOD   VEGETATION: SPRUCE TO 40'   ELEV:

TEST HOLE		
MILE	B,C,S	NUMBER
533	S	402A

O : WATER CONTENT (% OF DRY WEIGHT)
△ : ICE CONTENT (% OF SAMPLE VOLUME)



(FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	DEPTH (FEET)	GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P C F)	DRY DENSITY (P C F)	REMARKS
						GRAVEL 6" TOPSOIL	UF									SO4 : 0.07%
5					GW				5							
10				23 3LS		SAND, PEBBLES AND BOULDERS			10	/		34	66			
15									15							HOLE SLOUGHING
20					GW				20							
25					GC	CLAY TILL GRAVEL			25							
30									30							
35									35							
40									40							
45						DENSER GRAVEL CONTENT			45							
50				9f BLS	GC				50							
55						END OF HOLE			55							

PLATE 9

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

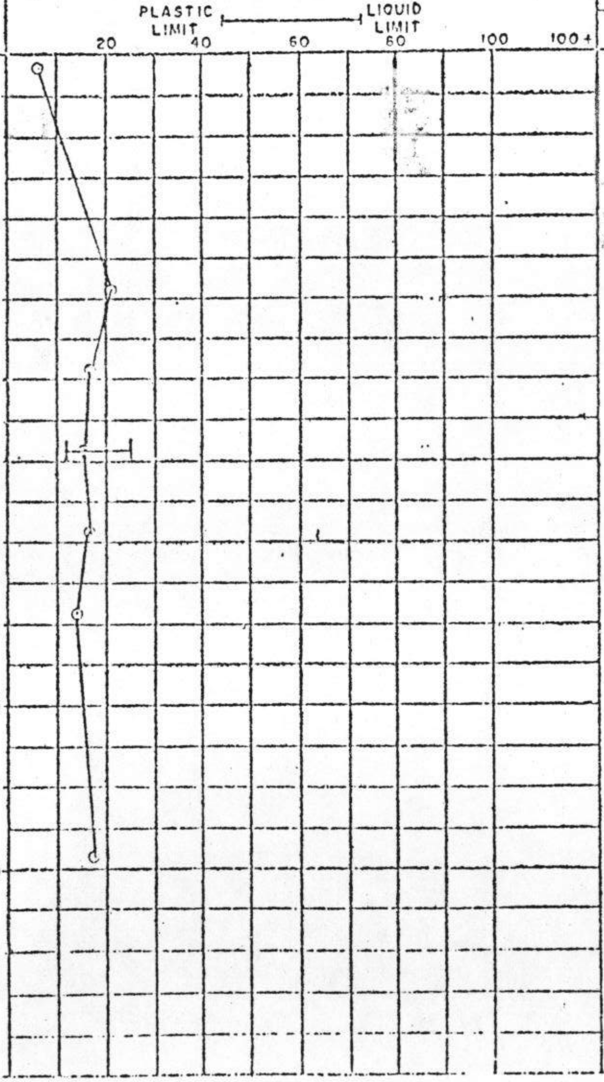
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

WN AEW FIELD ENG NWC DATE DRILLED 12/2/73 AIRPHOTO NO: CHAINAGE: 521+00 OFFSET: ℄
KD DGP TECH RIG: AIR SURFACE DRAINAGE: GOOD VEGETATION: SPRUCE TO 50' ELEV.

TEST HOLE

MILE	D,C,S	NUMBER
533	S	403A

○ : WATER CONTENT (% OF DRY WEIGHT)
△ : ICE CONTENT (% OF SAMPLE VOLUME)
PLASTIC LIMIT ├──┤ LIQUID LIMIT



(FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS
		⧄				SAND 2" TOPSOIL	UF								SO_4 = 0.09%
5						GRAVEL									
10				112 BLS		COARSE, DRY									HOLE SLOUGHING
15		⧄				TILL									
20		⧄													
25		⧄		35 BLS											
30		⧄				SAND & PEBBLES									
35		⧄													
40						CLAY, SAND AND PEBBLES									
45															
50		⧄		18 BLS											
55						END OF HOLE									

PLATE 10

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

WN A EW
KD DGP
FIELD ENG R S
TECH
DATE DRILLED 13/2/73
RIG: AIR
AIRPHOTO NO:
SURFACE DRAINAGE: GOOD
CHAINAGE: 519+00
VEGETATION: POPLAR & SPRUCE TO 80'
OFFSET: ℄
ELEV.

TEST HOLE		
MILE	D.C.S	NUMBER
533	S	404A

(FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS
					GP	6" TOPSOIL SAND	UF								SO4 = 0.09%
4						GRAVEL					22	78			
8				41 S.S	GC	SANDY WET			6		17	77			
12						CLAY- TILL			40		29	31			
16															
20				49 S.S											
24					GC	WET, GRAVEL, SAND									
28															
32															
36					GC										
40				100 BLS		END OF HOLE									
44															

DEPTH (FEET)

O = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)

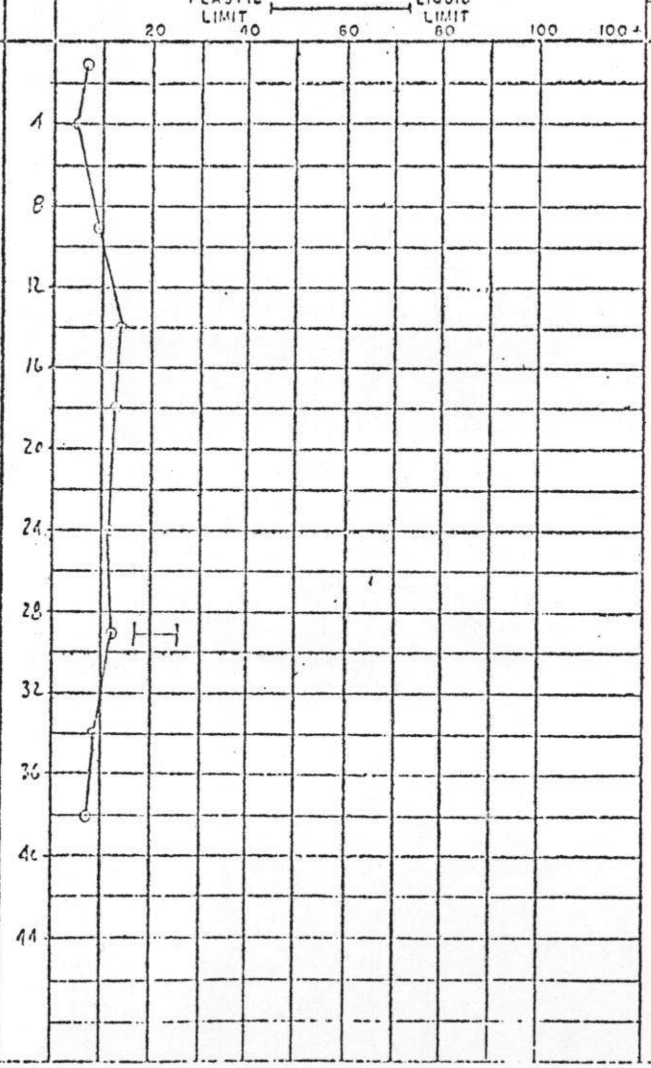


PLATE 11