



REPORT ON SUBSURFACE SOIL CONDITIONS FOR PROPOSED -

BRIDGE MILE 533.0

MACKENZIE HIGHWAY

Underwood McLellan & Associates Limited

ISSUE #39

# REPORT ON SUBSURFACE SOIL CONDITIONS FOR PROPOSED LITTLE SMITH CREEK BRIDGE MILE 533.7 MACKENZIE HIGHWAY NORTHWEST TERRITORIES

Prepared by Underwood McLellan & Associates Limited Consulting Professional Engineers.



Underwood McLellan & Arrociater Limited

QUBBER NO . 2106-032-45 April 25, 1973

Department of Public Works P.O. Box 488 Edmonton, Alberta

Attention: Mr. F. E. Kimball

Dear Sir:

Subsurface Soil Investigation Re: Proposed Little Smith Creek Bridge Mile 533.7, Mackenzie Highway

Attached please find our report on the subsoil and foundation conditions for the proposed Little Smith Creek Bridge at Mile 533.7 on the Mackenzie Highway in the Northwest Territories.

If you have any questions concerning the contents of this report, we would be pleased to discuss them with you at your convenience.

Yours very truly,

UNDERWOOD MCLELLAN & ASSOCIATES LIMITED

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Per:

D. G. Pennell, Ph.D., P.Eng.

DGP/pd

Att'd.

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### A. INTRODUCTION

The subsoil investigation undertaken at the Little Smith Creek forms part of the overall geotechnical investigation which was conducted from mile 450 to 550 of the Mackenzie Highway for the Department of Public Works, Government of Canada.

The present report summarizes the results of subsoil investigation operations conducted in connection with the proposed Little Smith Creek bridge at mile 533.7 of the Mackenzie Highway.

It was the intention of the present investigation to determine the vertical sequence of the subsoil materials, permafrost conditions and groundwater conditions existing which would be expected to influence the stability of the valley slope and the design and construction of the proposed bridge foundations.

The included conclusions and recommendations have been based upon exploration and sample acquisition

operations conducted at 10 positions along the proposed bridge centreline in the river channel and slopes of the bridge approaches.

In the formulation of conclusions and recommendations, the consistency and composition of the subsoil materials between and in the proximity to the individual test boring positions has been assumed, but not verified.

### B. FIELD AND LABORATORY INVESTIGATION

Ten (10) test holes have been drilled along the valley slopes and near the proposed bridge foundations in the river channel. In addition to the ten centreline test hole logs, borrow pit hole test logs in the vicinity of the bridge site have been included in the Appendix. The test holes drilled along the centreline were drilled to depths varying from 12 to 60 feet below existing site grades. The borrow pit test holes were generally advanced to the fifteen foot level.

Test borings along the centreline were drilled February 10 to 13, 1973, by Kenting Big Indian Drilling of Calgary, utilizing a Mayhew 1000 and Heli drill. Air recovery methods with "finger" bits were utilized in all test holes.

The locations of the individual test borings were selected and field located by Underwood McLellan & Associates Limited. The approximate test boring locations are graphically illustrated on the accom-

з.

panying Mosaic Plot and Test Hole Location Plan.

All variations of soil, permafrost and groundwater conditions encountered during the test boring operations were noted and representative samples of the existing subsoils were taken. The majority of samples were disturbed, having been obtained by the air recovery method. Sampling was also undertaken with a two-inch O.D. split spoon sampling tube. Standard Penetration blow counts were obtained in the gravel and glacial till with a solid 2" O.D. penetrometer.

In the laboratory, the samples of materials derived from the drilling operations were subjected to standard laboratory classification procedures including particle size distribution and Atterberg index property determinations. Moisture contents were obtained for all samples returned to the laboratory. Wherever a sufficient length of intact sample was retained in the penetrometer

tube, both wet and dry densities were obtained.

Permafrost samples were classified according to N.R.C. Technical Memorandum No. 79 "Guide to the Field Description of Permafrost." The excess ice content of permafrost samples was obtained by visual inspection and by measured thicknesses of ice lenses. Excess ice content was determined by the division of the ice thickness by the total sample thickness.

All soil samples were classified according to the Unified Soil Classification System.

The results of all laboratory and field tests are summarized in the Appendix on the test hole logs.

Test holes have been designated by mileage, type and number. Examples include 533B 564B, 533C 397A and 533S 400A. The letter symbols and numbers indicate:

533 - mileage

B, C, S - between mileage and test hole number

indicate borrow, centreline and structure test holes, respectively. 564, 397, 400 - test hole numbers A, B - after test hole numbers indicate drilling performed by Mayhew 1000 or Heli drill, respectively.

### C. 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.

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.

#### D. 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 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

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 crosssectional 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

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 <u>approximately</u> 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

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.<sup>2</sup> of pile can be obtained by  $P = \frac{1}{2} \mathbf{X}_{b} K_{p} H^{2}$ . 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 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.

### 2. Lateral Abutment Loads.

The magnitude of the pressure applied to the abutment will depend upon the characteristics of the backfill chosen for the approach fill and upon the movements that the wall undergoes during and subsequent to fill compaction. It is recommended that granular backfill from the river terraces or borrow pits be utilized for the abutment approach embankments which will be in the order of 20 feet high.

The most common procedures for analysis of the earth pressures against gravity retaining walls include the Rankine and Coulomb methods.

Generally, it is assumed that the wall rotates sufficiently to allow mobilization of the backfill shear strength which reduces the pressure to the active case, the lowest pressure that can be realized. This assumption applies reasonably accurately when the backfill consists of uncompacted granular material but pressures greater than the active occur whenever compaction of the backfill is undertaken.

While the total yield at different elevations on the abutment may be adequate for mobilization of the full backfill strength, a gradual buildup of the backfill also results in a gradual yielding of the wall. Therefore, by the time the backfill is complete only a fraction of the total yield of the wall has become effective in decreasing the earth pressure. Recent studies have shown that the actual earth pressure magnitude will exceed the active case and may

approach the earth pressure at-rest. Pressures greater than the at-rest case may occur if heavy equipment is utilized near the concrete abutment.

If the Coulomb approach is utilized, the angle of wall friction between concrete and coarse sand and gravel would be 20 degrees. The angle of internal friction for compacted granular material would be 38 degrees and the dry density may be taken as 115 pcf. For a simple vertical retaining wall abutment with horizontal backfill the Coulomb active coefficient would be .24 and the at-rest coefficient based on  $K_0 = 1 - \sin \emptyset$ would be .38. The pressure distribution behind a gravity retaining wall has been shown to be non-triangular, but as a first approximation and for simplicity, the triangular pressure diagram is recommended. It is also recommended that whenever backfill is compacted in lifts by vibratory compaction the at-rest coefficient be utilized for calculation of the ultimate earth pressure.

If driven piles are utilized to carry the approach abutments it is recommended that some batter piles be utilized to ensure adequate lateral support against earth pressures.

Positive drainage including horizontal perforated tile and weep holes should be provided to prevent development of hydrostatic pressures against the wall. In addition, drainage will prevent development of frost heaving forces against the abutments.

### 3. Bridge Approach Embankments and Borrow Material

The proposed centreline grades at the Little Smith Creek bridge site will involve primarily fill sections. The proposed fills will vary from 20 feet at the bridge crossing to 35 feet deep approximately 1000 feet south of the main channel. Large trees should be removed from the fill areas but surface vegetation should remain in place below the proposed embankments. Wherever the fill depth becomes shallow on the valley

slopes, the vegetation will provide insulation and tend to maintain permafrost conditions. The deep fills will overlie granular deposits which will provide adequate strength for the superimposed embankment loads. It is recommended that granular fill be used in all approach fills to the bridge abutments and this material should be compacted to 95% of Standard Proctor density utilizing vibratory compaction. Although fill construction may be performed during the winter with dry unfrozen gravel, it is recommended that embankments be placed in the summer. A graded gravel-sand mixture embankment should be placed at 2:1 side-slopes and it is mandatory that compaction of these fills be undertaken to ensure stability. If materials other than gravel are utilized, the strength characteristics of these materials must be evaluated to select stable side-slopes.

Common excavation will not be available from the valley slopes as the greatest portion of the grade-line has been selected to produce fill sections. Suitable granular embankment material is available throughout the valley slopes above the glacial till but its utilization as a construction material may be prohibited by land use regulations. Unfrozen glacial clay till may be utilized for embankments compacted during the summer but test holes drilled generally indicate that gravel and sand overburden are present above the till.

Suitable borrow pits were located south of the bridge site at station 531 + 00 and north at station 505 + 00. These pits contained gravel and clay till with the gravel generally, overlying the till. In order to obtain sufficient gravel for the embankments large borrow areas will likely necessitate development.

A 10 foot cut section is proposed on a north

terrace at station 515 + 00 in a soil strata consisting of four feet of sand over unfrozen dry gravel. This cut, should be stable when excavated at 2:1 and no grade-line revisions are recommended.

Another 10 foot cut has been proposed on the south slope near the uplands at station 544 + 00. This cut will be undertaken in 4 feet of surface gravel and to a depth of 6 feet in unfrozen silty clay with a moisture content of 20%. It is anticipated that back-slopes of 2:1 will ensure stability of the cut section.

#### APPENDIX

## EXPLANATION OF FIELD & LABORATORY TEST DATA

The field and laboratory test results as shown for a particular test boring by the Test Hole Log Data Sheet are briefly described below:-

#### NATURAL MOISTURE CONDITIONS & ATTERBERG LIMITS

The relation between the natural moisture content and depth is significant in determining the subsurface moisture conditions. The Atterberg Limits should be compared to the Natural Moisture Content of the subsurface soil as well as plotted on the Plasticity Chart.

## SOIL PROFILE & DESCRIPTION

Each soil strata is classified and described noting any special conditions. The unified classification system is used, and the soil profile refers to the existing ground elevation. When available the ground elevation is shown.

The soil symbols used are briefly shown below but are indicated in more detail in the Soil Classification Chart.



### TESTS ON SOIL SAMPLES

Laboratory and field tests are identified by the following symbols:

- QU unconfined compressive strength usually expressed in tons per square foot. This value is used in determining the allowable bearing capacity of the soil.
- fd dry unit weight expressed in pounds per cubic foot. This value indicates the density or consistency of the in-situ soil.

- 1 -

- C Consolidation test. These test results are separately enclosed and provide information on the consolidation or settlement properties of the soil strata.
- M.A. grain size analysis. These test results are separately enclosed and indicate the gradation properties of the material tested.
- SO<sub>4</sub> water soluble sulphate content is conducted primarily to determine whether sulphate resistant cement is required for the foundation structure.
- N standard penetration field test. This test is conducted in the field to determine the in-situ consistency of a soil strata. The "N" value recorded is the number of blows from a 140 lb. hammer dropped 30 inches (free fall) which are required to drive a 2" O.D. Raymond type sampler 12 inches into the soil.

The resistance and unconfined compressive strength of a cohesive soil can be related to its consistency as follows:-

N - BLOWS/Ft.	$QU - T/Ft.^2$	CONSISTENCY
2	0.25	very soft ·
2-4	0.25-0.50	soft
4-8	0.50-1.00	medium or firm
8-15 .	1.00-2.00	stiff
15-30	2.00-4.00	very stiff
30	4.00	hard

The resistance of a non-cohesive soil (sand) can be related to its consistency as follows:-



- 2 -

## SAMPLE CONDITION AND TYPE

The depth and condition of samples are indicated by the following symbols:

	UNDISTU	RBED	DISTURBED	LOST SAMPLE	
SAMPLI	TYPES				
•	U D.S. M R.C.		3" O.D. Sh drive samp moisture c rock core	elby tube sample le ontent sample sample	9

### PERCENTAGE WATER SOLUBLE SULPHATE CONCENTRATION

0 0.1 0.2 0.3 0.4 0.5 0.6 0.7%

Negli- gible	Posi- tive	Considerable	Severe
	RELATIV	E DEGREE OF SULPHATE AT	ГАСК

<u>Negligible</u> - Normal Portland Cement may be used. <u>Positive</u> - Normal Portland Cement may be used, provided the strength of the concrete is increased up to 500 psi higher than the compressive strength which would normally be used.

<u>Considerable</u> - Type V cement must be used and the concrete compressive strength should be increased to 500

- 3. -

psi higher than the compressive strength which would normally be used.

<u>Severe</u> - Type V cement must be used and the concrete compressive strength should be increased from 500 to 1000 psi higher than the compressive strength which would normally be used.

### GROUND WATER TABLE

The water table is indicated by the level of standing water in a test boring after equilibrium has been reached. This is generally taken 24 hours after the drilling operation. The water table is usually an inclined surface that is dynamic in nature with its highest level late in the winter or early spring gradually falling throughout the summer.

# UNIFIED SOIL CLASSIFICATION SYSTEM

м	AJOR DIVIS	KONS	GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
	GRAVEL.	CLEAN CRAVELS		GW	WELL-GAADED GRAVELS, GRAVEL- Sand Mixtures, Little on No fines
COARSE	SCILS	fines)	į.,	GP	POORLY-GRADED GRAVELS.GRAVEL- SAND MIXTURES. LITTLE OR NO FINES
SUILS	HORE THAN 50%	GRAVELS WITH FINES		ĠМ	SILTY GRAVELS, GRAVEL-SAND- SILT MIRTURES
	OH NO.4 SIEVE	OF FINES)		GC	CLAYEY GRAVELS, GRAVEL-SAND- CLAY WIXTURES
	SAND	CLEAN SAND (LITTLE		sw	WELL+GRADED SANDS, GRAVELLY Sands, Little on ND Fines
WORL THAN SOR DE WATERIAL IS	SOILS	OR NO FIRES		SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE DA NO FINES
200 SIEVE SIZE	MORE THEM SOL	SALES WITH FINES		SM	SILTY SANDS, SAND-SILT WINTURES
	TION CASSING NO. 4 SIEVE	OF FINCS)		SC	CLAYEY SANDS, SAND-CLAY MIXTURES
				ML	INDREANIC SILTS AND YEAY FINE SANDS, ROCK FLOUR, BILTY OR CLAYEY FINE SANDS OR GLAYEY SILTS WITH SLIGHT PLASTICITY
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SAMDY CLAYS, SILTY GLAYS, LCAN CLAYS
				OL	GRGANIC SILTS AND DRGANIC Stlty Clays of Low Plasticity
				мн	INDEGANIC SILTS, MICACCOUS OR DIATOMACTOUS FINE SAND OR SILTY SOLLS
MORE THAN SON OF WATERIAL IS SMALLED THAN NO. 210 SILVE SIZE	SILTS AND CLAYS	CIQUID LIWIT		СН	INDRGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				он	ORGANIC GLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
н	GHLY ORGANIC SOL	.5		PT	PEAT, HUNUS, SWAMP BOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE BOIL CLASSIFICATIONS. BOIL CLASSIFICATION CHART

PLASTICITY CHART

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# NATIONAL RESEARCH COUNCIL PERMAFROST CLASSIFICATION SYSTEM

Permafrost ground ice occurs in three basic conditions including non-visible, visible (less than one inch in thickness) and clear ice.

A. <u>Non-visible - N</u>

 $N_{f}$  - poorly bonded or friable frozen soil  $N_{bn}$  - well bonded soil, no excess ice  $N_{be}$  - well bonded soil, excess ice

B. <u>Visible</u> - V (less than 1" thick)

 $V_x$  - individual ice crystals or inclusions

 $V_{c}$  - ice coatings on particles

V<sub>r</sub> - random or irregularly oriented ice formations

V<sub>s</sub> - stratified or oriented ice formations

C. <u>Visible Ice</u> - (greater than 1" thick)

Ice - ice with soil inclusions

Ice + Soil - ice without soil inclusions.

A more complete description of this system is included in NRC publication TM 79.





# LITTLE SMITH CREEK CROSSING MILE 533.0



Viewing Crossing in a North Easterly Direction



Viewing Crossing in a South Westerly Direction



Viewing Crossing in a Southerly Direction. NOTE: Airstrip in Top Left Hand Corner.



Viewing Crossing in a Northerly Direction

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ÖV	NR	. <u>S</u>	FIE	LD E	NG A	2.5 D	ATE DRILLED 101	2/13 AIR	рнот	0 NO:			CHAINA	GE:	5	527	00	, , ,	0.000	OFF	SET	(c) c	¢.				TEST	HOLE	
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PLATE 4

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