

Underwood McLellan & Associates Limited

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Underwood McLellan & Arrociater Limited

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OUR FILE NO April 27, 1973

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

Mr. F. E. Kimball ATTENTION:

Dear Sir:

#### Subsurface Soil Investigation Re: Proposed Big Smith Creek Bridge Mile 545, Mackenzie Highway.

Attached please find our report on the subsoil and foundation conditions for the proposed Big Smith Creek bridge at mile 545 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

Mr ennell

Per:

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

DGP/rr

Encl.

ENGINEERING AND PLANNING CONSULTANTS

REPORT ON SUBSURFACE SOIL CONDITIONS FOR PROPOSED BIG SMITH CREEK BRIDGE MILE 545 MACKENZIE HIGHWAY NORTHWEST TERRITORIES

## Prepared by

UNDERWOOD McLELLAN & ASSOCIATES LIMITED Consulting Professional Engineers

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## A. INTRODUCTION

The subsoil investigation undertaken at Big 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.

It was the intention of the present investigation to determine the vertical subsoil material sequence, 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 9 positions adjacent to the proposed bridge centreline in the creek 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

- 1 -

individual test boring positions has been assumed, but not verified.

- 2 -

#### B. FIELD AND LABORATORY INVESTIGATION

A total of nine (9) test holes have been drilled in the Big Smith Creek valley. In addition to the nine (9) structure and centreline test hole logs, borrow hole test logs closest to the bridge site have been included in the Appendix. Test holes along the centreline were drilled to depths varying from 12 to 40 feet below existing site grades. Borrow test holes were generally advanced to the 15 foot level.

Test borings were drilled February 15, 16 and 19, 1973 by Kenting Big Indian Drilling of Calgary utilizing a Mayhew 1000 and Heli drill. Air recovery methods were employed in all test holes.

The locations of the individual test borings were selected and located by Underwood McLellan & Associates Limited. The approximate test boring locations are graphically illustrated on the accompanying Mosaic and Test Hole Location Plan.

All variations of soil, permafrost and groundwater

- 3 -

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. Standard Penetration blow counts were taken with a two-inch solid 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.

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.

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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. Example includes 543B632B, 544C448A and 545S441A. The letter symbols and numbers indicate:

543, 544, 545 - mileage.

B, C, S - between mileage and test hole number indicate borrow, centreline and structure test holes, respectively.

632, 448, 441 - test hole numbers.

A, B - after test hole numbers indicate drilling performed by Mayhew 1000 or Heli drill, respectively.

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#### C. SITE AND SUBSOIL CONDITIONS

The Big Smith Creek site is located at mile 545 (chainage 77+00) of the Mackenzie Highway in the North West Territories and flows into the Mackenzie River from the east having a drainage basin which extends to the Franklin Mountains.

The stream elevation at the proposed crossing is 340 (D.P.W. plan-profile datum) with the estimated high water mark at elevation 345. The main stream channel is approximately 8 feet deep with shale bedrock outcropping along the creek banks. The tree growth throughout the relatively shallow valley consists primarily of spruce, aspen and poplar from 15 to 60 feet in height.

Six test holes were drilled in the vicinity of the proposed bridge crossing including 442A, 443A and 444A which were advanced along the C.N.T. right-ofway down to the banks of the main channel and 445A, 446A and 447A which were advanced on the temporary winter road in the main creek channel. It was not

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possible to advance test holes on the centreline above the river channel as clearing was not allowed and drilling in the river channel on the centreline or C.N.T. line could not be undertaken as a result of free water and thin ice. Five of the six test holes drilled near the proposed crossing had surface elevations above the existing adjacent centreline grades.

There were significant variations in the depth to bedrock as indicated by the test holes which were all drilled west of the proposed centreline. This bedrock variation is indicative of the irregular river downcutting of erodible shales and siltstones. It is estimated that the bedrock elevation in the vicinity of the centreline and piers would be 336 and is covered with 4 to 8 feet of unfrozen gravel overburden.

The most northerly drilled test hole during this investigation is 441A but it was drilled 260 feet west of the proposed centreline. This test hole disclosed 30 feet of dense glacial till overlying

- 7 -

the shale bedrock. Two solid penetrometer blow counts were 21 and 109 in the till and the average moisture content was 10% which was about 3% below the plastic limit. The low plastic nature of the till was confirmed by liquid limit of 31% and plasticity index of 16. The water table was encountered at elevation 348 in test hole 441A which was coincident with the north terrace elevation immediately above the creek channel.

Two test holes drilled on the south slopes encountered 3 to 8 feet of silty sand and silty clay over relatively dry gravel.

The 9 test borings advanced in the Big Smith Creek valley indicated non-frozen soil conditions, although it is difficult to establish permafrost conditions in the bedrock.

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#### D. 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 Big Smith Creek bridge foundations and approaches.

#### 1. Pier and Abutment Foundation Design

A pier and spread footing type foundation is recommended as a result of the surface outcropping of shale bedrock along the creek shores and drill encounter of bedrock near the surface in test holes advanced adjacent to the centreline. It is recommended that piers be placed at approximately elevation 330 in the shale bedrock. Because drill holes were not advanced along the centreline, it is necessary to confirm competent bedrock during excavation for the bridge piers. An allowable assumptive bearing capacity of 10 tons per square foot may be employed in undisturbed intact bedrock.

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Approach bridge embankments of 20 foot depth are proposed at this site. The abutments may be placed on spread footings directly in the approach fill if the embankment is composed of properly compacted granular fill as recommended in Section D3.

As a result of the relatively shallow approaches the abutments may be designed as walls retaining the complete fill for a depth of approximately 20 feet. For this alternative the abutments should bear in bedrock at approximately elevation 336.

If inferior materials or compaction techniques are utilized in the embankment, the abutment may be carried on piles into the bedrock or the second alternative of the abutment bearing directly in the bedrock may be utilized.

Although steel - H or pipe piles may be utilized for the bridge piers, preference is given to the shallow spread footing type foundation as a result

- 10 -

of the near surface occurrence of the bedrock.

If piles are employed for either the abutments or piers, they should 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.

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It is not anticipated that any difficulty would be experienced in penetrating the shallow overburden depth above the bedrock.

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 a pile can be obtained by  $P = \frac{1}{2} \chi_{b} K_{p} H^{2}$ . The submerged unit weight of the granular material above the bedrock 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. Generally, the lateral load tends to exceed the passive resistance, consequently, the above simplified approach is on the conservative side. Additional lateral resistance can be considered if the bedrock is penetrated.

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#### 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 15 to 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

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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 build-up 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 indicated 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 compaction equipment is utilized against 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

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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. These design parameters are based upon "average" gravel conditions but wherever deep abutments are proposed the actual shear strength parameters of the backfill should be established to accurately evaluate the lateral pressures. In many cases 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 should be utilized for calculation of the ultimate earth pressure.

Wherever potential fill settlement is anticipated a driven pile foundation has been previously recommended for the abutments which will provide

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significant lateral resistance but it is further recommended that some batter piles be utilized to ensure adequate lateral support against earth pressures and fill movement. Whenever batter piles are utilized for horizontal resistance, the lateral restraint of vertical piles is neglected.

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 Grades and Embankments

The centreline grades at the proposed location of the Big Smith Creek will involve fill embankments from 500 feet north of the bridge to the south uplands. These fills will vary from 20 feet at the main creek channel to 10 feet on either side and will overlie unconsolidated

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river deposited silts, sands and gravels. Some settlement of the surface materials will occur under the superimposed embankment loads but instability of the foundation material will not result. All the surficial deposits investigated throughout the valley were unfrozen except for seasonal frost penetration.

It is recommended that granular fill be compacted at all deep approach fills to 95% of Standard Proctor density utilizing vibratory compaction.

Although construction may be performed during the winter, dry unfrozen granular materials will allow satisfactory compaction. It is emphasized that compaction of these deep approach fills is mandatory to ensure slope stability and prevent settlements at the bridge approaches. All granular fill slopes should be constructed at 2:1. Common excavation will not be available from the bridge site valley slopes as the grade line has been proposed entirely to produce fill

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sections.

Some difficulty will be experienced in locating suitable stable embankment fill. Consideration should be given to the development of granular borrow pits along the south terrace in sand and gravel deposits as disclosed in test hole 450A. Suitable borrow pits were located on the south uplands but haul distances are 2 to 3 miles. A coarse sand deposit exists at station 21+75, one mile west of the centreline. In addition, shale outcroppings are present at station 17+00, 500 feet east of the centreline.

This subsurface investigation was terminated north of the bridge and the most northerly test hole drilled was 441A, 260 feet west of the centreline. This hole indicated 30 feet of very hard till over bedrock. Consequently, although the soil conditions were not investigated at the proposed cut, station 88+00, it is anticipated that the strata are similar to test hole 441A and potential instability will not result.

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#### 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.
- Jd dry unit weight expressed in pounds per cubic foot. This value indicates the density or consistency of the in-situ soil.

- С 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.
- S04 water soluable sulphate content is conducted primarily to determine whether sulphate resistant cement is required for the foundation structure.
- standard penetration field test. N 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: -



WIDTH B OF FOOTING IN FEET

ALLOWABLE SOIL PRESSURE IN T/FT<sup>2</sup> (WATER TABLE BELOW DEPTH

2

2B)

- 2 -

## SAMPLE CONDITION AND TYPE

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

CUDEDICUDED DICIOUDED DOGI ONNEL	UNDISTURBED	DISTURBED	LOST	SAMPLE
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## SAMPLE TYPES

U	-	3" O.D. Shelby tube sample
D.S.	-	drive sample
М	-	moisture content sample
R.C.	~	rock core sample

#### PERCENTAGE WATER SOLUBLE SULPHATE CONCENTRATION

0 0.1 0.2 0.3 0.4 0.5 0.6 0.7%

Negli- gible	Posi- tive	Consid	lerable		Severe
	RELATIV	E DEGREE OI	SULPHATE	ATTAC	K

Negligible - 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

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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

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M	AJOR DIVIS	IONS	GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
	GRAVEL SNO	CLEAN GRIVELS		GW	ALL-GRADID GRAVELS, LRAVIL Sand JATURES, LITTLE ON No Field
GCARSE	SCILS	(LITTLE GA NG	: . !	GP	PUGREY-GRADED GRAVELS, SRAVEL- Samu Mirtures, Little Or Nu Fires
SCILS	WORE THAN SON OF COARSE FRAC-	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL-SAND- Silt Hirtures
	ON LO.4 SIEVE	or rinks)	1.00	GC	CLAYEY BRAVELS. GRAVEL-SAND- GLAY WIXIURES
	SAND	CLEAN SHAD (LITTLE		sw	WELL-BRADED SAADS, GRAVELLY Saads, Little of ND FILES
MORE THAN SOS	SOILS	GR NG FINES)		5P	POCKLY-GRADED SANDS, GAAVELL- SANDS, LITTLE ON NO FINES
200 SIEVE SIZE	MORE THAN SOL OF CORRSE FRAC-	SANETS WITH FINES		ŞM	SILTY SANDS, SAND-SILT, WIRTURES
	NO. 4 SIEVE	OF FINES)		sc	CLAVEY SANDS, BAND-CLAY WIRTURES
				ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEV FINE SANDS OR CLAYEV SILTS WITH SLIGHT PLASTICITY
FINE GRAINED SQILS	SILTS AND GLAYS	LIGNID LIMIT		CL	HABBGANIE CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, Sandy Clays, Silty Clays, Lean Clays
				OL	GREANIC SELTS AND DREAMIC SILTY CLAYS OF LON PLASTICITY
				мн	INGREANIC SILTS, MIGACEOUS (- Siltomaceous fine sand or Silty soils
MORE THAN SOY OF MATERIAL IS MALLES THAN NO. 200 SIEVE SIZE	SILTS AND QLAYS	GREATER THAN 50		СН	INDRGANIC CLAYS OF NICH PLASTICITY, FAT CLAYS
				он	DAGANIC GLAVE OF MEDIUM TO HIGH PLASTIGITY, GRAAVIC BILTS
н	GHLY ORGANIC SOLL	s		РΤ	PCAT, HUMUE, SWAMP SDILS WITH HIMH GAGANIC CONTENTS

TES DUAL SYMPOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

#### SOIL 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. Non-visible - N

Nf - poorly bonded or friable frozen soil
Nbn - well bonded soil, no excess ice
Nbe - well bonded soil, excess ice

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

V<sub>x</sub> - 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.





BIG SMITH CREEK CROSSING MILE 545.



Viewing Crossing in a Easterly Direction.



Viewing Crossing in a North Easterly Direction

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