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MACKENZIE

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Arctic Red River

BANKS ISLAND VICTORIA ISLAND

DEPARTMENT OF PUBLIC WORKS MACKENZIE HIGHWAY MILE 346 - 450 GEOTECHNICAL INVESTIGATION

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NORTHWEST TERRITORIES

Lake

Fort Good Hope

Norman Wells

Mile 450 Wrigley

Great Bear

Mile 346

Fort Simpson

Yellowknife

Great Slave Lake

Hay River

Enterprise

BRITISH COLUMBIA

FOUNDATION REPORT FOR CROSSING AT WILLOW LAKE RIVER



FOUNDATION REPORT FOR CROSSING

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WILLOW LAKE RIVER MILE 394 MACKENZIE HIGHWAY NORTHWEST TERRITORIES

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1.0 INTRODUCTION

This report contains the results of the foundation investigation and recommendations for foundation and embankment design for the Willow Lake River crossing at Mile 394 on the Mackenzie Highway, Northwest Territories. The field investigation at this site was part of an overall geotechnical investigation conducted by Acres Consulting Services from Mile 346 to Mile 450 of the Mackenzie Highway for the Department of Public Works, Government of Canada.

The purpose of the investigation was to determine the site foundation, permafrost and groundwater conditions and to provide recommendations on the design and construction of the proposed bridge abutments, piers and approach embankments.

2.0 SITE AND GEOLOGY

The Willow Lake River crossing site is located at Mile 394.7 (chainage 985 + 00) on the Mackenzie Highway. This site is approximately one mile upstream from the junction of Willow Lake River with the Mackenzie River.

The stream bed elevation at the proposed crossing site is approximately 250 ft. (D.P.W. datum). The length of the bridge is 900 feet and 4 spans are proposed.

The crossing site is located in a shallow valley approximately 3/4 mile wide and 50 feet deep, incised into

2.0 SITE AND GEOLOGY - Continued

the fluvial sands and glacial tills which mantle the Mackenzie River valley in this section of the project.

The deepest boring, to a depth of 101 feet, did not encounter bedrock at this site, although shale is encountered at shallow depth several miles to the north along the highway centreline. Bedrock in the area is horizontally bedded shale, sandstone and limestone of the Fort Simpson Formation of Devonian age.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

A total of seven testholes were drilled at the bridge site along centreline to depths ranging from 15 to 101 feet at the locations shown in Figure 1. Figure 2 shows a centreline section of the valley.

Two testholes were drilled at the abutment locations to depths of 101 and 55 feet and three holes were drilled at pier locations to depths of 51, 71 and 66 feet respectively.

The drilling and sampling at the bridge site was done between January 30 and February 7, 1973 by Kenting Big Indian Drilling of Calgary utilizing a Gardner-Denver 200 "helidrill" mounted on a Foremost 60 tracked vehicle.

Sampling was done using the air recovery percussion method for disturbed samples, 3 inch thin-walled Shelby tubes for undisturbed cohesive soil samples, and the

3.0 FIELD INVESTIGATION AND LABORATORY TESTING - Continued

standard split spoon for granular soil samples. Standard Penetration blow counts were taken with the split spoon sampler.

Samples were logged in the field and classified according to the Unified Soil Classification System as shown in Figure 3. Ice contents were classified according to the N.R.C. Technical Memorandum No. 75 "Guide to the Field Description of Permafrost" as shown in Figure 4.

Moisture contents were obtained for all samples returned to the laboratory and the samples were subjected to routine classification tests including grain size distribution and Atterberg limits. The results of all laboratory and field tests are included in the testhole logs appended to this Report.

4.0 FOUNDATION CONDITIONS

The following boreholes were drilled to investigate the foundation conditions at the bridge abutment and pier locations.

HOLE NUMBER	LOCATION
394-с-в	South Abutment
394-S-C	South Pier
394-S-D	Centre Pier
394-S-E	North Pier
394-S-F	North Abutment

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4.0 FOUNDATION CONDITIONS - Continued

The detailed soil descriptions for these holes are given in the borehole logs appended to this Report.

The boreholes drilled for the three piers showed unfrozen, medium density, silty-sands and gravel to a depth of about 20 feet. Blow counts ranged from 20 to 59 and averaged 30.

This stratum is underlain by approximately 20 feet of dense, dark grey, sandy-silt with minor clay, having $W_L = 45$ percent, $W_p = 20$ percent and an average water content of 30 percent. Blow counts from the Standard Penetration test ranged from 22 to 52 and averaged 32 indicating that the silt consistency ranges from very stiff to hard.

Below a depth of about 45 feet, the soil is a clayeysilt with blow counts ranging from 22 to 39 and averaging 32 indicating a very stiff to hard consistency.

No frozen ground was encountered in the three pier testholes.

Testhole 394-C-B, drilled on the south abutment location, showed silty-sand and sandy-silt to a depth of 46 feet. The soil was frozen to a depth of 33 feet. Moisture contents varied from 20 to 30 percent in the lower section of the hole but increased to a maximum of 45 percent in the upper 10 feet of the hole. Blow counts of 46 and 19 were recorded at depths of 35 and 40 feet respectively, in the unfrozen soil, indicating a very stiff to hard consistency.

4.0 FOUNDATION CONDITIONS - Continued

Testhole 394-S-F, drilled on the north abutment location, showed loose to medium dense silty-sand to a depth of 18 feet. Standard Penetration Test blow counts ranged from 4 to 33. This stratum is underlain by dense sand and gravel to a depth of approximately 60 feet with an average blow count of 44. Dense grey clayey-sandy-silt occurs to a depth of 101 feet. A thaw-consolidation test on a sample from a depth of 16 feet in this hole showed a consolidation of 0.34 percent.

The two testholes drilled on the centreline of the approach embankments some 500 feet back from the river (394-C-G and 394-C-K) showed silty-sand and clayey-silt with moisture contents in excess of 20 percent in the top 20 feet. The soil sampled in these holes was frozen throughout.

5.0 RECOMMENDATIONS AND CONCLUSIONS

The following conclusions and recommendations are made, based upon the results of the investigation program, for the design and construction of the Willow Lake River bridge foundations and approach fills.

5.1 Pier and Abutment Foundation Design

The subsurface investigation program described previously shows the piers to be underlain by unfrozen, medium density, silty-sands and gravels to a depth of 20 feet. Very stiff to hard, unfrozen, sandy and clayey-

5.1 Pier and Abutment Foundation Design - Continued

silt underlies this stratum with the clay content increasing with depth. Under the abutments, some 20 feet of frozen silty-sand overlies silty sands and gravels. The upper 20 feet of the abutment holes are characterized by relatively high water contents and standard penetration blow counts in 394-S-F of 4 to 7 indicate a loose relative density in at least part of this stratum.

In view of the subsurface conditions under the abutments and of scour considerations, a driven friction pile foundation is recommended for all piers and abutments at this site. The suitable types include pre-cast reinforced concrete, and steel pipe and H piles. Steel H piles are recommended for use at this site in view of their high driving strength, high load capacity and ease of splicing.

It is recommended that all piles used for pier foundations should be driven to a depth of not less than 50 feet or "refusal". As an initial guide, it is recommended that "refusal" be considered to be 240 blows per foot (20 blows per inch) under a hammer rated at 15,000 ft-lb.

Ultimate pile capacity should be established in the field by use of dynamic pile driving formulae. The Janbu formula (Terzaghi and Peck, 1967^{1}) is recommended with the use of a factor of safety of 3 to establish an allowable load per pile. Design loading should not exceed 40 tons per pile for a minimum H pile size of HP 10 x 57.

¹Terzaghi, K., and Peck, R.B., 1967. <u>Soil Mechanics in</u> <u>Engineering Practice</u>, John Wiley and Sons, 2nd edition, p. 229.

5.1 Pier and Abutment Foundation Design - Continued

It is recommended that all piles used for abutment foundations should be driven to a depth not less than 60 feet into the existing soil or to "refusal". The recommendations for "refusal", establishment of allowable load per pile, and design loading limits are as outlined previously for the pier foundations.

The abutment foundations may be placed on pile-supported footings located in the embankment. If this procedure is adopted, it is recommended that the abutments be placed on select, well-compacted granular fill. Settlement of the foundation under the weight of the approach embankments would subject the piles supporting the abutments to significant negative skin friction which must be allowed for in the pile design. It is recommended, as an alternative, that the abutment sites be preloaded for a minimum of 2 summer months with the full height of the approach embankment to allow consolidation of the foundation soil, then a portion of the embankment fill be removed to allow pile driving and construction of the bridge abutments.

5.2 Lateral Abutment Loads

The approach fills will have a maximum height of approximately 30 feet and it is recommended that they be constructed of well-compacted granular fill from the approach cuts or the adjacent borrow pits. Thus earth pressures against the concrete abutment and wing walls will be larger than the active case and will approach the earth pressure at-rest case.

5.2 Lateral Abutment Loads - Continued

It is recommended that the lateral earth pressure design should use a triangular load distribution using a coefficient of earth pressure at-rest (K_0) of 0.40 with due allowance for any surcharges or live loads acting near the wall.

Either the Coulomb or the Rankine method of calculation of earth pressure against the wall can be used. It is recommended that the angle of shearing resistance of the granular fill be taken as 38 degrees and the dry density of the granular fill be taken as 125 pcf. If the Coulomb method of analysis is adopted, the angle of wall friction between the concrete and the granular fill should be taken as 20 degrees.

The use of extensive measures to ensure proper drainage of the backfill behind the abutment is recommended to prevent the development of hydrostatic pressure against the wall. The measures adopted should include the use of perforated steel pipe drains at the base of the wall, weep holes through the wall and the use of select, free-draining granular fill immediately behind the wall.

The use of batter piles to ensure sufficient lateral support against lateral earth pressure is recommended in view of the nature of the upper 20 feet of the abutment foundations.

5.3 Bridge Approach Embankments

The grade presently proposed for the Willow Lake River bridge will result in embankments with a maximum height

5.3 Bridge Approach Embankments - Continued

of approximately 30 feet. These embankments will overlie the upper silty-sand and sandy-silt deposits which have a relatively high water content.

Some settlement of the embankments will occur and, to allow consolidation to take place prior to abutment construction, it is recommended that the abutments be constructed a minimum of two months prior to bridge construction.

The embankment should be compacted to a minimum density of 98 percent Standard Proctor density utilizing vibrating rollers. The maximum recommended lift thickness is 9 inches.

Data from the project hydrology consultants (Bolter, Parish, Trimble Ltd. of Edmonton) shows that occasional abnormally high water levels will occur due to ice jams on the Mackenzie River. Hence, the approach embankments must be designed for stability against rapid drawdown conditions. If the embankments are constructed from free-draining granular fill containing a minimum of fines, embankment sideslopes as steep as 2.5 horizontal to 1 vertical can be used. If the embankment fill is not free-draining, the maximum recommended embankment sideslopes are 3.5 horizontal to 1 vertical.

It is recommended that the embankment sideslopes be armoured with a minimum thickness of 18 inches of rip-rap having a minimum diameter of 6 inches. Size and thickness of the rip-rap should be increased in those areas of the embankment subject to high velocity water flow. Sufficient cobbles and boulders to

5.3 Bridge Approach Embankments - Continued

satisfy this requirement, as well as to provide riprap protection for the abutments, should be available in the adjacent borrow pits (392-No. 1 and 396-No. 1) at haul distances of approximately 1-1/2 miles.

APPENDIX

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LEGEND

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PHOTOGEOLOGIC AND BOREHOLE MAPPING

F	- Frictional soils (sand, gravel)
T	- Transitional soils (silt, mixed silty soils)
С	- Cohesive soils (clay)
0	- Organic soils (peat)
R	- Bedrock, outcrop or under shallow overburden
ICE	- Massive ground ice
	- Example - O/T organics overlying transitional soils
().	- Example - (ICE) possible or occasional ground ice
A	- Beach
Sand .	- Slides or slumping of slopes
X	- Spring
¥ 421-BT-1	- Test Trenches
• 421-C-D	- Centreline boreholes
• 421-B-J	- Borrow source boreholes
	- Outline of potential borrow sources A-1

NOTES RELATING TO PHOTOMOSAICS

- 1. The drill hole locations marked on the photomosaics are transferred from the locations marked on the airphotos used in the field and have been plotted relative to the topographic features and not in accordance with the actual mile posts. In some instances the borehole chainages as determined from field surveys do not agree with the mileages noted on the drawings.
- The photomosaics on which the data has been plotted were those supplied to Acres by D.P.W. on February 8, 1973.

DEFINITIONS

SYMBOL	
wl	- Liquid limit
wp	- Plastic limit
w _n	- Natural water content
ľp	- Plasticity index
psf	- pounds per square foot
pcf	- pounds per cubic foot
ĸ _o	- Coefficient of earth pressure at rest

UNIFIED SOIL CLASSIFICATION SYSTEM

SOIL CLASSIFICATION CHART

M	AJOR DIVISION	S	LETTER SYMBOL	TYPICAL DESCRIPTIONS
	GRAVEL	CLEAN GRAVELS	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	GRAVELLY SOILS	{little or no fines)	GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
SOILS	more than 50 % of coarse fraction	GRAVELS WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND- SILT MIXTURES
	RETAINED on no. 4 sieve	amount of fines)	GC	CLAYEY GRAVELS, GRAVEL-SAND- CLAY MIXTURES
	SAND AND	CLEAN SAND	sw	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
more than 50% of material is LARGER	SANDY SOILS	fines)	SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
siève size	more than 50% of coarse fraction	SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES
	PASSING no.4 sieve	amount of fines)	sc	CLAYEY SANDS, SAND-CLAY MIXTURES
			ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
FINE GRAINED SOILS	SILTS AND CLAYS	liquid limit LESS then 50	CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
Tore than 50 %			мн	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
of material is SMALLER than no. 200	SILTS AND CLAYS	liquid limit GREATER than 50	сн	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
sieve size			он	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIG	HLY ORGANIC SOIL	.S	PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

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NRC/ACFEL ICE CLASSIFICATION SYSTEM

ICE CLASSIFICATION CHART

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			BONDED	EXCESS ICE	ND	•
FROZEN SOIL	,		INDIVID CRYSTA INCLUS	UAL ICE ILS OR SIONS	V	¢
	SEGREGATED ICE IS VISIBLE BY EYE	v	ICE CO ON PAI	ATINGS RTICLES	Va	:
	(ICE I INCH OR LESS IN THICKNESS)		RANDI IRREGULARI ICE FORI	OM OR LY ORIENTED MATIONS	۷,	
			STRATIE DISTINCTL ICE FOR	IED OR Y ORIENTED MATIONS	V	i
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