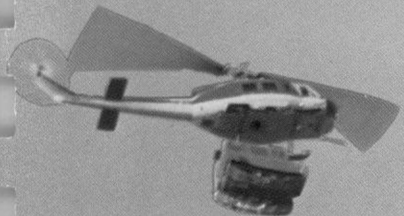


Public Works Canada

Travaux Publics Canada

83

Western Region

Region de l'Ouest

#42

42

SUPPLEMENTARY REPORT

GEOTECHNICAL INVESTIGATION

PROPOSED DONNELLY RIVER BRIDGE
ALTERNATE ALIGNMENT C-17, MILE 689.7
MACKENZIE HIGHWAY

PUBLIC WORKS CANADA

WESTERN REGION

C0012

SUPPLEMENTARY REPORT
FOUNDATION INVESTIGATION
PROPOSED DONNELLY RIVER BRIDGE
ALTERNATE ALIGNMENT C-17
MILE 689.7, MACKENZIE HIGHWAY

Submitted By: R. D. Cook, P. Eng.
Soils Engineer
Special Services
Western Region

TABLE OF CONTENTS

I PRELIMINARY

The initial foundation investigation at the Donnelly River was undertaken by E. W. Brooker & Associates Ltd. in the winter of 1973, and summarized in a foundation report dated August, 1973. Since that time, a tentative route revision has been proposed in the Chick Lake area, which would shift the crossing at the Donnelly River roughly 1¼ miles downstream. This alternate routing is shown on the 1"=1000 mosaic in Appendix A. Test borings at the revised crossing site were carried out by Public Works Canada in October of 1973 (2 Holes) and April, 1975 (2 Holes).

A profile of the revised crossing (Alternate Alignment C-17) showing the four boreholes drilled and the inferred subsoil stratigraphy has been included in Appendix A (Drawing No. A-1). Borehole logs are included in Appendix B.

II SUBSOIL CONDITIONS

Shale underlies both crossing sites at shallow depths. At the original site, data obtained by E. W. Brooker & Associates Ltd. revealed shale at depths in the order of 5' below the present stream-bed. On revision C-17 the shale was encountered roughly at the stream-bed level on both sides of the channel (Note - Drilling was not attempted in mid-channel as the river was open in April, 1975 and the bedrock profile below the channel on Drawing A-1 has been estimated.)

... 2

Permafrost was present at least to the shale surface. Moisture (ice) contents in the shale were low and it was impossible to confirm permafrost in the shale from examination of drill cuttings. The shale was categorized as soft to medium hard by the drilling crew. It is believed there is an extensive thaw zone directly below the river channel as there is a substantial year round flow which would promote thawing.

III PROPOSED STRUCTURE

The bridge consultants for the Donnelly River (Shawinigan Engineering Co. Ltd.) have recommended a bridge length in the order of 350-370 feet for the original crossing site with piers near the edges of the river channel, and abutments set back from the crest of the river bank. It is assumed a similar configuration would be employed at the tentative revised crossing site. Piers located near the edge of the stream flow may be subjected to lateral forces on the outside of the piers, as a result of unequal frost penetration during the winter - i.e. the rate and depth of frost penetration on the inside, or stream side, of the pier is much less than on the outside of the pier due to the warming effect of the water, thus frost heave forces can be generated with lateral (inward) tilting of the piers. As shale is very shallow at the Donnelly River sites this situation should not develop, however backfill material placed on the outside of piers should be coarse granular to avoid the possibility of heaving forces.

... 3

IV FOUNDATION RECOMMENDATIONS

Subsurface conditions at the tentative revised crossing site are practically identical to those at the original site investigated by E. W. Brooker & Associates Ltd. in 1973. The E.B.A. report covers most geotechnical aspects of bridge design and construction in permafrost terrain and, with the exception of the comments below on the bearing elements for the structure, the recommendations in the E.B.A. report have application at either site.

Included herein are seven pages taken from the E.B.A. report which deal only with the subsoil conditions at the original site and recommendations for design and installation of the foundation piling. Three types of pile foundations are considered feasible by E.B.A.:

(1) 50 foot, steel H-piles (12BP53) driven with an energy of 24,000 ft. lbs. to a set of 1 inch per blow, designed for an allowable static load of 70 kips.

(2) 50 foot, closed-end, steel pipe piles (10" diameter @ 40 lbs./ft.) placed in pre-drilled lead holes roughly 9" in diameter, and driven with an energy of 24,000 ft. lbs. to a set of 1 inch per blow, and designed for an allowable static load of 60 kips.

... 4

(3) 50 foot, size 12 Douglas fir piles, placed in predrilled lead holes the size of the pile tip and driven with an energy of 24,000 ft. lbs., to a set of 1 inch per blow and designed for an allowable static load of 40 kips.

The suitability of all three types of piling is concurred with, however it is considered that the recommended loadings and/or pile lengths are conservative. It is anticipated that the steel piling will meet practical refusal in the shale at much shallower depths than indicated by E.B.A. - probably at penetration of 20 feet or less (i.e. sets of 8-10 blows per inch for energy of 24,000 ft. lbs.). At practical refusal the piling may be designed for approximately 75% of the full structural strength of the pile section acting as a column. Steel H-piling are considered the most practical at this site as installation will be most simple - a heavier section than the 12BP53 is recommended to withstand the hard driving stresses anticipated.

The proposed method of installation for timber piling is considered impractical as it will be impossible to drive a timber pile without damage into the dense frozen shale with a lead hole no larger than the pile tip. (Past experience in driving timber piles into lead holes in much less dense subsoil in permafrost terrain has proven most difficult). If timber piles are employed it is recommended they be considered as end-bearing

... 5

only and designed for a load of 40 kips. A pile penetration of 20 feet into the shale would ensure sound bearing, and it is recommended a lead hole the size of the pile butt be drilled from 0 to 15 feet, and a lead hole the size of the pile tip be drilled from 15 to 20 feet. (0 assumed to be the shale surface or the base of the pier or abutment). This sizing of lead holes should allow the pile tip to be driven to refusal onto shale and permit development of a load capacity of 40 kips, plus provide lateral pile support.

No consideration was given by E.B.A. to placing the piers and/or abutments directly upon the shale which is a viable alternative to piles. There is weathered shale at the bedrock surface, however suitable bearing should be available at depths of 3' into the shale. Design bearing pressures of up to 12 kips/ft.^2 are considered feasible on relatively sound shale. The shale may tend to weather if exposed to air or water hence the surface of the excavation should be kept free from water and concrete should be placed relatively soon after excavation into the shale. It is considered likely that the shale will be frozen adjacent to the river channel, however, as it is at a very low moisture content, the volumetric heat capacity will be low and the heat of hydration from the mass concrete will likely cause thawing into the shale and ample time will be available for adequate concrete curing before re-freezing. To facilitate

... 6

curing, type III high early strength cement and, possibly some calcium chloride additive, should be utilized. There is concrete aggregate available in an esker roughly 2 miles east of the route at Mile 703 (15 miles distant), near Elliot Creek at Mile 660 (30 miles distant), and at Ft. Good Hope - Mile 725 (35 miles distant).

V EXTRACTS FROM E. W. BROOKER & ASSOCIATES LTD. REPORT

This report should be considered as an addendum to the foundation report by E. W. Brooker & Associates Ltd. for the Donnelly River, and the overall report applied to either the original or alternate crossing sites. The following seven pages are taken from the E.B.A. report.

RDCook
R. D. Cook, P. Eng.
Soils Engineer
Special Services
Western Region

2.3 Subsurface Conditions

Based on observations from the boreholes, inferred stratigraphic sections have been compiled and are presented as Drawings No. A-4 and A-5, Appendix A. The generalized centerline stratigraphy noted at the site is summarized in the following Sub-Sections.

2.3.1 Stratigraphy on Centerline - North Valley Wall

Material Description		Approximate Average Depth Below Existing Grade (FT)	Approximate Range of Thickness (FT)
ORGANIC SILT, ORGANIC CLAY, PEAT,	- black to dark brown, organic, fibrous, clayey and silty, V - (5% to 15%), moisture content (M.C.) avg. = 85%	0 - 3.2	0.5 - 7.0
CLAY OR SILT (TILL)	- medium to dark brown, silty or clayey, sandy and gravelly, low plastic, V - (0 to 10%), M.C. (avg.) = 35%	3.2 - 9.0	4.0 - 6.0
SILT (WEATHERED SHALE)	- grey, clayey, shale fragments, soft, low to medium plastic, NB, M.C(avg.) = 14%	9.0 - 21.6	10 - 17
SHALE	- grey, clayey silt to silty clay shale, weathered, soft, low to medium plastic, NB, M.C. (avg.) = 9.7%	21.6 - Depth of Penetration	Not Established

The following additional information, which may influence design or construction decisions, was also obtained during the investigation at the subject site.

1. The maximum depth of borehole penetration was 88 feet.

2. A grey silty clay layer was encountered in Boreholes 689-S-5 and 689-S-1, beneath the organic layer, and overlying the silt (weathered shale) and shale, respectively. The clay layer averaged 4.75 feet in thickness and exhibited excess ice contents estimated up to 30 percent (NB to V). The moisture content of the clay stratum averaged 27.3 percent. At the location of Boreholes 689-S-5 and 689-S-1, the shale was noted at depths of 8.5 feet and 12.5 feet, respectively.

3. Due to a lack of downhole temperature measuring equipment and the absence of visible ice at depths greater than an average of about 6 feet in the boreholes, it was difficult to accurately determine to what depth permafrost extended in the borings. However, it is believed that all borings encountered frozen material to the maximum depth of penetration and that NB is the correct ice classification for the silt (weathered shale) and shale strata.

... 9

4. Borehole 689-S-1 was the only borehole in which an unfrozen zone could be positively identified. This zone was noted at a depth of about 8 feet, where free water was noted to be entering the borehole.

2.3.2 Stratigraphy on Centerline - South Valley Wall

Material Description		Approximate Average Depth Below Existing Grade (FT)	Approximate Range of Thickness (FT)
PEAT	- dark brown to black, clayey, silty, organic, fibrous, V-(15% - 20%), M.C. (avg.) = 53%	0 - 2.7	1.0 - 4.0
CLAY	- medium brown to grey, silty, some sand, low plastic, V-(10% - 15%) to NB, M.C. (avg.) = 31.9%	2.7 - 14	4 - 14.0
SHALE	- grey, clay-silt shale, some sand, weathered, soft, low plasticity, NB, M.C. (avg.) = 10.6%	14 - Depth of Penetration	Not Established

The following additional information, which may influence design or construction decisions, was also obtained during the field investigation.

1. The maximum depth of borehole penetration was 38 feet.

2. A medium brown to grey, silty, fine grained sand layer, was noted in Boreholes 689-S-2 and 689-S-12, overlying the shale stratum.

... 10

3. All boreholes are believed to have encountered frozen material to the maximum depth of penetration.

III. CONCLUSIONS AND RECOMMENDATIONS

3.1 Foundation Types

The following foundation types are believed to be feasible for founding a bridge structure at the subject site. At present, preference has not been given to any of the types listed, as final selection of a foundation system should be determined in conjunction with economic and structural design considerations.

1. Driven 'H' Piles.
2. Closed End Pipe Piles Driven in Prebored Holes.
3. Timber Piles Driven in Prebored Holes.

3.2 Foundation Design Parameters

The recommended foundation types listed in Subsection 3.1, may be designed in accordance with the following parameters.

3.2.1 Driven Steel H-Piles

The design of steel H-piles is dependent upon the type of support that is provided the pile in or on the bearing material. In the case of the Donnelly River bridge crossing, it is considered that the shale bedrock material provides the only suitable bearing support for foundation elements. However, as the insitu density or consistency of the shale has not been precisely established; nor have the thermal conditions of the shale been accurately determined, only tentative pile designs or driving criteria can be provided at this time.

... 11

As a guide to the establishment of a pile design, it is recommended that standard H-piles 50 feet in length, with a minimum nominal size of 12 inches by 12 inches, and a minimum weight of 53 pounds per foot (12BP53), be considered for preliminary design purposes. It is believed that the suggested pile section can be driven, with an energy of 24,000 foot pounds, to a set of 1 inch per blow, measured over the last foot of driving. It is believed that piles driven to these specifications will permit an allowable static design load of 70 kips to be used. Although preboring is not considered necessary for the installation of steel H-piles in frozen ground, at this site, it may be desirable in very hard seasonally frozen ground to ensure the alignment of the driven pile section. This will be particularly true if very long sections are to be driven.

It is essential that the fill be placed to final grade before the piles are driven or pre-bored in order to prevent damage to the pile and to ensure proper compaction of the fill, which will limit negative skin friction loads on the pile. On site inspection and supervision of the driving of test piles or the initial piles of the foundation system is considered absolutely necessary in order to establish the final design bearing capacity. It is also considered essential that a pile driving record be maintained for all piles. The driving record of all piles should be reviewed as is practical, by the geotechnical consultant, to ensure the design intention has been realized.

3.2.2 Prebored Driven Closed End Pipe Piles

As in the case of steel H-piles, pipe pile design is based on the means of support achieved. Preboring of closed end pipe piles, driven into the shale stratum, is considered necessary to facilitate the driving operation and maintain alignment of the piles. The prebored hole size should be 85 to 90 percent of the outside pile diameter to ensure a 'snug' fit,

... 12

and should extend the full length of the intended pile penetration. It is essential that the fill be placed to final grade before the piles are driven or pre-bored in order to prevent damage to the pile and ensure proper compaction of the fill, which will limit negative skin friction loads on the pile.

For preliminary design purposes, it is recommended that closed end pipe piles with a minimum length of 50 feet, a minimum nominal diameter of 10 inches, and a minimum weight of 40 pounds per foot be considered. It is believed that the suggested pile section can be driven, with an energy of 24,000 foot pounds, to a set of 1 inch per blow, measured over the last foot of driving. It is believed that piles driven to these specifications will permit an allowable static design load of 60 kips to be used. Driven piles must penetrate to at least the full pre-bored depth. As for steel H-piles, inspection of the driving of test piles, or the first few piles of the foundation system is considered absolutely necessary to confirm or alter the design bearing capacity recommended herein. A pile driving record must be kept for all piles for immediate review by the geotechnical consultant.

3.2.3 Prebored Driven Timber Piles

For preliminary design purposes, it is recommended that Number 12 Douglas Fir timber piles, with a minimum length of 50 feet, be considered for foundation support. The piles should be pressure treated with creosote and should have a minimum creosote retention of 12 pounds per cubic foot.

It is essential that the fill be placed to final grade before the piles are driven or pre-bored to prevent damage to the pile and to ensure proper compaction of the fill, which will limit negative skin friction loads on the pile. Preboring into the shale stratum, is considered essential to limit pile damage in frozen ground. Prebored holes should have a maximum

... 13

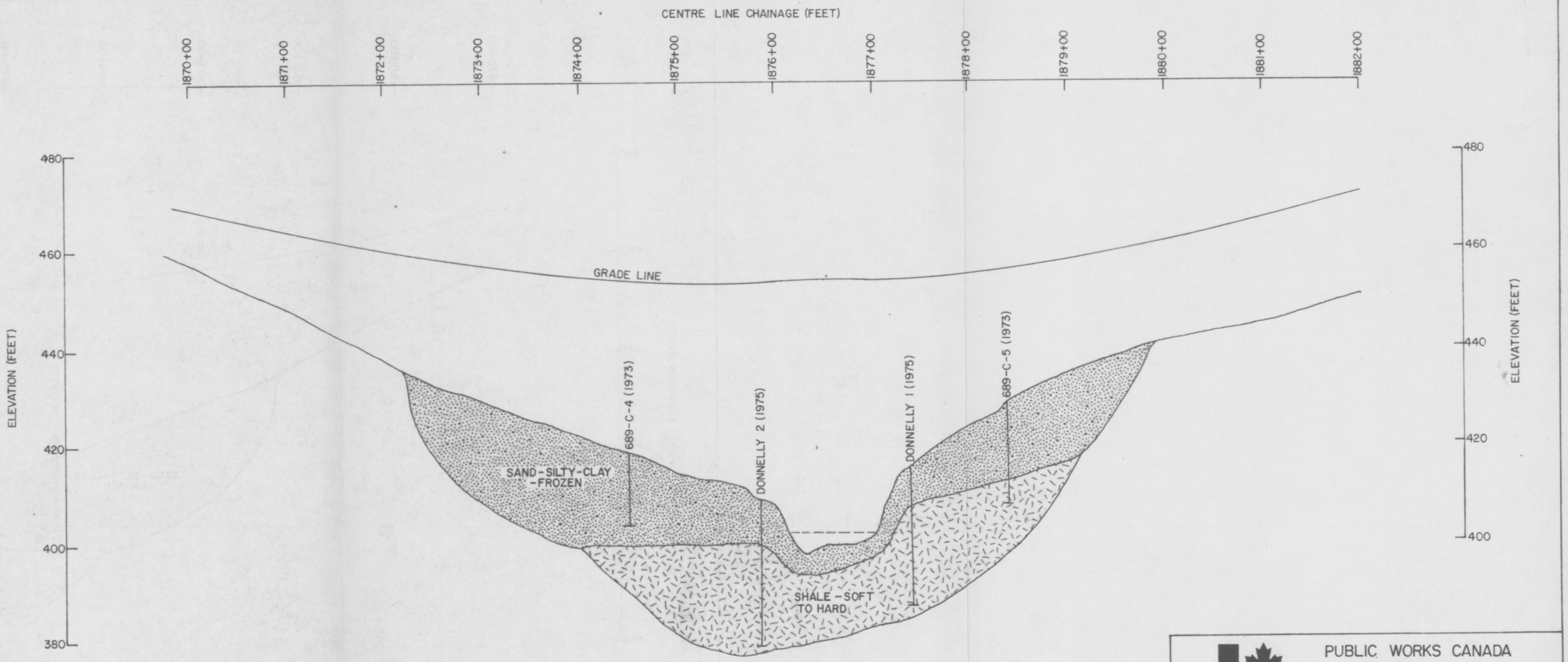
diameter equivalent to the pile tip diameter to permit a snug fit, and should extend for the full depth of anticipated pile penetration. If excessive driving resistance is encountered, overboring of approximately the upper half section of the hole may be required. Details of the overbore should be determined in the field, on the basis of driving records obtained from test piles.

As the pile capacity will depend on the preboring and overboring of the pile holes, which remains to be proven in the field, the set to which the piles can be driven cannot be predicted. However, if the suggested pile section can be driven, with an energy of 24,000 foot pounds, to a set of 1 inch per blow, measured over the last foot of driving; it is believed that an allowable static design load of 40 kips may be used. Piles must penetrate to at least the full prebored depth.

If timber piles cannot be driven to the foregoing specifications, their design bearing capacity must be determined on the basis of field driving records or load tests. Field inspection by qualified personnel is considered necessary during the preboring, overboring and driving of test piles or the first few foundation piles, to establish or alter the preliminary design capacity and method of installation recommended herein. A driving record should be kept, for all piles installed, for immediate review by the geotechnical consultant.

3.3 Negative Skin Friction

The effect of negative skin friction, on individual piles and pile groups, will be dependent upon the occurrence and magnitude of settlement within both the fill surrounding the piles and the natural subgrade materials. At the crossing site, it is considered that all subgrade materials, with



NOTE: 1. STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN ASSUMED.
SCALE: VERT. 1"= 20'
HORIZ. 1"=100'

PUBLIC WORKS CANADA
WESTERN REGION

DONNELLY RIVER CROSSING
ALTERNATE ALIGNMENT (C17)
MILE 688.9
MACKENZIE HIGHWAY

DWG NO A-1

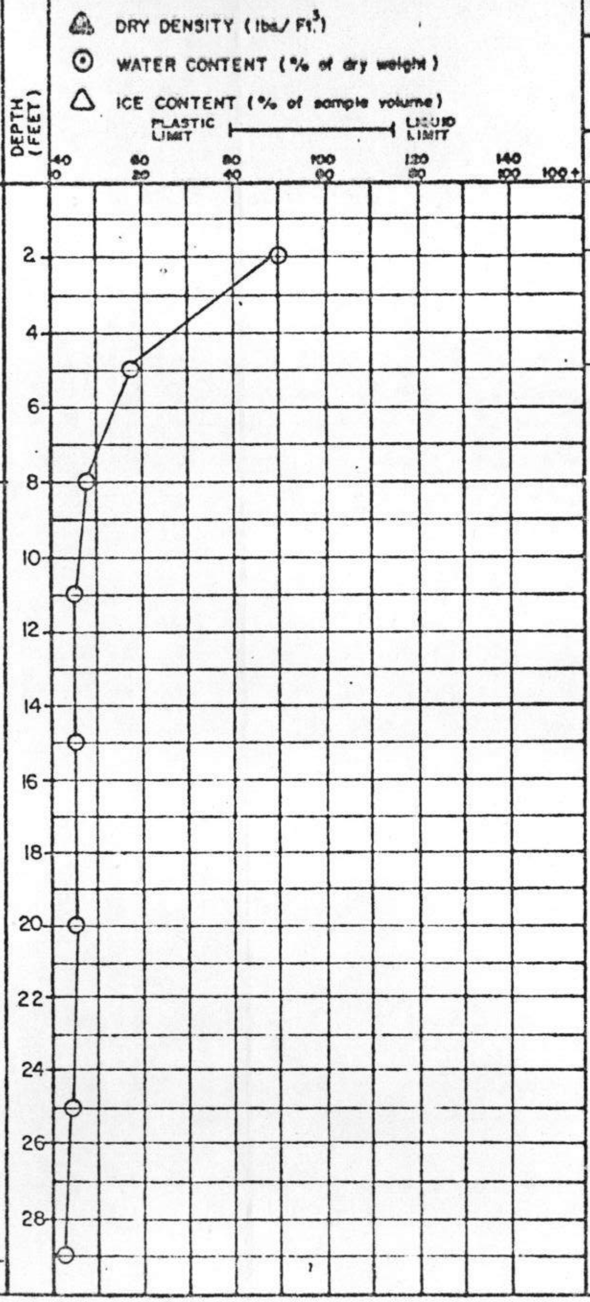
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DRILL HOLE REPORT

SITE: DONNELLY RIVER - SOUTH BANK

FIELD ENG.
DATE DRILLED. 9/4/75
AIRPHOTO NO.
CHAINAGE.
OFFSET. ℄
TECH. PRONYCH
RIG. AIR
SURFACE DRAINAGE.
VEGETATION. SPRUCE
ELEV.
TEST HOLE #1 (1975)
MILE 689

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	CLAY %	SILT %	SAND %	GRAVEL %	RELATIVE THAWED MOISTURE CONTENT	REMARKS
					Pt	Peat 4"								
						Clay - Silty								
2						Sandy			70	-	28	2	Wet	
4					CL	Pebbles		V_x						
					Cl	Low - Medium Plastic			73	-	26	1	Moist	
6														
8						8' Shale - Soft							Damp	
10							F							
								Nbn					Damp	
12														
14														
						Soft - Hard							Humid	
16														
18														
20													Humid	
22														
24														
													Humid	
26														
28														
						Siltstone & Shale							Dry	
						Bottom of Hole - 29'								



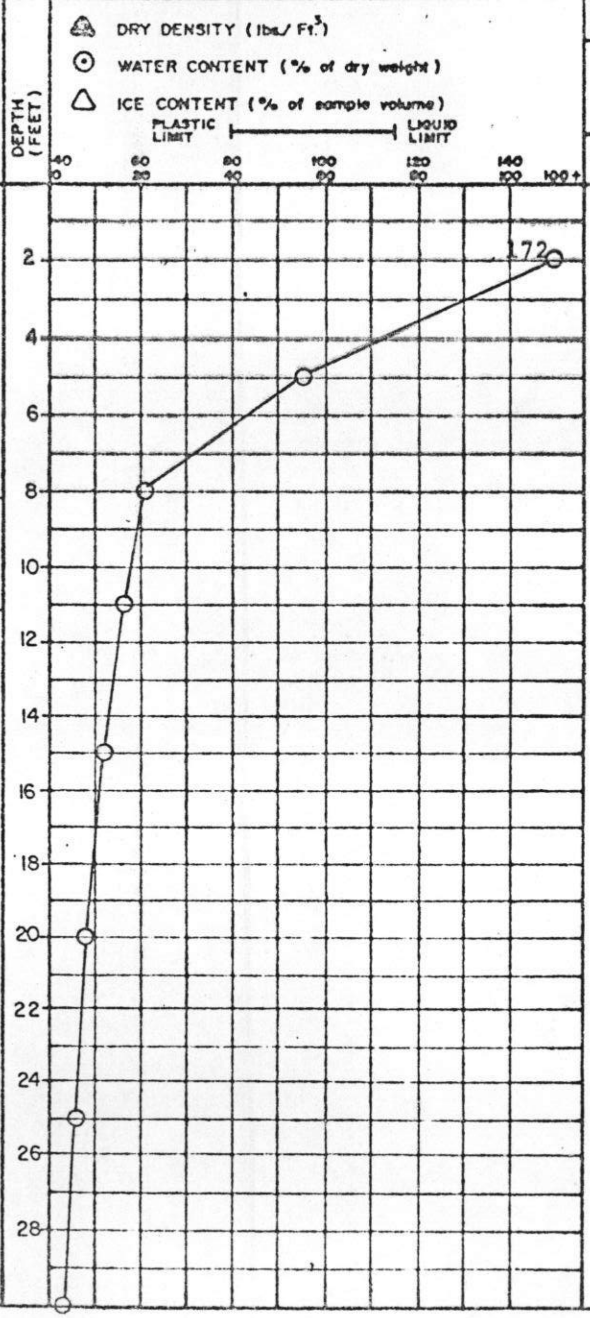
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DRILL HOLE REPORT

SITE: DONNELLY RIVER - NORTH BANK

FIELD ENG.
DATE DRILLED. 10/4/75
AIRPHOTO NO.
CHAINAGE.
OFFSET. ℄
TECH. PRONYCH
RIG. AIR
SURFACE DRAINAGE.
VEGETATION. SPRUCE 8"
ELEV.
TEST HOLE #2 (1975)
MILE 689
REMARKS

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 %	RELATIVE THAWED MOISTURE CONTENT
					Pt	Peat — 4"							
						Clay - Silty							
2						Organic to 4'							Sat.
4					CL	Silty Sandy							
						Low - Medium Plastic			-86	-	14	0	Sat.
6					Cl			VS					
8						Shale Fragments			-45	-	50	5	Wet
						— 9'							
10						Shale - Soft		V_c-V_r					
							F						Moist
12													
14													
													Damp
16													
18													
20													Damp
22								Nbn					
24													
													Humid
26													
28													
													Humid



Bottom of Hole - 30'

3074

DS-14.5.74 b

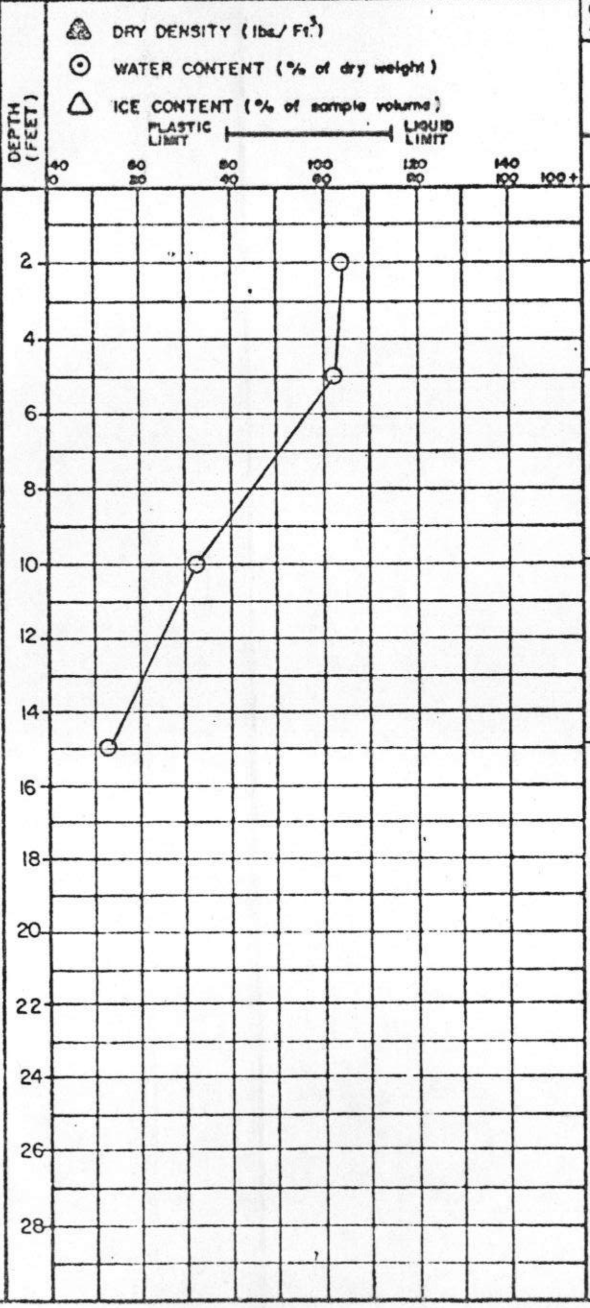
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DRILL HOLE REPORT

SITE: DONNELLY RIVER - NORTH SIDE

FIELD ENG.
DATE DRILLED. OCT./73
AIRPHOTO NO.
CHAINAGE.
OFFSET. ℄
TECH. PRONYCH
RIG. HELI-AIR
SURFACE DRAINAGE.
VEGETATION. SPRUCE - 8"
ELEV.
TEST HOLE #4 (1973)
MILE 689

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 %	RELATIVE THAWED MOISTURE CONTENT	REMARKS
					Pt	10" Organic Material								
2					CL	Clay - Silty Pebbles Low - Medium Plastic		V$_s$	89	-	11	-	Sat.	
					Cl		F		94	-	6	-	Sat.	
10					CL	Clay - Silty, Sandy Low Plastic			81	-	19	-	Wet	
						Trace of Coal, etc.		V$_c$ - V$_r$	53	-	45	2	Moist	
16						Bottom of Hole @ 16'								



DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DRILL HOLE REPORT

SITE: DONNELLY RIVER - SOUTH SIDE

FIELD ENG.	DATE DRILLED. OCT./73	AIRPHOTO NO.	CHAINAGE.
TECH. PRONYCH	RIG. HELI-AIR	SURFACE DRAINAGE.	VEGETATION. SPRUCE
OFFSET. ℄	ELEV.	TEST HOLE #5 (1973)	MILE 689

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 %	RELATIVE THAWED MOISTURE CONTENT	REMARKS
					Pt	1' Organic Material								
2						Clay - Silty, Sandy		V$_c$ - V$_r$	– 75	–	24	1	Wet	
4						- Medium Plastic								
						- Grey	F		– 47	–	48	5	Moist	
6					Cl	- Occasional Cobbles & Boulders								
10									– 64	–	35	1	Moist	
15									– 52	–	46	2	Damp	
15½'						Shale Grey Firm, Hard, Dry		Unable to Detect Permafrost in Shale						
20									– 6	–	71	23	Damp	
21'						Bottom of Hole @ 21'