CONSULTING ENGINEERS
AND PLANNERS

Reid, Crowther & Partners Limited

7410 Blackfoot Trail S.E., P.O. Box 5600, Postal Station "A", Calgary, Alberta, Canada T2H 1X9, Telex 038-22780, Telephone 253-3301, Area Code 403

PLEASE REFER TO FILE No. 9554-3

January 11, 1974

Department of Public Works
10th Floor, One Thornton Court
P.O. Box 488
Edmonton, Alberta
T5L 2K1

Attention Mr. F. E. Kimball
Project Manager NWT Roads
Western Region

Gentlemen:

Mackenzie Highway - Preliminary Engineering Phase 1B
Whitesand Creek Bridge Mile 459.7

We are pleased to present herein our Phase 1B on the Whitesand Creek Bridge which has been prepared in accordance with Mr. Kimball's letter of October 3, 1973.

As instructed, the Report includes a summary of the hydrology and geotechnical reports; impact statements on the temporary and permanent crossings by the environmental consultant; and, a brief description of the proposed structure and alternate systems considered together with preliminary drawings and cost estimates.

We trust that the content of our Report provides a basis for approval in principle and authorization to proceed with final design.

Yours very truly,

R. C. Aitken, P. Eng.
Manager, Transportation Division

RCA/mm

Enclosures

MACKENZIE HIGHWAY

PRELIMINARY ENGINEERING PHASE 1B

WHITESAND CREEK BRIDGE MILE 459.7

INDEX

MACKENZIE HIGHWAY

PRELIMINARY ENGINEERING PHASE 1B

WHITESAND CREEK BRIDGE MILE 459.7

INTRODUCTION:

The proposed crossing of Whitesand Creek is in a stable reach of the creek approximately 1,000-feet upstream from the Mackenzie River. At this location the main valley is approximately 900-feet wide and 50-feet deep with water 50-feet wide and 1 to 2-feet deep flowing clear and fast over a bed of gravel and boulders. There is evidence of moderate driftwood. Approximately 500-feet downstream from the highway centre line the channel splits around an island which is a likely cause of ice jams on Whitesand Creek. Extensive ice damage was noted near the proposed crossing about 12-feet above the stream bed.

Forest stands in the vicinity of the crossing extend to the top of the banks and consist of white spruce and aspen to 40-feet in height. Past disturbance from winter road construction, however, has been so extensive that anticipated disturbance due to construction of the new bridge will be insignificant.

Whitesand Creek has a catchment area of 130 square miles extending to the Franklin Mountains. At the design discharge of

9,200 c.f.s. the average velocity of the flow through the bridge opening will be 12 f.p.s. and the anticipated depth of bed scour is about 2-feet. Flow, at all stages is confined within the present banks.

The deeply incised valley controls the vertical alignment of the highway at this crossing and the clearance from the underside of the girder flanges to the estimated level of the backwater caused by ice jams on the Mackenzie River is 10-feet (Dwg. P601). Recorded extremes in the level of the Mackenzie at Wrigley, approximately 14 miles south from Whitesand Creek, have been used to estimate the elevation of this backwater condition.

Fish access to the Mackenzie River is good and fishery potential is high with fish population including the following species: grayling, round whitefish, lake chub, long-nose sucker, and slimy sculpin. It is not known whether this creek is used as a spawning area.

Since the completion of the preliminary reports, archaeological surveys have indicated that the low to medium priority given to this site should be revised to high discovery probability. Four pre-historic sites have been located in the vicinity and close surveillance should be maintained throughout construction.

The profile of the highway has been changed since December, 1972 and this change has added 10-feet to the length of the bridge.

A further modification in this submission moved the piers out of the active stream channel lessening the effect of pier construction on the river and permitting a more flexible construction schedule. Comments by F. F. Slaney & Company Limited on the environmental impact of the proposed bridge and the temporary crossing are included in Appendix B.

Although the geotechnical information was gathered under adverse conditions, the information obtained is quite adequate for the design of the bridge piers and abutments. Results of the investigation can be summarized as follows.

Below the river, the soils consist of alternating layers of sand and gravel for a depth of approximately 60-feet. At this elevation a sandstone and shale bedrock was confirmed. One of the two holes drilled from the floor of the stream valley indicated the presence of permafrost to a depth of 30-feet at which level the water table was encountered. Below this level the soil was classified as semi-frozen. The second hole, drilled to a depth of 40-feet, did not disclose any permafrost.

Boring logs are included in Appendix C of this Report.

DESIGN CONSIDERATIONS:

Proposed Structure.

The proposed bridge is a three span structure with spans of 60'-130'-60'; the deck is cast-in-place concrete supported by two welded plate girders with composite action in regions of positive bending moment. Cast-in-place concrete piers and abutments supported on piled foundations are recommended for the substructure. Dwg. P601 shows the proposed layout.

Straight girders with tapered end spans are proposed as being appropriate for a structure of this size. A longer structure, such as the Ochre River does benefit from the use of curved soffits.

The piers, in particular the north pier, are considerably shorter than those for the River Between Two Mountains and the Ochre River. A simple tapered pier has therefore been proposed rather than the more complicated form used at the other sites. In order to relieve the large flat surfaces of the piers, particular attention should be given to obtaining a symmetrical arrangement of form panels and ties and the concrete should be sandblasted.

"Spill through" abutments have been shown (Dwg. P602) on the assumption that approach embankments may not be in place when the bridge is constructed; if in fact embankments are placed ahead of bridge construction, then a more economical "perched abutment", supported by piles driven through the approach fills, will be substituted.

Subsurface conditions are such that piled foundations are recommended for both piers and abutments - (a) to protect piers against local scour and (b) to reduce differential settlement between piers and abutments to a negligible amount.

For the purpose of this preliminary design and cost estimate H-piles have been assumed although we share the soils consultant's reservations on the feasibility of successfully driving them through the dense gravel layers. There is a distinct possibility that some H-piles will "hang up" and that additional piles will have to be driven since we doubt that pile testing during construction is really practical at these remote sites. Therefore, it would appear that extra H-piles will have to be delivered to each bridge site if lengthy delays during construction are to be avoided. A preliminary investigation indicates that open ended pipe piles drilled to bedrock may be a more predictable and economical solution at this and other bridge sites along the east bank of the Mackenzie and will be considered during final design.

In consideration of the fact that many of the bridge sites along this stretch of the highway have similar subsurface conditions we would recommend that a comprehensive program of pile driving and load testing be carried out at one or two representative sites. Such a program would investigate H-piles installed by dynamic and vibrating equipment and open ended pipe piles seated into bedrock and would provide:

- a more rational basis for design;
- valuable information to contractors tendering on the project.

The riverbanks adjacent to the bridge piers will be protected by riprap as indicated on the Drawings - limits of riprap will be determined during the final design phase of the project. It is understood that embankments will be grassed except beneath the structure where slope protection will be provided.

It has been suggested (M.H.E.W.G. Report #2) that more information on ice and backwater levels could be obtained from a field check of the effect of the island at the mouth of Whitesand Creek. Since the highway profile allows 10-feet of clearance above the design high water level, information obtained from the check will not affect the design of the bridge. Further field investigation is unnecessary.

Design Criteria.

Specifications:	C.S.A. S6
	A.A.S.H.O.
	A.W.S. D1.1
Materials:	Structural Steel - C.S.A. G40.21 Grade 50A
	Deck Concrete - f'c - 4,000 p.s.i.
	Substructure Concrete - f'c - 3,000 p.s.i.
	Reinforcing Steel - C.S.A. G30.12 60 Grade
Loading:	Live Load - H.S. 25 + Impact
	Future Wearing Surface - 30 p.s.f.
	Ice Pressure - 250 p.s.i.
	Ice Thickness - 5'-0"
Piles:	70 Ton Compression (Group I C.S.A. S6)
	100 Ton Compression) (Group VIII C.S.A. S6) 30 Ton Tension)

Alternate Systems.

Before selecting the three span structure shown, four deck systems and a three span cantilever structure were evaluated. Deck systems considered were:

- concrete deck, three girders;
- concrete deck, two girders with transverse floor beams;
- concrete deck, two girders without transverse floor beams; and,
- open steel deck, two girders with transverse floor beams.

The concrete deck/two girder system proved to be more economical than the other concrete deck system and the welded steel deck; in estimating the cost of the open steel deck system, a 6½-inch Armco welded deck was considered. From discussions with the manufacturer, it appears that a steel deck could be installed for around $12.00/sq. ft.; however, the plate girders and floor beams required to support a steel deck would be in the order of 10 lbs./sq. ft. heavier than the plate girders required to support the concrete deck so that the relative cost of a steel deck would be in the order of $19.00 - $20.00/sq. ft. (assuming structural steel at $0.75/lb.). Therefore deck concrete could cost up to $500.00/cu. yd. including formwork (but excluding reinforcing steel) before the steel deck would be economically competitive.

From this preliminary investigation it is evident that the steel deck, although structurally adequate, will be considerably more expensive than a concrete deck and, in addition, the riding qualities of the steel deck will be inferior to those of a concrete deck. The alternate concrete and steel deck two girder systems with floor beams are shown on Dwg. P604.

A three span cantilever system was investigated in which the cantilevered end spans varied between 1/6 and 1/7 of the overall bridge length and the centre span increased from 130-feet, as proposed, to 180-feet. However, there was no economic advantage in this system, since the weight of structural steel required for the longer centre span increased the cost of the superstructure sufficiently to offset savings which might be realized by reducing the scale of the abutments. Furthermore, it was found that live load deflections were greater than would be considered desirable for a structure of this type, being in the order of 1/150 of the cantilever span (-2¼-inch, +1-inch). Therefore, this system was discarded in favour of the proposed conventional structure, although it should be noted in passing that this cantilever system would be suitable for shorter structures in the range of 175-feet.

TEMPORARY CROSSING:

The proposed location of the temporary crossing is approximately 500-feet upstream from the permanent bridge at the existing winter road crossing.

The proposed structure, a single span Bailey bridge supported by rock filled timber cribs, is shown on Dwg. P603. The temporary bridge is designed to carry a maximum live load of 52 tons (D9G Cat).

The underside of the structure has been set at an elevation 3-feet above the estimated high water level on Whitesand Creek. At this height, the temporary bridge will have little effect on the river at flood stage and it will not be subjected to damage during ice breakup; however, it should be noted that the structure and the approaches may be inundated for a short period of time each year by backwater conditions on the Mackenzie River.

The hydrology consultant has advised that there are, at present, no reliable methods of predicting backwater conditions due to ice but there is a possibility that inundation of the bridge could occur every spring. However, he further notes that the bridge will be above the ice run on the Whitesand and that he would not anticipate damage to the structure because of backwater conditions if the bridge is well anchored against uplift forces.

Alternatively, the bridge can be raised and lengthened but this would of course increase the cost significantly.

Local scour at the timber cribs is anticipated and, since pile driving is impractical, a protective apron of riprap will be required. However, if sufficient rock is not available, a filter fabric protected by triple-twist wire mesh is recommended - the mesh will be attached to the timber cribs and weighted at its extremities by drill stem or other appropriate means.

The elevation of the bridge is such that the approaches can be tied to existing grade or the highway grade depending on whether or not the highway embankment is in place when bridge construction commences.

SCHEDULING:

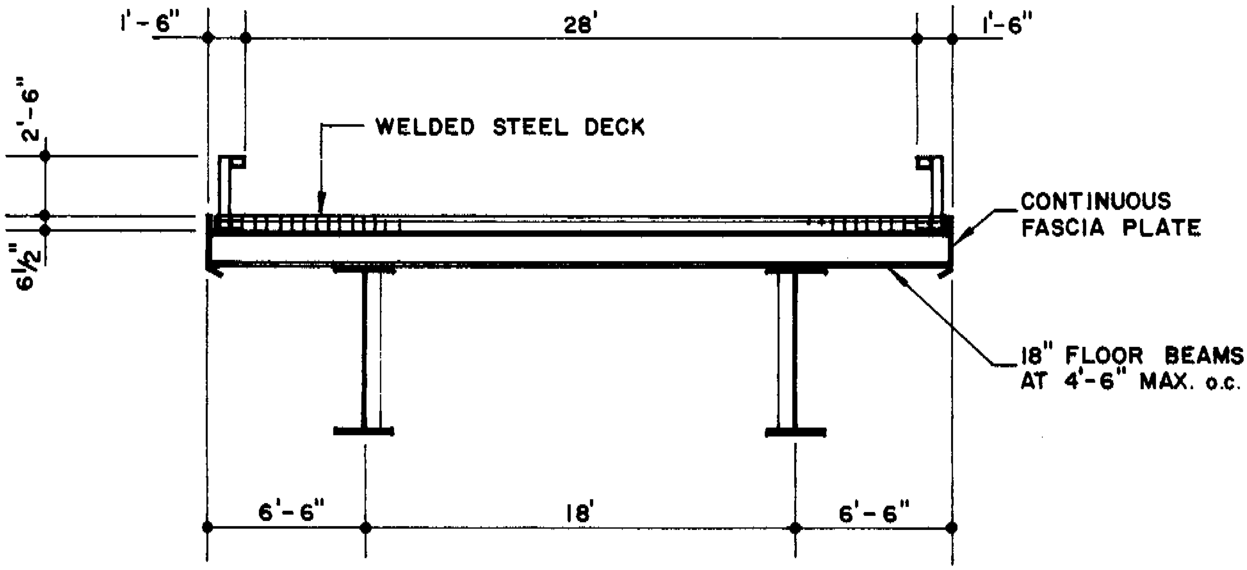
The following schedule assumes that highway construction will commence in the spring of 1975 and that all bridges on this section of the highway will be completed in the fall of 1976.

Allowing six months for structural steel delivery the tentative schedule will require bridge drawings and contract documents to be completed not later than October, 1974 and possibly sooner depending on the period of time required for approvals.

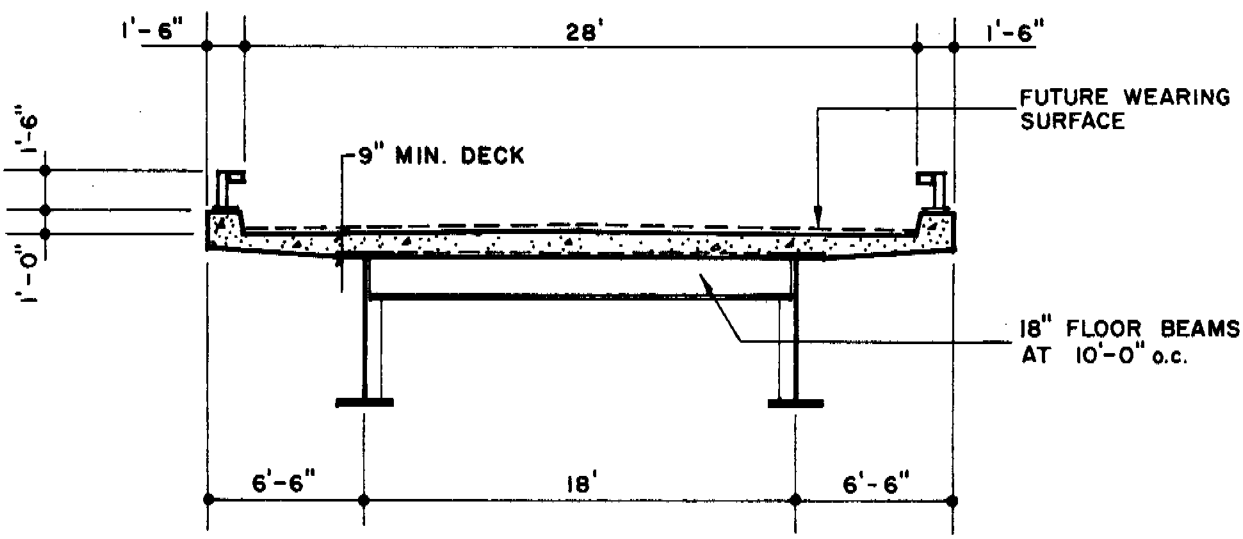
Schedule.

- Deliver concrete aggregates to Whitesand Creek mouth - Fall 1974.
- Construct temporary bridge - Winter 1974-75.
- Start pier construction - February, 1975.
- Start abutment construction - May, 1975.
- Erect structural steelwork - Winter 1975-76.
- Place deck and complete structure - Summer 1976.

It would appear that completion of the four bridges between Mile 411.0 (River Between Two Mountains) and Mile 460 (Whitesand Creek) by late 1976 is impossible unless some of them are designated for winter construction and each phase of construction is carefully scheduled. We realize that these decisions will be made as part of an overall construction plan which will cover all aspects of the project; to assist the Department in this major undertaking we have prepared a bar chart showing a possible construction schedule for these bridges. This schedule assumes that all bridges will be let as one contract.



STEEL DECK



CONCRETE DECK

ALTERNATE DECK SYSTEMS

9554 – P604

WHITESAND CREEK BRIDGE:

Introduction.

The preliminary bridge design has three spans supported by two riverbank piers. Both piers are located well above the low water level. The north pier is located at the September, 1972 low water level and the south pier slightly higher.

Environmental Impact.

The environmental impact appears negligible other than for fish. Fish activity appears concentrated from early May to mid-October. There is a probable upstream migration of round whitefish from mid-October to mid-December.

Little wildlife sign was observed in this area.

Recommended procedures are:

- The preferable time for any river disturbance is from mid-December to May 1. Little disturbance of fish is anticipated if encroachment on the river is minimal from mid-October to mid-December.
- Restore the disturbed channel to its original contour.
- Erect the spans when the river is frozen.
- Riprap should not be taken from the river bed.
- Drainage from the cut on the north bank should be diverted away from the foundations.

Summary.

The design submitted has minimal environmental effects if no stream bed sources of riprap are utilized.

WHITESAND CREEK TEMPORARY CROSSING:

Introduction.

The proposed temporary crossing bridge for Whitesand Creek is located approximately 600-feet upstream from the proposed permanent highway crossing. The bridge is a single span design supported by rock-filled timber cribs located above the low water level on either shore.

Environmental Impact.

Other than for fish the environmental impact appears negligible.

The cribs will cause a slight increase in current velocities in high water conditions. The south pier will require the most protection. Any construction activities encroaching on the creek cross section are preferable between January and April. August to mid-October is another period of low fish activity. By erecting the span when the creek is frozen the least possible disturbance will be caused to the river bed and banks.

Prehistoric sites are known in the immediate area therefore surveillance is required during any excavation.

Summary.

There will be minimal environmental effects of the cribs and embankment riprapping can be implemented during low water with no encroachment on the running stream and the spans erected on the ice when the river is frozen.

The effect of taking riprap for bank protection and crib fill from the channel cross section would be detrimental depending on the location of the source and amount required. Alternate sources of riprap should be considered.

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:-

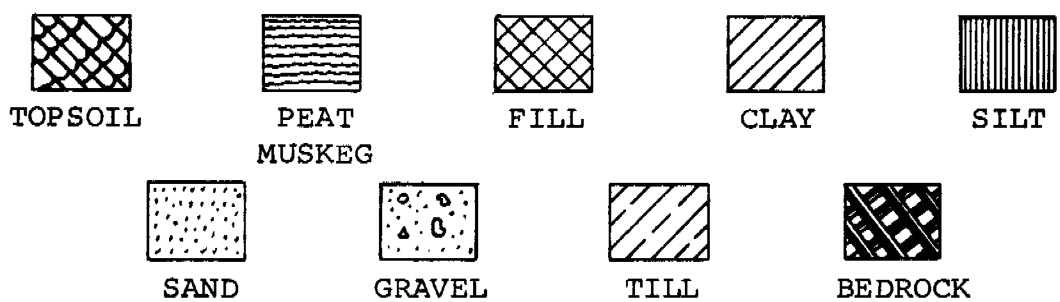
NATURAL MOISTURE CONDITIONS & ATTERBERG LIMITS

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.

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

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.

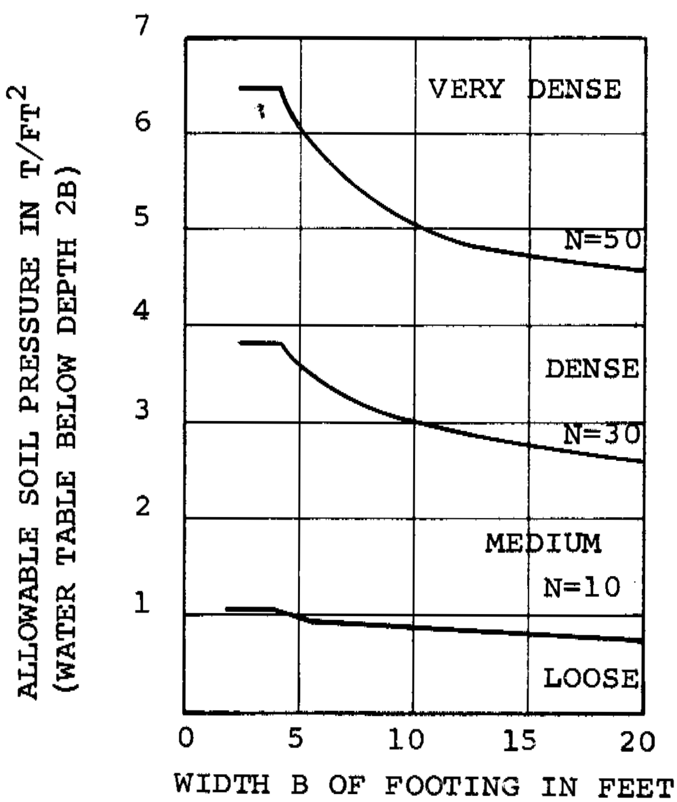
SO_4 - water soluble sulphate content is conducted primarily to determine whether sulphate resistant cement is required for the foundation structure.

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:-



SAMPLE CONDITION AND TYPE

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

UNDISTURBED DISTURBED LOST SAMPLE

SAMPLE TYPES

U	-	3" O.D. Shelby tube sample
D.S.	-	drive sample
M	-	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%

Negligible	Positive	Considerable	Severe
RELATIVE DEGREE OF SULPHATE ATTACK			

Negligible - Normal Portland Cement may be used.

Positive - 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.

Considerable - Type V cement must be used and the concrete compressive strength should be increased to 500 psi higher than the compressive strength which would normally be used.

Severe - 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.

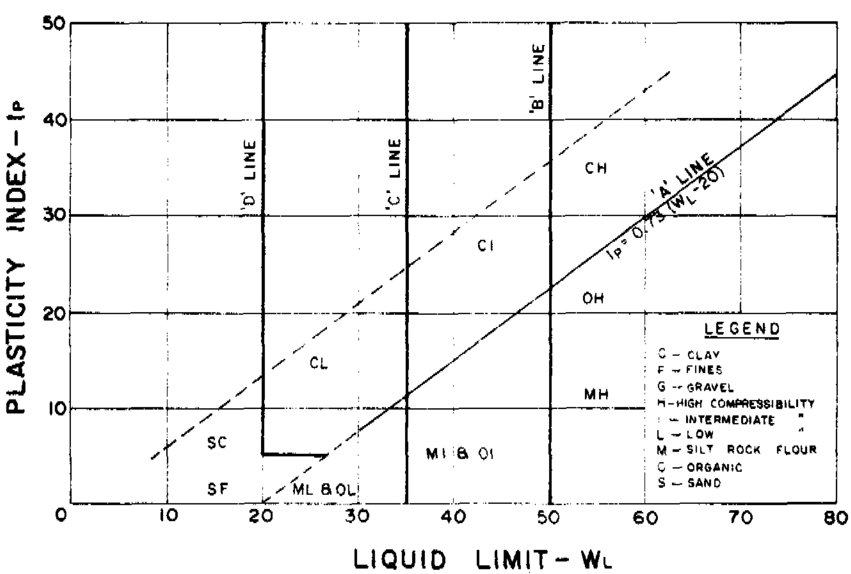
GROUND WATER TABLE

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.

MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISION			GROUP SYMBOL	GRAPH SYMBOL	COLOR CODE	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE-GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 200 SIEVE)	GRAVELS MORE THAN HALF COARSE GRAINS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	GW		RED	WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > 6$ $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3	
			GP		RED	POORLY GRADED GRAVELS, AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY GRAVELS (WITH SOME FINES)	GM		YELLOW	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE P.I. LESS THAN 4
			GC		YELLOW	CLAYEY GRAVELS, GRAVEL-SAND-(SILT) CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE P.I. MORE THAN 7
	SANDS MORE THAN HALF FINE GRAINS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)	SW		RED	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > 4$ $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1$ to 3	
			SP		RED	POORLY GRADED SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY SANDS (WITH SOME FINES)	SM		YELLOW	SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE P.I. LESS THAN 4
			SC		YELLOW	CLAYEY SANDS, SAND-(SILT) CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE P.I. MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT PASSES 200 SIEVE)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML		GREEN	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (see below)	
		$W_L > 50\%$	MH		BLUE	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS		
	CLAYS ABOVE "A" LINE ON PLASTICITY CHART NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL		GREEN	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS		
		$30\% < W_L < 50\%$	CI		GREEN-BLUE	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50\%$	CH		BLUE	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS BELOW "A" LINE ON CHART	$W_L < 50\%$	OL		GREEN	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINE CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
		$W_L > 50\%$	OH		BLUE	ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS			Pt		ORANGE	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE	

PLASTICITY CHART



FOR LABORATORY CLASSIFICATION OF FINE-GRAINED SOILS

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

N_f - poorly bonded or friable frozen soil

N_{bn} - well bonded soil, no excess ice

N_{be} - well bonded soil, excess ice

B. Visible - V (less than 1" thick)

V_x - individual ice crystals or inclusions

V_c - ice coatings on particles

V_r - random or irregularly oriented ice formations

V_s - stratified or oriented ice formations

C. Visible Ice - (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.

SAMPLE CONDITION AND TYPE

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			GP		RED	POORLY GRADED GRAVELS, AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY GRAVELS (WITH SOME FINES)	GM		YELLOW	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE P.I. LESS THAN 4
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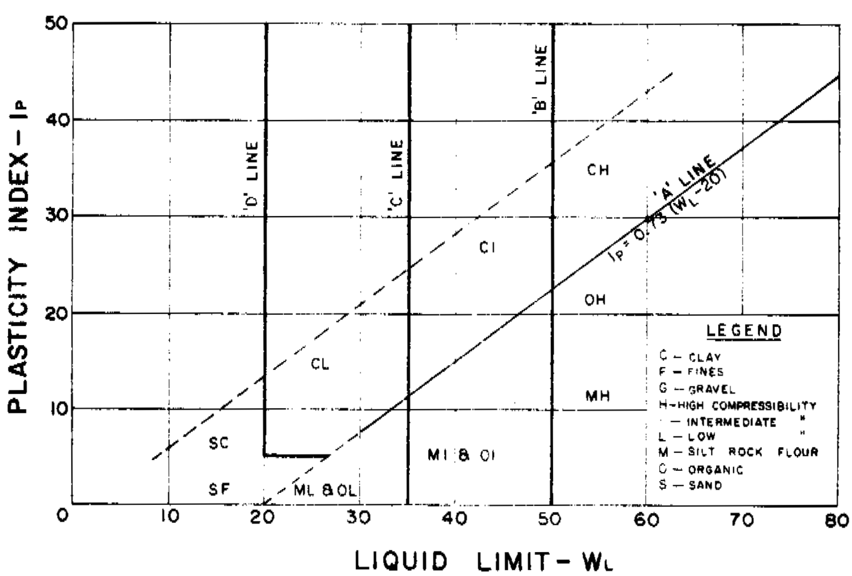
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT PASSES 200 SIEVE)	SILTS BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT	$W_L < 50\%$	ML		GREEN	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (see below)	
		$W_L > 50\%$	MH		BLUE	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS		
	CLAYS ABOVE "A" LINE ON PLASTICITY CHART NEGLIGIBLE ORGANIC CONTENT	$W_L < 30\%$	CL		GREEN	INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS		
		$30\% < W_L < 50\%$	CI		GREEN-BLUE	INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50\%$	CH		BLUE	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS BELOW "A" LINE ON CHART	$W_L < 50\%$	OL		GREEN	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINE CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
		$W_L > 50\%$	OH		BLUE	ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS			Pt		ORANGE	PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE	

PLASTICITY CHART



FOR LABORATORY CLASSIFICATION OF FINE-GRAINED SOILS

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

N_f - poorly bonded or friable frozen soil

N_{bn} - well bonded soil, no excess ice

N_{be} - well bonded soil, excess ice

B. Visible - V (less than 1" thick)

V_x - individual ice crystals or inclusions

V_c - ice coatings on particles

V_r - random or irregularly oriented ice formations

V_s - stratified or oriented ice formations

C. Visible Ice - (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.

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

DWN. A E N | FIELD ENG. DM | DATE DRILLED 12/1/73 | AIRPHOTO NO: | CHAINAGE: 881+70 | OFFSET:
CKD DGP | TECH | RIG: AIR | SURFACE DRAINAGE: GOOD | VEGETATION: POPLAR & CONIFER TO 45' | ELEV: 293.85

TEST HOLE

MILE	B,C,S	NUMBER
459	S	132 A

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

DEPTH (FEET)	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	REMARKS
0	Pt	PEAT FIBROUS MUSKEG	F		
	SP	SAND FINE		Nbn	
8–16	GP	GRAVEL SOME BOULDERS			
16–32	SP	SAND LITTLE CLAY			
32	GW	GRAVEL (water table) COARSE SAND CLAY MIXED WITH SAND LENSES	UF	FREE WATER	
	GP	FINE, SAND LENSES SANDSTONE LAYER @ 62'	F	Nbn	
72–86	CH	CLAY BLUISH GREY, MED PLASTIC, SANDSTONE OR SHALE LAYERS		Nbn	ROCKS IN SAMPLE ARE FROM WALLS, AS CLAY COMING UP.
86–90		SHALE VERY HARD, SANDSTONE			
		END OF HOLE			

Depth scale: 8, 16, 24, 32, 40, 48, 56, 64, 72, 80, 88, 96

Sample type marks at approximately 34, 40, 46, 50, 56, 62, 64, 72, 74, 80 ft.

Plot columns: PLASTIC LIMIT, LIQUID LIMIT; 20, 40, 60, 80, 100, 100+ (no data plotted). GRAIN-SIZE ANALYSIS: CLAY %, SILT %, SAND %, GRAVEL %; WET DENSITY (P.C.F.); DRY DENSITY (P.C.F.); SAMPLE NUMBER; SAMPLE TYPE; % RECOVERY; PENETRATION RESISTANCE — blank.

M.H.E.W.G. REVIEW
REPORT #2, NOVEMBER, 1973:

Comment 12. (h).

Page 7 of the Foundation Report contains the statement that "the main stream flow in recent years has been against the north bank....". The consultant did not recommend bank protection measures, yet did not comment on bank stability either.

Two drill logs are provided for test hole 132A (Plates 8 and 8A); there are differences in depth of holes (40 - 90 ft.), permafrost occurrence (upper 20 feet frozen; completely frozen, but free water 30 to 32 feet) and in reported geology (for example: a silty sandy gravel from 20 to 28 feet, versus a somewhat clayey sand from 16 to 30 feet). One wonders which log is the correct one.

Response - Underwood McLellan Associates Limited.

Although the main stream flow has been against the north bank, instability in the banks was not noted. Of course, consideration should be given to fill protection wherever encroachment into the creek is proposed.

The assessor's comments with regard to test hole 132A are well taken. During drilling of the first test hole to 40 feet the lower portion of the test hole was in doubt as a result of caving and was finally lost at the 40 foot depth. An adjacent test hole was drilled at a slightly different elevation but caving was not as severe consequently, we feel that the 90 foot test hole is most representative of the stratigraphic sequence.