MACKENZIE HIGHWAY

preliminary engineering - phase 1B

OCHRE RIVER BRIDGE

mile 454.6

Reid, Crowther & Partners Limited

CONSULTING ENGINEERS
AND PLANNERS

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PLEASE REFER TO FILE No. 9554-3

December 13, 1973

Department of Public Works
10th Floor, One Thornton Court
P.O. Box 488
Edmonton, Alberta
T5J 2K1

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

Gentlemen:

Mackenzie Highway - Preliminary Engineering Phase 1B
Ochre River Bridge Mile 454.6

We are pleased to present herein our Phase 1B on the Ochre River 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.

The M.H.E.W.G. Assessment Report #2 was received after the Report drawings and text were completed but fortunately the major item i.e. the elimination of guide banks and river training and the lengthening of the structure, has been incorporated into the Phase 1B proposal. The response of the hydrological and geotechnical consultants to specific questions are included in Appendix D.

Department of Public Works
Edmonton, Alberta

December 13, 1973

We are in agreement with Bolter, Parish & Trimble's comment on clearance above Mackenzie high water level in that the structure can be raised 2-feet without changing the concept of the structure. However, if the decision is made to raise the bridge by 2-feet then the profile should be raised by 9-feet± and not 7-feet± as indicated in the text.

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

OCHRE RIVER BRIDGE MILE 454.6

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MACKENZIE HIGHWAY
PRELIMINARY ENGINEERING PHASE 1B
OCHRE RIVER BRIDGE MILE 454.6

INTRODUCTION:

The proposed crossing of the Ochre River is in a stable reach of the river immediately downstream from a partially vegetated gravel bar island and approximately 1,000-feet from the Mackenzie River.

At this location, the channel is approximately 250-feet wide at normal water level; the water is clear and fast-flowing; the stream bed consists of gravel and boulders; and, the banks are approximately 25-feet high and well defined. The forest cover, consisting of black and white spruce and aspen up to 60-feet in height, extends to within a few feet of low water except in the immediate vicinity of the proposed crossing where the trees have been cleared for the telegraph line and the existing road.

The drainage area of the Ochre River is approximately 450 square miles and the estimated design discharge is 18,600 c.f.s. at an average velocity of 8-feet per second; flow at all stages will be confined within the existing banks. No general bed scour is anticipated.

The highway grade at the proposed crossing will be controlled not by flow in the Ochre River, but by backwater due to ice jams on the Mackenzie River. Backwater has been estimated on the basis of recorded extremes on the Mackenzie River at Wrigley, approximately 27 miles south of the Ochre River.

Fish access to the Mackenzie River is good and it appears that the fishery potential of the Ochre River is high. The following species have been noted - grayling, trout perch, round white fish, long-nose sucker, white sucker, slimy sculpin and lake chub.

Several archaeological and historic sites have been identified in the vicinity of the proposed crossing; these are shown on the site plan - Dwg. P501. These sites should be clearly identified and fenced off prior to start of construction.

The previous submission dated December, 1972, based on the proposal by the hydrology consultant, provided a bridge of minimum length and cost. However, this proposal required extensive river training, including a guide bank extending 250-feet upstream from the bridge to the existing gravel bar island and, the partial excavation of this island. This proposal is considered to be unacceptable both from the viewpoint of fisheries and future maintenance of the guide banks and therefore an alternate solution with a considerably longer bridge is now proposed. The proposed bridge shown on Dwg. P501 is now 114-feet longer than the previous proposal but the river training has been deleted and the average velocity at flood stage has been reduced from 11-feet per second to 8-feet per second.

To further reduce the impact of the proposed crossing and to permit more flexible scheduling of construction, the bridge piers have been located so that they are virtually out of the river at normal flow. Comments by F. F. Slaney & Company Limited on the environmental impact of the proposed bridge and the temporary crossing are included in Appendix B.

A minimum clearance of 3-feet from the underside of the bridge superstructure to the estimated high water due to ice jams on the Mackenzie has been used to establish the highway grade. It should be noted that the proposed structure, having a considerably longer center span, will be deeper than the previous submission; as a result, the highway profile has to be raised to a minimum elevation of 326 (top of deck) i.e. 7-feet± higher than the original profile shown on the Interim Report.

The geotechnical investigation, although carried out under adverse conditions, is quite adequate for the design of the bridge piers and abutments. The results of the investigation can be summarized as follows. Permafrost was encountered from 20-feet below stream bed to 80-feet (the depth of the deepest hole). The stratigraphic sequence below the river consists primarily of coarse/fine to coarse gravel with clay layers in the upper portions. Below the granular deposits, a hard dark blue "shale-like" clay was encountered at a depth of 70-feet± which, it is believed, is bedrock. Coring of this strata was unsuccessful due to caving and loss of a core barrel. Bedrock in the area is believed to consist of cretaceous sedimentary deposits of interbedded shale, silt stones and sandstones. 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 72'-260'-72'; the deck is cast-in-place concrete supported by two welded plate girders with composite action in areas of positive moment. Cast-in-place piers and abutments with piled foundations are recommended for the substructure. The proposed layout is shown on Dwg. P501.

It is fair to say that the Ochre River Bridge is now a significant structure and, while we acknowledge that cost is the primary consideration, the appearance of the bridge should not be ignored completely. On this basis, we have proposed that girder soffits be curved rather than straight and that the bridge piers be slightly flared towards the base to create an impression of strength and stability and to break up the large plane surfaces of these relatively massive piers. We have used a shape similar to this on other structures and we consider it to be visually superior to a straight shaft with round nosings, particularly when formwork panels and tieholes are arranged symmetrically and the exposed concrete surfaces sandblasted.

"Spill through" abutments have been shown (Dwg. P502) on the assumption that the 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. The unconventional bearing arrangement at the abutments is required by the unbalanced superstructure which produces uplift forces under all loading conditions.

Subsurface conditions are such that piled foundations are recommended both for piers and abutments - (a) to protect piers against local scour and (b) to reduce differential settlement between piers and abutments to a negligible amount - the proposed structure with its short, stiff end spans will be particularly sensitive to differential movements at supports.

For the purposes 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 they will be given further consideration 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, 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.

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)
30 Ton Tension)(Group VIII C.S.A. S6)

Alternate Systems.

Before selecting the three span structure, four deck systems and a four span 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 systems 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. P504.

Under normal conditions one would anticipate that a four span structure would be more economical than a three span structure however, in this instance, it was found that the additional steel required for the longer span was largely offset by the cost of the additional river pier. This pier proved to be relatively expensive because of its size, the high cost of concrete and piling, and the environmental constraints which recommend that construction in the river be carried out during winter. Scheduling of the three span structure will be more flexible and estimating the cost of the structure will be more reliable since the greatest unknown is the risk factor which contractors will apply to concrete and piling unit prices.

TEMPORARY CROSSING:

The proposed location of the temporary crossing is approximately 250-feet downstream from the permanent bridge at the existing winter road crossing.

The proposed structure, a two span Bailey bridge supported by rock filled timber cribs, is shown on Dwg. P503. 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 the Ochre River. 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 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 inundation of the bridge could occur every spring. However, he further notes that the bridge will be above the ice run on the Ochre 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.

Consideration was given to building a native timber structure, however the local timbers are such that the clear span would at best be 15-feet. An arrangement of short spans and timber cribs would be quite impractical on this river both from the environmental and hydrological viewpoints. Native timber will be suitable for deck and crib construction.

SCHEDULING:

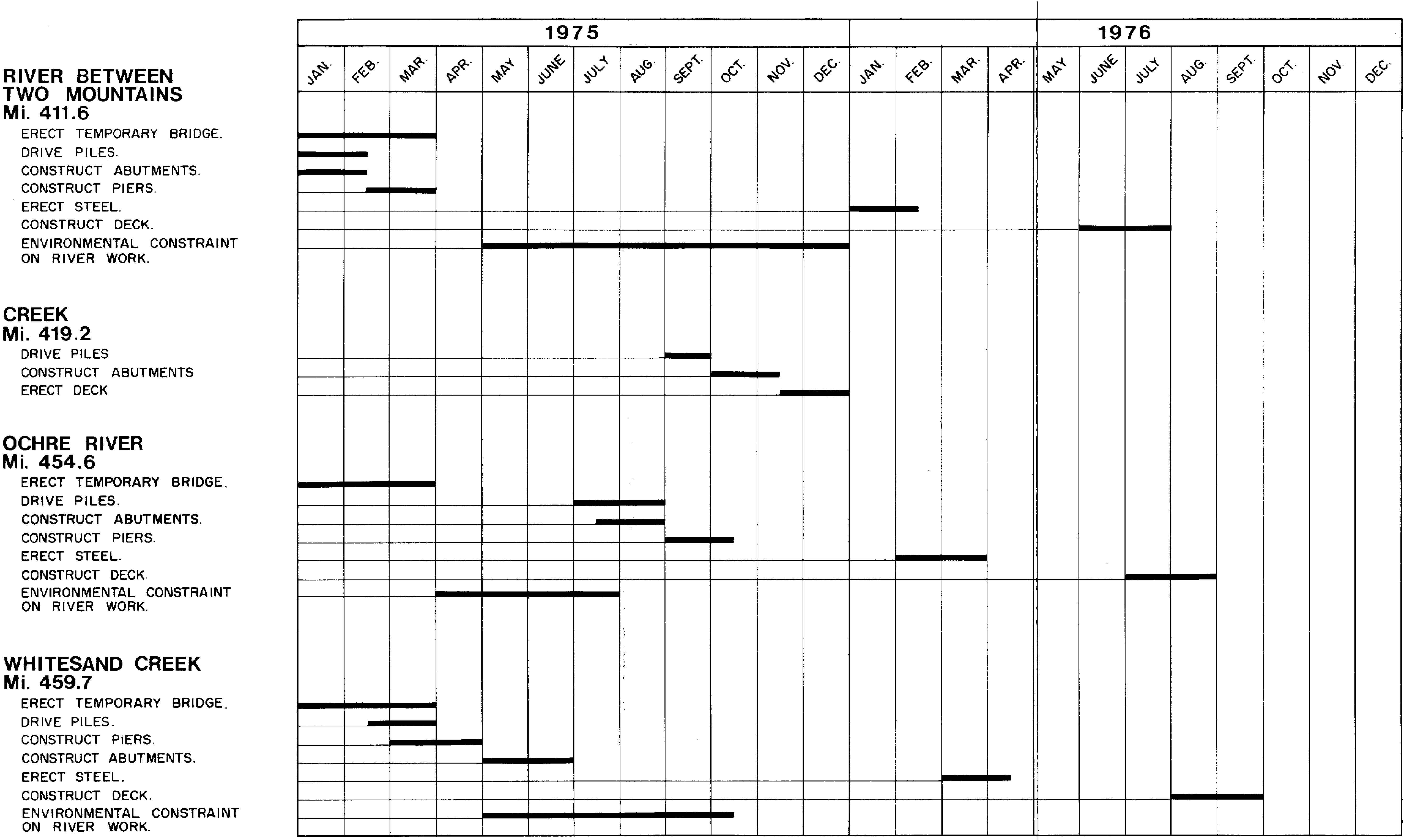
The following schedule assumes that highway construction will commence in the spring of 1975 and that all bridges on this section of the highway will be completed in the fall of 1976.

Allowing six months for structural steel delivery the tentative schedule will require bridge drawings and contract documents to be completed not later than April, 1975 and possibly sooner depending on the period of time required for approvals.

Schedule.

- Deliver concrete aggregates to Ochre River mouth - Fall 1974 or Summer 1975.
- Construct temporary bridge - Winter 1974-75.
- Start abutment construction - July, 1975.
- Start pier construction - August, 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.



NOTE: NAVIGATION SEASON ON MACKENZIE RIVER
JUNE 15th. TO OCTOBER 15th.

MACKENZIE HIGHWAY
CONSTRUCTION SCHEDULE
FOR BRIDGES BETWEEN
Mi. 411 & Mi. 460

COST ESTIMATE:

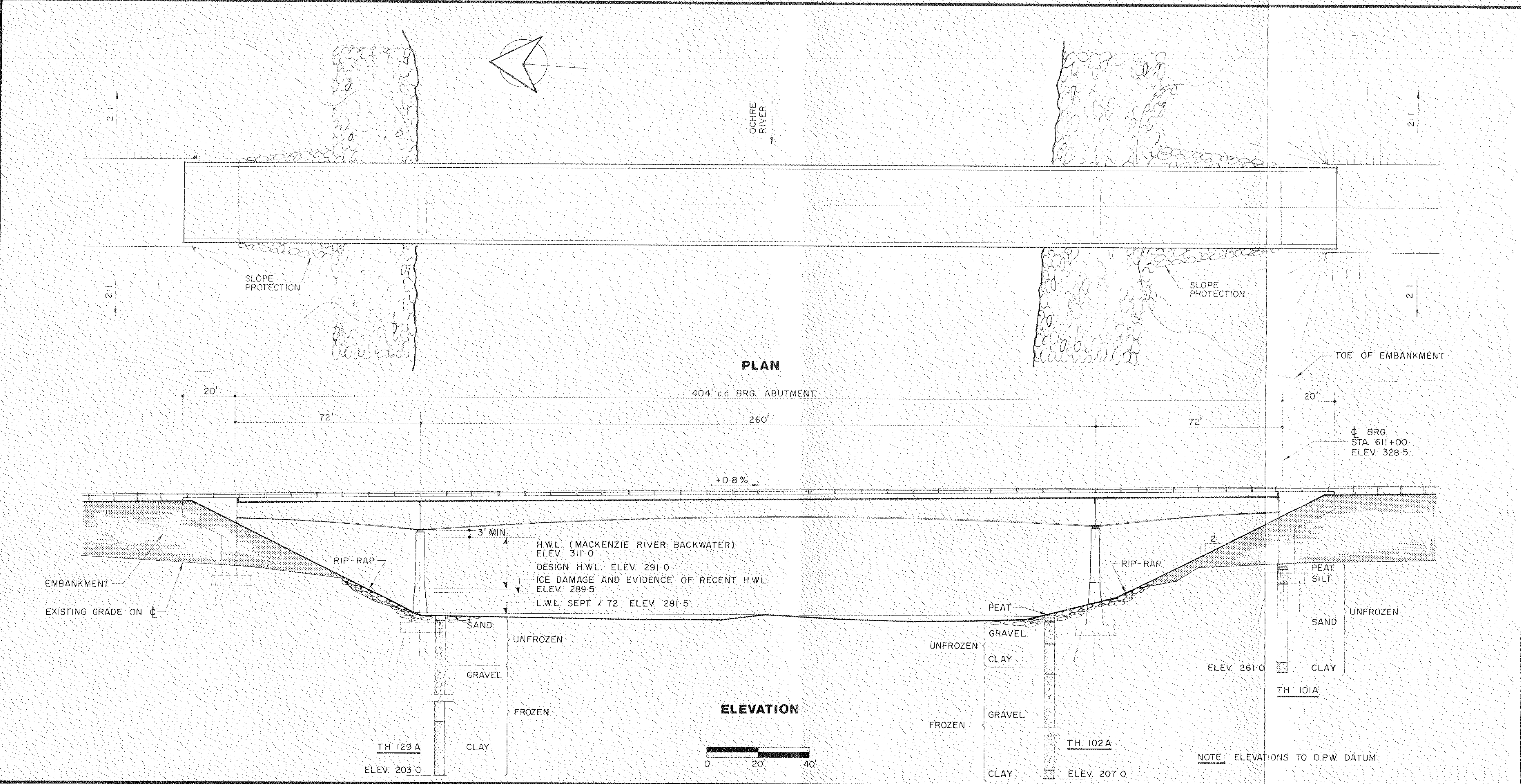
Estimating costs at this stage of the project has proved to be even more difficult than it was one year ago, when the Interim Report was prepared, because of the unstable prices which are currently being experienced throughout the construction industry.

We have reviewed the unit prices which were developed for the Interim Report and, while some of the units now appear to have been on the low side, they were generally speaking quite realistic at that time. These unit prices have therefore been used as a basis for the present estimate but increased to reflect current prices (December, 1973). A detailed breakdown of the estimate is given on the following page.

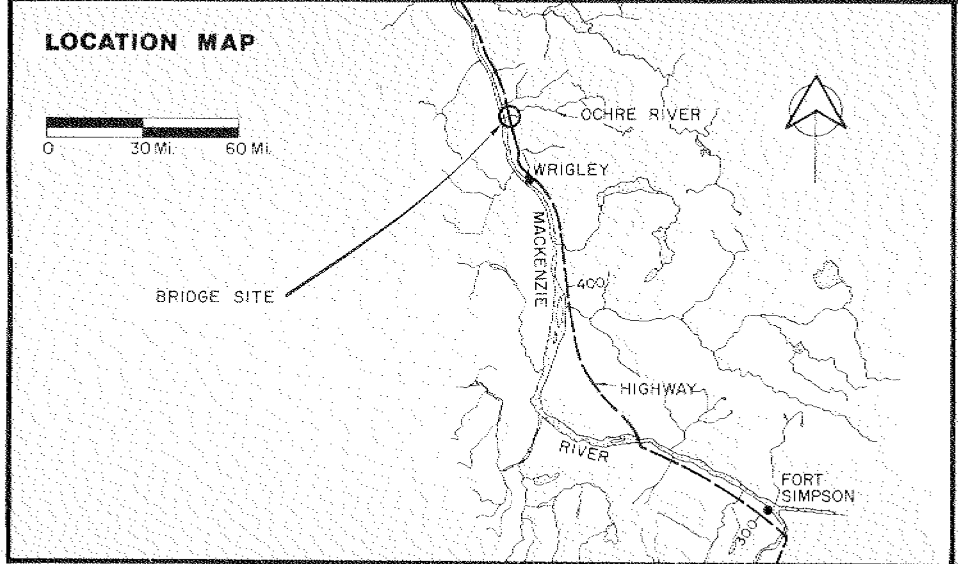
OCHRE RIVER BRIDGE MILE 454.6

COST ESTIMATE - DECEMBER, 1973

Item	Quantity	Unit	Unit Price	Amount
Excavation & Backfill				
- Rock	--	--	$ --	$ --
- Gravel	800	cu.yd.	50.00	40,000
- Riprap	1,100	cu.yd.	25.00	27,500
Piles	3,000	lin.ft.	30.00	90,000
Concrete				
- Foundations	300	cu.yd.	300.00	90,000
- Piers & Abutments	420	cu.yd.	300.00	126,000
- Deck	475	cu.yd.	350.00	166,300
- Approach Slab	45	cu.yd.	300.00	13,500
Reinforcing Steel	300,000	lb.	0.40	120,000
Structural Steel	250	ton	1,500.00	375,000
Handrail	890	lin.ft.	30.00	26,700
Expansion Joint	65	lin.ft.	100.00	6,500
Bearings	8	each	1,500.00	12,000
Sub-Total				$1,093,500
15% Contingency				$ 164,000
7% Engineering & Administration				$ 88,500
TOTAL				$1,346,000
Temporary Bridge				$ 200,000

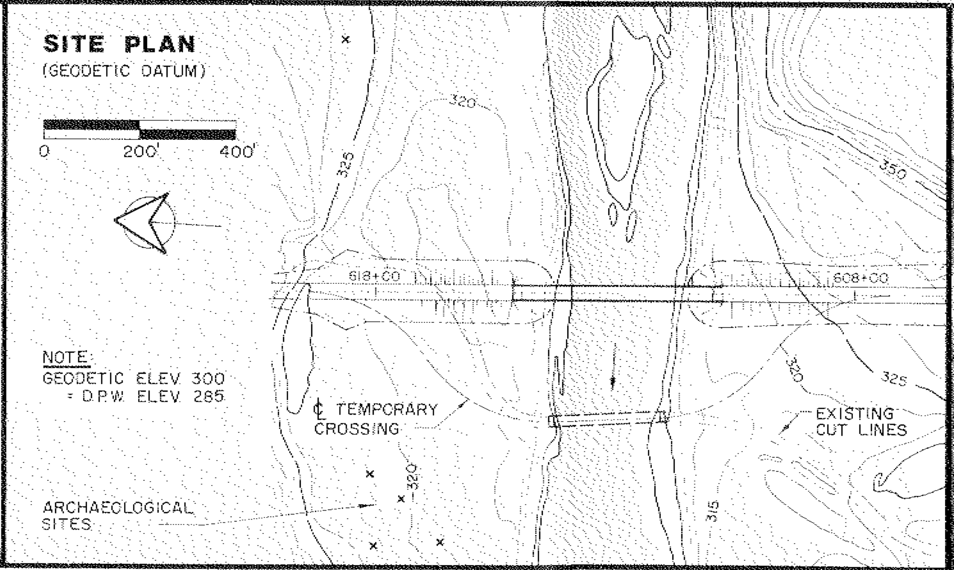


LOCATION MAP



SITE PLAN

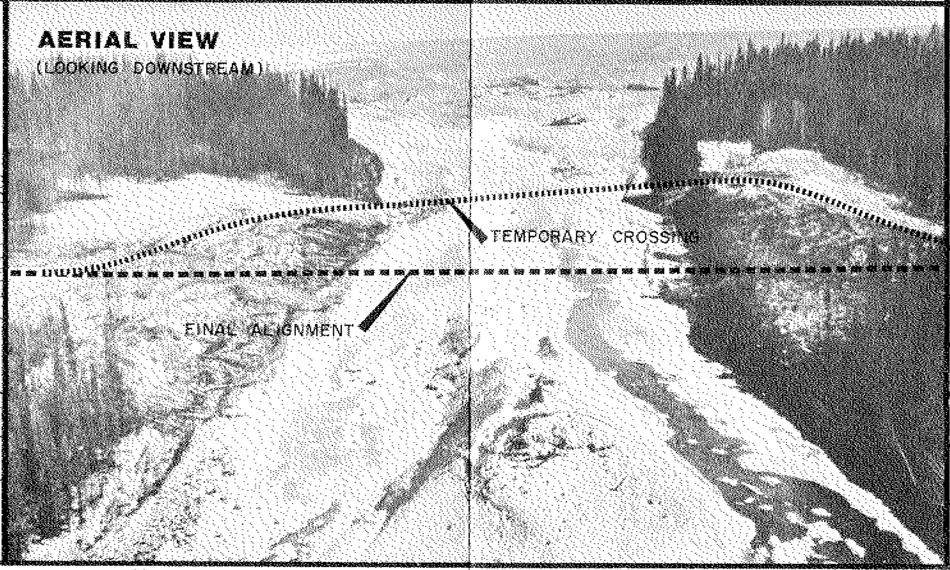
(GEODETIC DATUM)



NOTE:
GEODETIC ELEV. 300
= D.P.W. ELEV. 285

AERIAL VIEW

(LOOKING DOWNSTREAM)



Department of Public Works
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WESTERN REGION
J.A. BROWN
REGIONAL DIRECTOR

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CONSULTING ENGINEERS & PLANNERS
VANCOUVER CALGARY EDMONTON REGINA WINNIPEG TORONTO

A detail no. détail no.
B location drawing no. sur dessin no.
C drawing no. dessin no.

revisions date

project title titre du projet

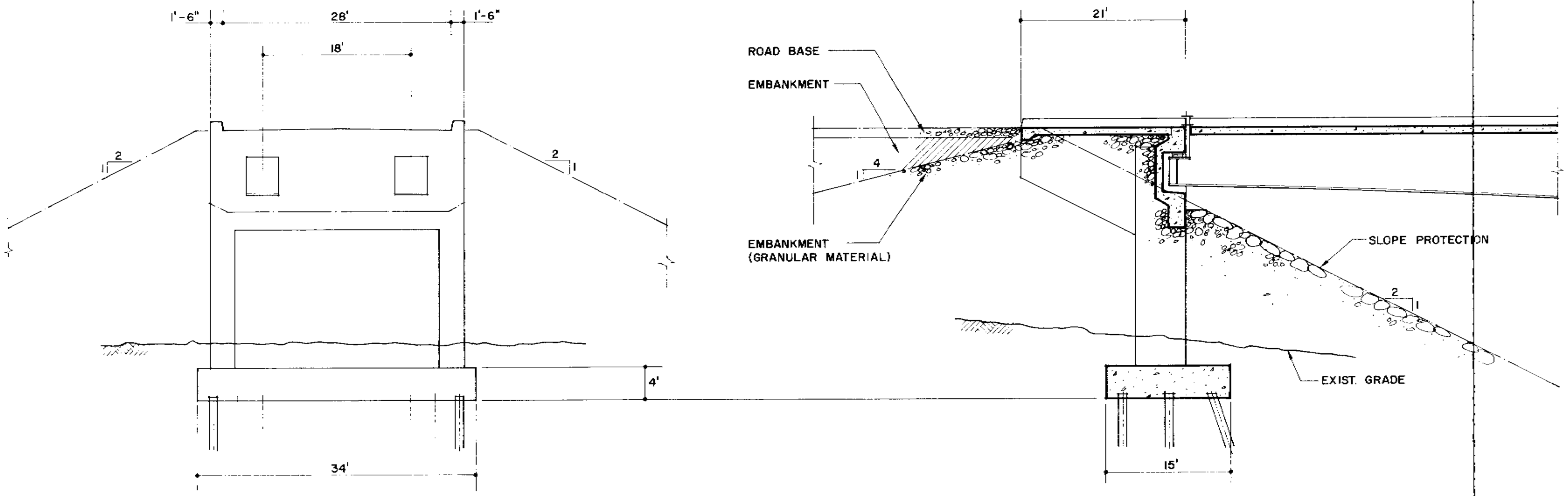
MACKENZIE HIGHWAY

drawing title titre du dessin

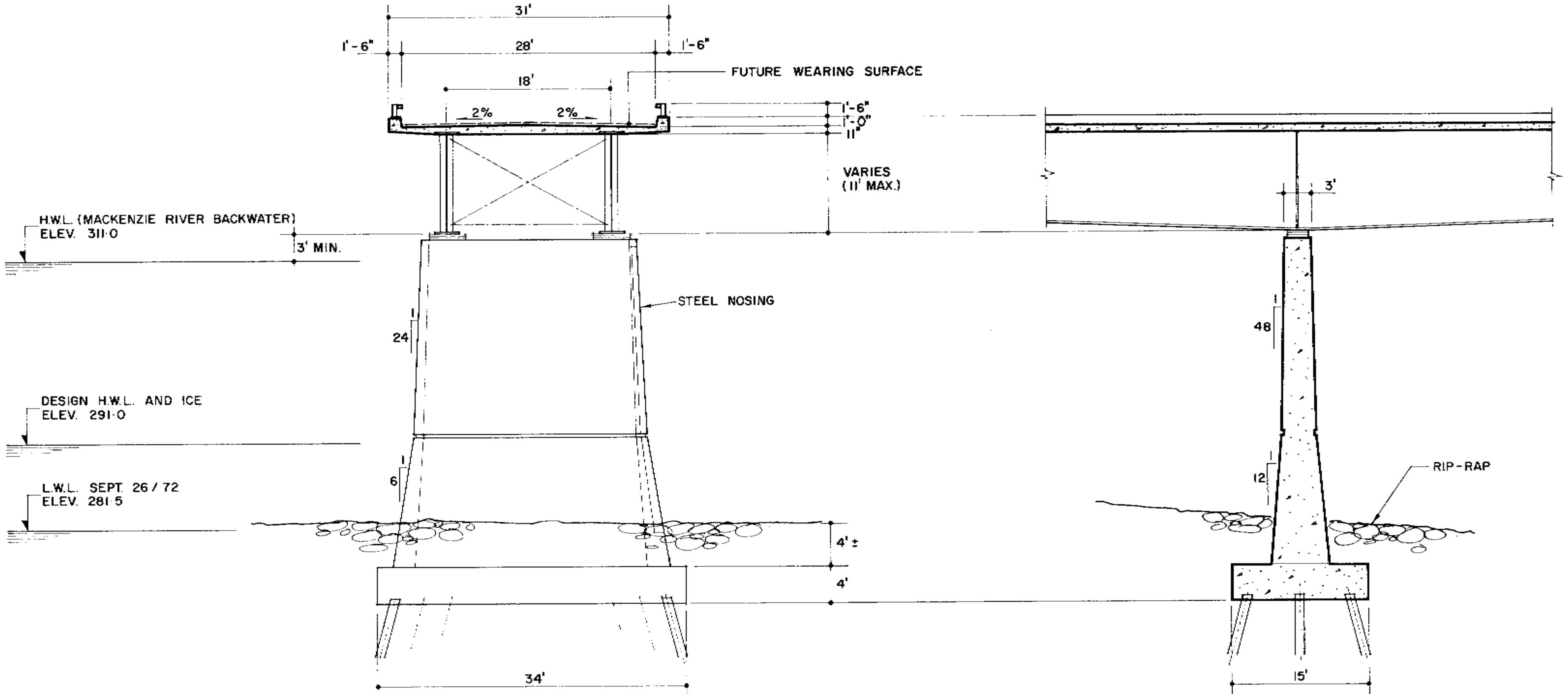
OCHRE RIVER BRIDGE
Mile 454.6

GENERAL LAYOUT

designed by conçu par
date DECEMBER 1973
drawn by dessiné par
date
reviewed by examiné par
date
approved by approuvé par
date
Tender Soumission
D.P.W. Project Manager Administrateur de projets M.T.P.
project number no. du projet
85032
drawing no. dessin no.
9554-P 501



ABUTMENT



PIER

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OCHRE RIVER BRIDGE
Mile 454.6

DETAILS

designed by conçu par
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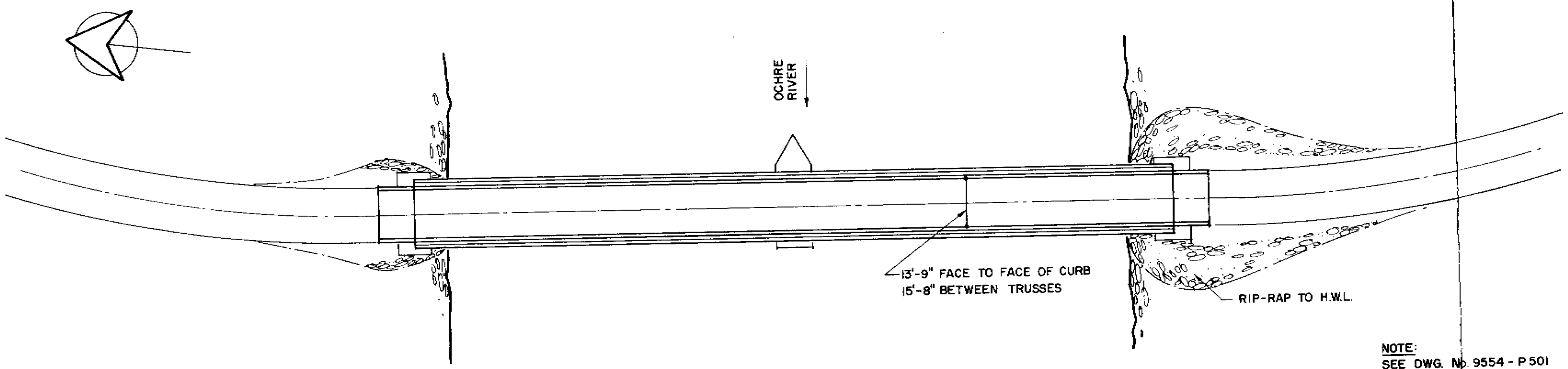
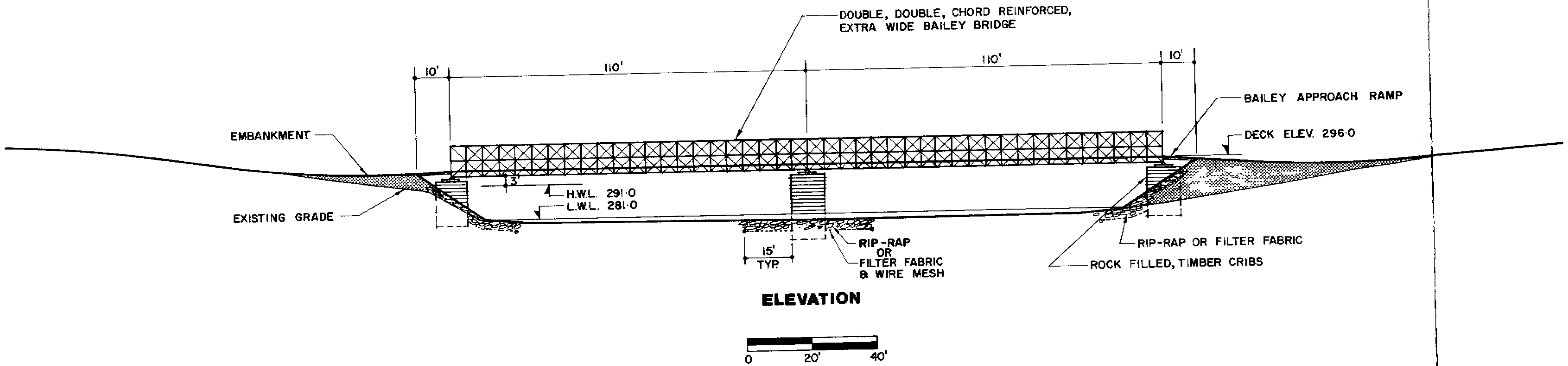


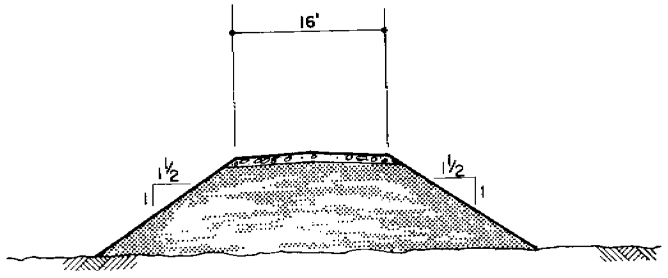
PLAN

NOTE:
SEE DWG. No. 9554 - P501
FOR LOCATION.

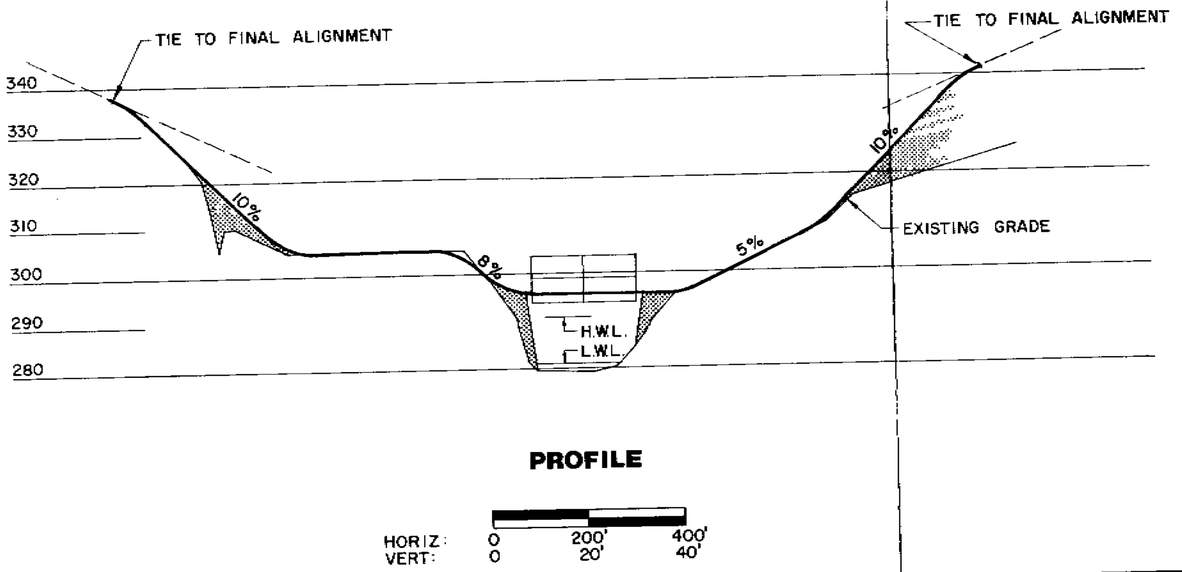
ELEVATIONS ARE TO
D.P.W. DATUM.



ELEVATION



ROADWAY SECTION



PROFILE

Department of Public Works
Ministère des Travaux publics

WESTERN REGION
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OCHRE RIVER BRIDGE - Mile 454.6

TEMPORARY CROSSING

designed by conçu par
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reviewed by examiné par
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Tender Soumission
D.P.W. Project Manager Administrateur de projets M.T.P.
project number no. du projet
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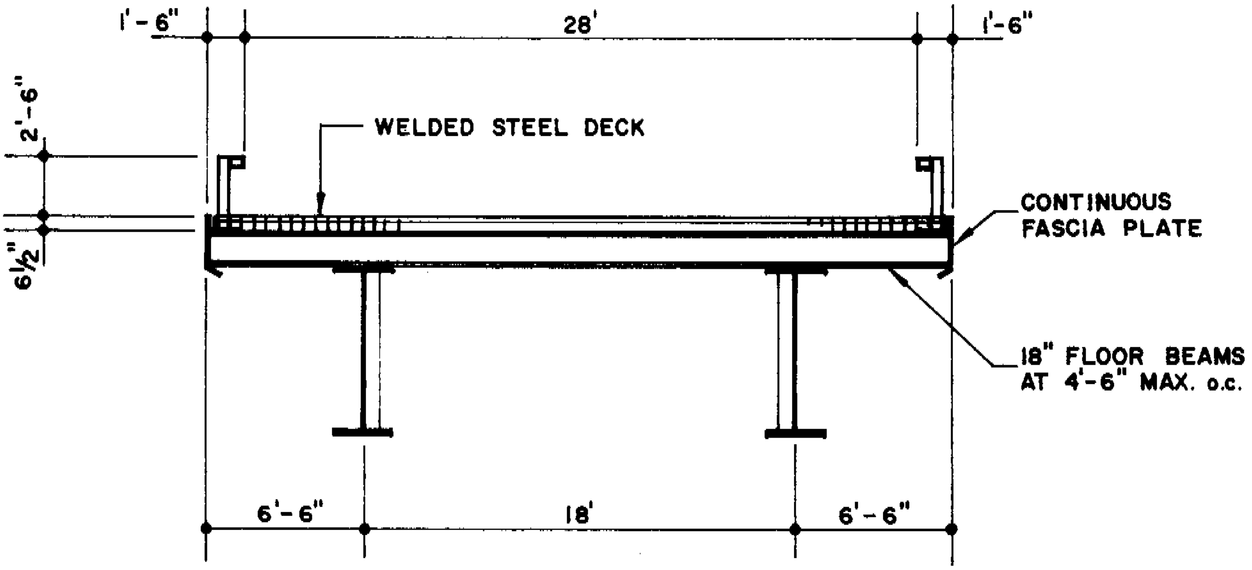
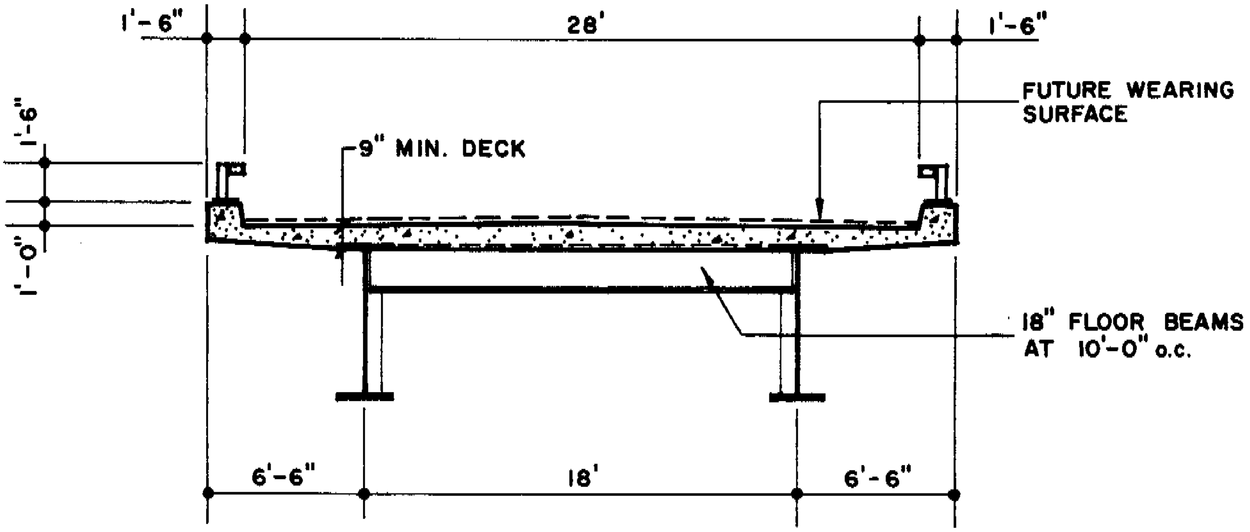
drawing no. dessin no.
9554 - P503



STEEL DECK



CONCRETE DECK

ALTERNATE DECK SYSTEMS

OCHRE RIVER BRIDGE:

Introduction.

Two bridge designs are being considered for the crossing of the Ochre River. The three span design has two short end spans and a center span supported on two riverbank piers. The four span design has an additional pier located mid-river.

Environmental Impact.

Other than for fish, the environmental impacts for both designs are negligible and equal.

The design having the least impact is the three span structure because of its minimal encroachment on the cross section of the river channel.

The four span design has the additional encroachment on the river cross section by the requirement for a mid-channel pier. The mid-channel pier would cause a slight increase in current velocities in high water conditions and is also a potential restraint for ice and floating debris. During construction activities, access and excavation work will cause some disturbance of the stream bed and will result in temporary turbidity and silting downstream.

Recommended procedures are:

- Build the piers when the river is frozen.
- Provide access from the south bank and excavate the pier footing during a low water period. If the piers are poured during the summer, the preferred operating period is August and the first half of September.
- Restore the disturbed channel to its original contour and replace rock armour upon completion.
- Erect the spans when the river is frozen.

Summary.

The three span design has minimal environmental effects and should be given primary consideration.

OCHRE RIVER TEMPORARY CROSSING:

Introduction.

The proposed temporary crossing bridge design for the Ochre River is located approximately 300-feet downstream from the proposed permanent highway crossing. The bridge is a two span design supported by three rock filled timber cribs located at mid-stream and above the low water level on either shore.

Environmental Impact.

Other than for fish, the environmental impact appears negligible.

The piers will cause a slight increase in current velocities in high water conditions. The mid-channel pier is a potential restraint for floating ice and debris. Access and excavation work will cause some disturbance of the stream bed that will result in temporary turbidity and silting downstream. A location above the proposed highway center line utilizing the gravel bar would offer less disturbance in building the center pier but would require longer spans to locate the shore piers as favourably as the proposed design.

Recommended procedures are:

- Build the cribs during low water preferably from mid-February through April. A shear may be necessary for the mid-river pier.
- Erect the spans when the river is frozen.
- Riprap the center pier to minimize scour.
- Restore the disturbed channel to its original contour.
- Investigate other sources of riprap.

The riprap requirements for both the temporary and permanent crossings are apparently greater than can be readily supplied by the Ochre River. The Fisheries Service deliberated for a period before allowing utilization of granular material from the bed of the Martin River for the temporary crossing, and could possibly refuse to permit the use of the quantities of stream bed material required in this and other structures.

Alternate sources of riprap should be investigated in the Mount Gaudet area or to the east. It could be hauled during the winter. Sheeting of the interior of the cribs for utilization of granular material would minimize riprap requirements.

- Archaeological surveillance is required during excavation on the banks because of indicated potential prehistoric activity. A deteriorated cabin near the north end of this crossing should be preserved.

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:-

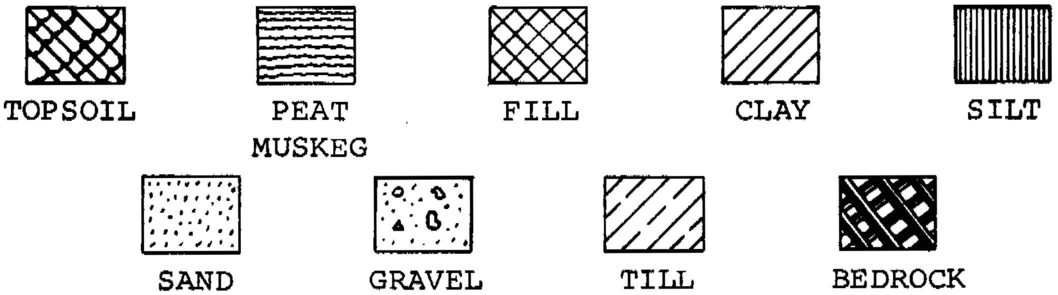
NATURAL MOISTURE CONDITIONS & ATTERBERG LIMITS

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.

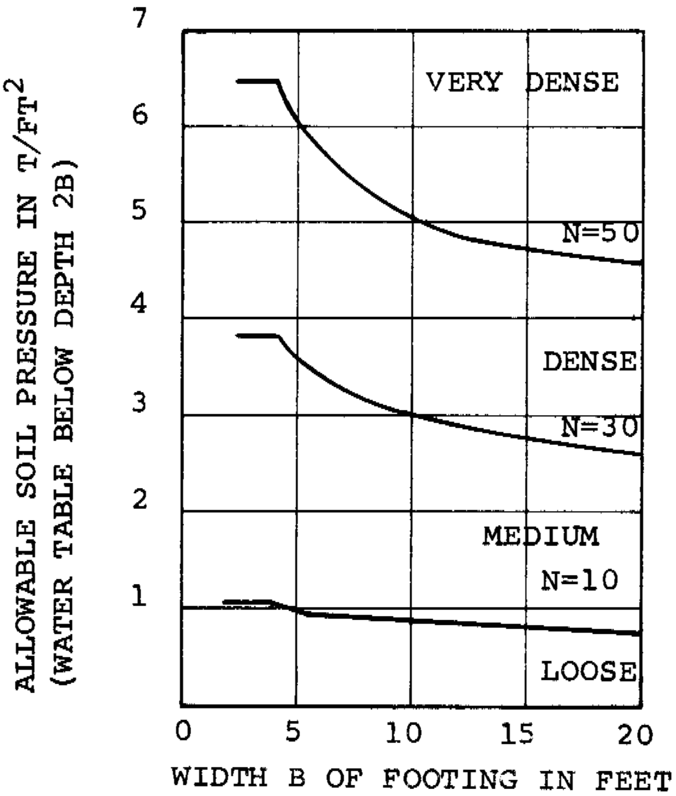
SO_4 - water soluble sulphate content is conducted primarily to determine whether sulphate resistant cement is required for the foundation structure.

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%

<table>
<tr><td>Negligible</td><td>Positive</td><td>Considerable</td><td>Severe</td></tr>
<tr><td colspan="4">RELATIVE DEGREE OF SULPHATE ATTACK</td></tr>
</table>

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.

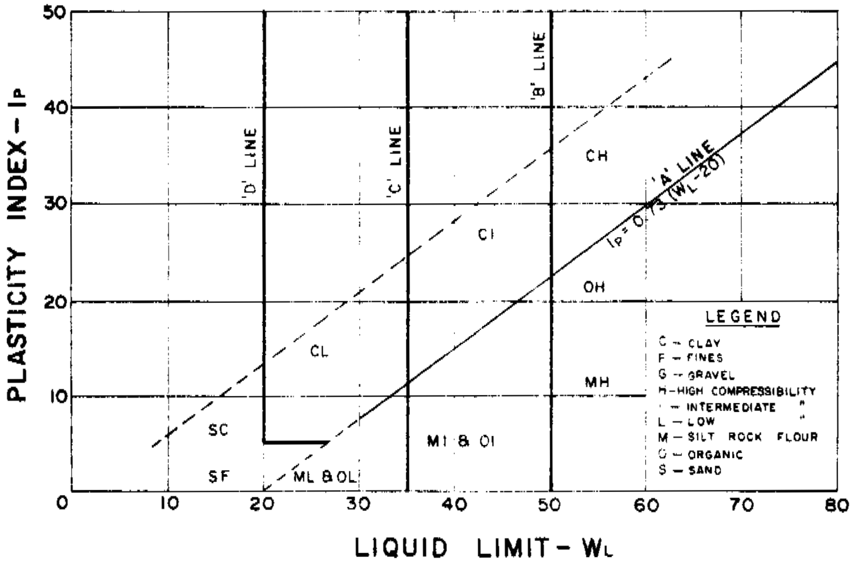
GROUND WATER TABLE

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.

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: A.E.W
FIELD ENG: D.M
DATE DRILLED: 10/1/73
AIRPHOTO NO:
CHAINAGE: 614+25
OFFSET: ℄
CKD: D.G.P
TECH
RIG
SURFACE DRAINAGE: WEST TO EAST
VEGETATION: POPLAR & CONIFER TO 40'
ELEV 283.3

TEST HOLE

MILE	B,C,S	NUMBER
454	S	129 A

○ = WATER CONTENT (% OF DRY WEIGHT)
△ = ICE CONTENT (% OF SAMPLE VOLUME)
PLASTIC LIMIT ├──┤ LIQUID LIMIT
20 40 60 80 100 100+

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	DEPTH (FEET)	GRAIN-SIZE ANALYSIS CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (P.C.F.)	DRY DENSITY (P.C.F.)	REMARKS
						SAND GRAVEL	UF									
8					GW	BOULDERS, FINE			8							
16						SANDY CLAY LENSE COARSE, SOME COBBLES			16							
24							F	Nf	24							
32						GREY BROWN CLAY LENSE			32							
40						COARSE WITH SAND STONE LENSES			40							
48									48							
56								Nbn	56							
64					CH	CLAY BROWN, MED - HIGH PLASTIC, TRACE OF SAND			64							
72						VERY HARD BLUISH GREY SOME SAND HIGH PLASTIC		Nbn	72							
80						END OF HOLE			80							

UNDERWOOD McLELLAN & ASSOCIATES LIMITED

DRILL HOLE REPORT

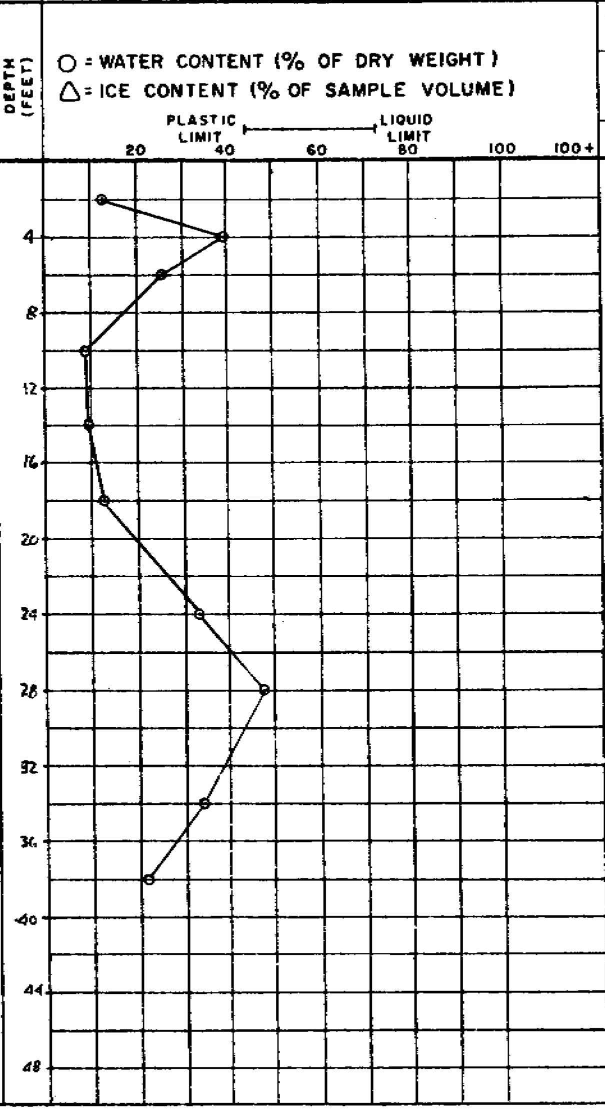
DEPARTMENT OF PUBLIC WORKS, CANADA
MACKENZIE HIGHWAY

DWN: A.E.W | FIELD ENG: RS | DATE DRILLED: 11/12/72 | AIRPHOTO NO: | CHAINAGE: 611+00 | OFFSET: ℄
CKD: DG.P | TECH: DM. | RIG: AIR | SURFACE DRAINAGE: NORTH | VEGETATION: NONE | ELEV: 302.82

TEST HOLE

MILE	B,C,S	NUMBER
454	5	101A

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION	CLAY %	SILT %	SAND %	GRAVEL %	WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS
0–1					Pt	PEAT GRANULAR MUSKEG	UF								
1–7		P			ML To OL	SILT CLAYEY, TRACES OF ORGANIC, SANDY									
7–19					SP	SAND COARSE, SOME SILT SOME GRAVEL WATER TABLE @ 15'				8	10	82			
19–38					SM	FINE - MEDIUM, SILTY TRACES OF GRAVEL VERY WET									
38–42						CLAY HARD, SHALE LIKE									
42						END OF HOLE									



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

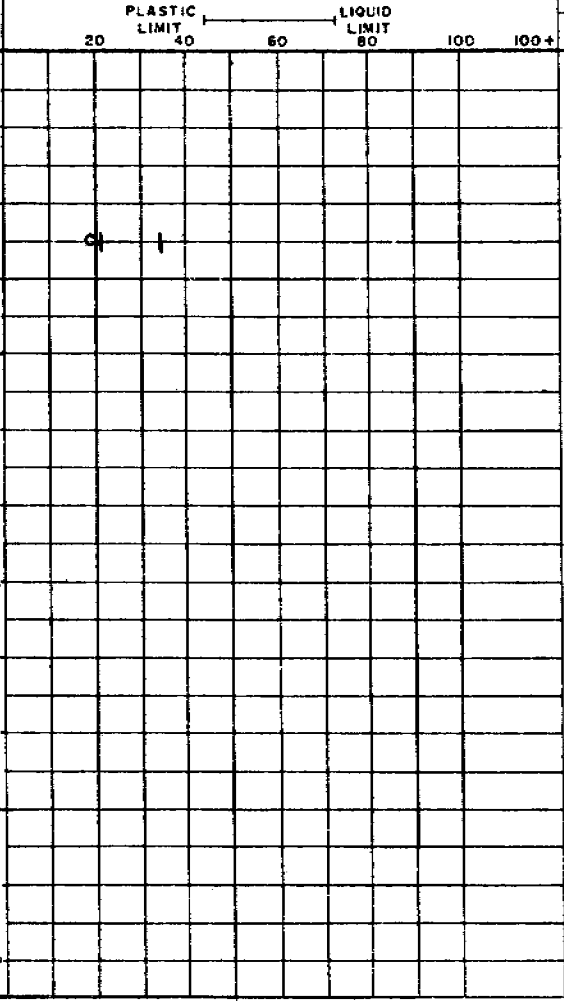
DWN. A E W.	FIELD ENG. T. S	DATE DRILLED 11/12/72	AIRPHOTO NO:	CHAINAGE: 611 + 90	OFFSET: ℄	TEST HOLE
CKD D. G. P	TECH D M	RIG AIR	SURFACE DRAINAGE:	VEGETATION:	ELEV 282.9	

MILE	B,C,S	NUMBER
454	S	102 A

DEPTH (FEET)	SAMPLE NUMBER	SAMPLE TYPE	% RECOVERY	PENETRATION RESISTANCE	UNIFIED SOIL SYMBOL	SOIL DESCRIPTION	LIMITS OF FROZEN GROUND	ICE DESCRIPTION
					Pt	PEAT BLACK TOPSOIL	UF	
					GP	GRAVEL COARSE WATER TABLE @ 4'		
8								
						CLAY		
16					CL	GREYISH BROWN		
							F	
24						GRAVEL		Nbn
32					GP			Nf
40						MAINLY COARSE WITH THIN SEAMS OF FINE GRAVEL		
48								
56								
64					GW			
72					CL	CLAY - SANDY, DARK BLUE		Nf
80						END OF HOLE		
88								
96								

○ = WATER CONTENT (% OF DRY WEIGHT)

△ = ICE CONTENT (% OF SAMPLE VOLUME)



GRAIN-SIZE ANALYSIS				WET DENSITY (PCF)	DRY DENSITY (PCF)	REMARKS
CLAY	SILT	SAND	GRAVEL			
%	%	%	%			

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

Comment 11. (g).

It is however very subject to Mackenzie backwater and the consultants estimated elevation of 311 feet (31 feet above bed level) seems unusually low when one considers that the Ochre could peak, during a jam. Certain vegetation patterns suggest that the Ochre River has topped its floodplain in recent times.

Note the discrepancy between Underwood et al and Reid et al consultants in their elevations. Further note that Mackay reports (1973) ice push on the Mackenzie from 40 to 45 feet resulting from the effects of the Ochre delta and a shoal in the Mackenzie across the river. If coarse well drained material, adequately riprapped is used, encroachment on this high flood plain might be acceptable.

Response - Bolter Parish Trimble Ltd.

The estimated elevation of 311 (field survey datum) was based on backup from the Mackenzie River and not on the bed level of the Ochre. The water elevation of the Mackenzie on September 4, 1972, 2250 feet downstream of the crossing was 268.4 (field survey datum). We had allowed (our January report) 42.6 feet from this water level to estimated high water due to ice jam on the Mackenzie. Afflux caused by an M1 backwater curve has been calculated to be negligible (less than 0.1') at the bridge site even during the design flood on the Ochre (18,600 c.f.s.). The mean velocity at the crossing assuming both the extremes occurred at the same time would be in the order of 1.7 f.p.s. This results in a very low gradient of the water surface profile from the mouth and would present a lake like appearance.

We have a report by D. K. MacKay and J. R. Mackay which notes the ice push at the delta of the Ochre as 12.5 m (41 feet) above the June 30, 1970 level of the Mackenzie. We had estimated (our January report) a 42.6 rise due to ice jams on the Mackenzie above the September 14, 1972 level. We estimate the June 30 level to be 3.5 feet higher than the September 14 level, thus using the D. K. MacKay and J. R. Mackay report we would estimate the highwater due to ice jams at the Mackenzie to be (44.5 + 268.4 + 0.1) elevation 313.0 (field survey datum). At this elevation the freeboard would be 1.0' below the bottom flange. We do not believe this to be serious because of the 'lake' created and the low velocities expected. The north end of the bridge could however be easily raised 2.0' in the final design to provide the minimum freeboard originally proposed.

Response - Reid, Crowther & Partners Limited.

Soils consultants were working to field datum - Reid, Crowther and Partners used approximate geodetic datum for Interim Report.

Comment 11. (h).

In test-hole 101A (611 + 00) a water content of almost 50 per cent was obtained, at a depth of 28 feet. The assessors wonder whether the soil skeleton consisted of water, with the mineral particles in suspension, or whether the field classification SM should be reviewed.

Test holes 94A, 95A, 96A and 101A, though drilled on December 9, 1972, were apparently entirely free of frost. The assessors wonder whether there would be reasons, other than the drilling method used, for this unseasonal absence of seasonal frost, and to what extent reliance can be placed on the determination of permafrost distribution at this site, as reported by the consultant.

According to the log, test hole 99A is frozen throughout, but the text of the foundation report on page 11 considers it to be free of frost.

Response - Underwood, McLellan & Associates Limited.

After drilling 3 relatively deep test holes at extreme cost, it was decided that sufficient data had been collected. The stratigraphic sequence consisted of deep granular deposits over a "shale-like" clay which was believed to be weathered bedrock. Numerous attempts were made to core the bedrock but was unsuccessful as a result of caving gravel. Placement of casing in deep gravel deposits is most often impossible with the drilling equipment which was available for this project. A "Becker type" drill or equal, if available, would be recommended for future bridge investigations.

Re: Assessors comment with regard to soil sample moisture content in test hole 101A at 28 feet. Samples reported were disturbed wash samples recovered in water consequently the high moistures although the particular 28 foot sample may have some clay. Report indicates on Page 10 - "The moisture content of the sand below elevations 305 was high as a result of submergence".

Assessors comments relative to seasonal frost at test holes 94A, 95A, 96A and 101A are irrelevant. Seasonal frost was not recorded primarily because the depth was negligible. The recorded absence of seasonal frost at this site in no way reflects on the evaluation of the permafrost at this site.

Foundation report incorrectly stated on Page 11 that test hole 99A was unfrozen, although more detailed reference was made on Page 25 to proposed cuts in frozen soils represented by test hole 99A. Presence of permafrost as indicated in test hole log should be considered correct.