CONSULTING ENGINEERS
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January 11, 1974

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
River Between Two Mountains Bridge Mile 411.6

We are pleased to present herein our Phase 1B on the River Between Two Mountains 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

RIVER BETWEEN TWO MOUNTAINS BRIDGE MILE 411.6

INDEX

MACKENZIE HIGHWAY

PRELIMINARY ENGINEERING PHASE 1B

RIVER BETWEEN TWO MOUNTAINS BRIDGE MILE 411.6

INTRODUCTION:

The proposed crossing of the River Between Two Mountains is in a stable reach of the river about 4,000-feet upstream from the Mackenzie River.

The channel at this location is approximately 150-feet wide with well established banks (approximately 15-feet high) incised into an old flood plain; the water is clear and fast-flowing; the stream bed is armoured with boulders up to 2-feet in diameter; and, visible driftwood is light. Stands of mixed spruce, pine, larch, aspen and birch 20 to 40-feet high extend to the top of the river-banks, except in the vicinity of the proposed south abutment where the trees have been cleared for the telegraph line and existing road.

The River Between Two Mountains drains a catchment area of approximately 1,300 square miles, including extensive lake storage, and the estimated design discharge is 23,400 c.f.s. at an average velocity of 9-feet per second in the main channel. At this flow, there will be overtopping of the existing banks in the vicinity of the bridge and general bed scour of approximately 5-feet to elevation 260 can be expected.

The highway grade at the crossing is controlled by backwater conditions caused by the formation of ice jams on the Mackenzie River. Backwater has been estimated from recorded extremes at Wrigley, approximately 24 miles north of the River Between Two Mountains.

Fish access to the Mackenzie River is good and fishery potential is high. Fish populations include the following species: lake trout, northern pike, walleye, Arctic grayling and several species of whitefish. In the case of the Arctic grayling it is suspected that this river forms a spawning area and a possible migration route between Fish Lake and the Mackenzie River; consequently, siltation due to construction activity must be kept to a minimum.

Although the archaeological survey has not discovered any sites of historic or prehistoric interest in the vicinity of the highway alignment, a close surveillance should be maintained during construction, especially along the riverbanks.

The previous submission of December, 1972, provided a bridge of minimum length and cost in which the three spans were proportioned to produce a structural balance. Such a layout placed the piers within the active stream channel, a location which is undesirable from the point of view of environmental impact. For this reason, and to permit more flexible scheduling of construction, the bridge piers have been relocated to be 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 since the center span is longer than that shown in the previous submission, the highway profile has to be raised to elevation 302.5 (top of concrete deck) i.e. 3-feet above the original profile shown on the Interim Report.

Although the geotechnical information was gathered under adverse conditions, it is quite adequate for the design of the bridge piers and abutments. Results of the investigation can be summarized as follows. The stratigraphic sequence below the river consists of very dense sand and gravel with boulders and a trace of silt for the entire depth of the test holes (60-feet). This material was unfrozen. Although bedrock was not encountered at this site, information obtained at other sites shows bedrock is generally horizontally bedded shale, silt stone, sandstone and limestone of the Fort Simpson formation of Devonian Age. 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 68'-130'-68'; 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 abutments and piers are recommended for the substructure; scour considerations necessitate the use of piled foundations for the piers and abutments. The proposed layout is shown on Dwg. P301.

Straight girders with tapered end spans are proposed, since it is our opinion that curved soffits do not add to the aesthetic qualities of a structure of this size. To create an impression of strength and stability and to break up the large plane surfaces of the relatively massive piers, we have proposed that the piers be slightly flared toward the base. We have used a shape similar to this on other structures and 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 surfaces sandblasted.

"Spill through" abutments have been shown (Dwg. P302) 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 on piles driven through the approach fill will be substituted.

For the purpose of this preliminary design and cost estimate, H-piles have been assumed for the piers although we share the geotechnical consultant's reservations on the feasibility of driving them through the dense gravel and boulders. 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 piles seated into bedrock and would provide:

- a rational basis for design;
- information of considerable value 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
	Concrete - Deck - f'c - 4,000 p.s.i. - Substructure - 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 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 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; and, although structurally adequate, the riding qualities of a steel deck are inferior to those of a concrete deck. The concrete transverse floor beam and open steel deck investigated are shown on Dwg. P304.

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 center span increased from 130-feet as proposed to 190-feet. However, there was no economic advantage in this system since the weight of structural steel required for the longer center 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 (-2½-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 175-feet overall length range.

TEMPORARY CROSSING:

The proposed location of the temporary crossing is approximately 300-feet upstream 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. P303. 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 River Between Two Mountains. 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 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 River Between Two Mountains 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. 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 October, 1974 and possibly sooner depending on the period of time required for approvals.

Schedule.

- Deliver concrete aggregates to mouth of River Between Two Mountains - Fall 1974.
- Construct temporary bridge - December, 1974 - January, 1975.
- Start abutment construction - January, 1975.
- Start pier construction - February, 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.

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 Reports were 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 shown on the following page.

RIVER BETWEEN TWO MOUNTAINS BRIDGE MILE 411.6

COST ESTIMATE - DECEMBER, 1973

Item	Quantity	Unit	Unit Price	Amount
Excavation & Backfill				
- Rock	--	--	$ --	$ --
- Gravel	300	cu.yd.	50.00	15,000
- Riprap	1,000	cu.yd.	25.00	25,000
Piles	1,600	lin.ft.	30.00	48,000
Concrete				
- Foundations	270	cu.yd.	300.00	81,000
- Piers & Abutments	315	cu.yd.	300.00	94,500
- Deck	305	cu.yd.	350.00	107,000
- Approach Slab	45	cu.yd.	300.00	13,500
Reinforcing Steel	218,000	lb.	0.40	87,000
Structural Steel	110	ton	1,500.00	165,000
Handrail	590	lin.ft.	30.00	17,500
Expansion Joint	65	lin.ft.	100.00	6,500
Bearings	8	each	1,250.00	10,000
Sub-Total				$670,000
15% Contingency				$100,500
7% Engineering & Administration				$ 53,500
TOTAL				$824,000
Temporary Bridge				$200,000