MACKENZIE HIGHWAY

WILLOWLAKE RIVER BRIDGE

PHASE 1 (B)

REPORT

T. LAMB McMANUS & ASSOCIATES LTD.

CONSULTING ENGINEERS

MACKENZIE HIGHWAY

WILLOWLAKE RIVER BRIDGE

PHASE 1 (B)

REPORT

DECEMBER 1973

T. LAMB, McMANUS & ASSOCIATES LTD.

Consulting Engineers

10214 - 112 Street
Edmonton, Alberta

Telephone: 426-0516

MACKENZIE HIGHWAY

WILLOWLAKE RIVER BRIDGE

INDEX

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December 5, 1973.

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

Attention: Mr. A. L. Perley
O.I.C. Civil

Dear Sirs:

Re: Willowlake River Bridge

Enclosed please find the report on our preliminary investigation covering Phase 1 (B) of our commission in connection with the Willowlake River Bridge, Mile 395, Mackenzie Highway.

A complete preliminary design has been carried out based on the finalized concept of a five span continuous steel girder bridge with concrete piers and reinforced concrete deck.

Our estimated cost of the project, based on a construction period of June 1974 to October 1975, is Two Million, Seven Hundred and Seventy-five ($2,775,000.00) Dollars.

We trust this will be satisfactory.

Yours very truly,

T. LAMB, McMANUS AND ASSOCIATES LTD.

R. N. McManus, P. Eng.

RNM:nn
Enc.

COMMISSION

The authorization to complete Phase 1 (B) "Complete Preliminary Design and Finalized Concepts" was contained in a letter from Mr. N. A. Huculak, Regional Highways Engineer, Western Region, dated July 30, 1973, and addressed to T. Lamb, McManus and Associates Ltd. A copy of this letter is attached as Appendix 1 of this report.

WILLOWLAKE RIVER BRIDGE

PHASE 1 (B) REPORT

TERMS OF REFERENCE

The general terms of reference outlined in our Phase 1 (A) report have not been changed to any great degree. However, there have been reports from the environmental consultant, F. F. Slaney & Company Ltd.; the hydrology consultant, Bolter, Parish & Trimble Ltd.; the geotechnical consultant, Acres Consulting Services Ltd.; and a review of the preliminary design submissions by the Mackenzie Highway Environmental Working Group since that time. These reports have all had some effect on the design and estimated cost of the structure since the scour requirements have increased and the pier bases have been lowered. The principal points raised in each of these are discussed and evaluated in the Investigations and Recommendations section of this report.

The requirements governing the design of the structure are that we follow the recommendations of the R.T.A.C. regarding width of roadway and structural geometry. The bridge is to be two lane, HS-25 designed for 60 m.p.h. An overload provision covering HS-40 (Alberta) loading on one centre lane only at 125% of basic stresses is to be considered and its extra cost, if any, estimated.

The most recent editions of the Canadian Standards Association Standard S6 "Design of Highway Bridges" and the American Association of State Highway Officials "Standard Specifications for Highway Bridges" are to govern the design.

Concrete slab decks are to be designed with 1/2" additional cover on top bars as an exposed wearing surface. A design allowance of 30 p.s.f. is to be provided for a future asphalt wearing surface.

Specifically, the letter of July 30, 1973, instructs us to investigate in greater detail Alternate 5 of our Phase 1 (A) report, viz. the five span continuous steel girder structure with reinforced concrete substructure and deck. A copy of this letter is included as Appendix 1 of this report. A subsequent meeting with Department of Public Works highway design personnel was held to discuss a change in bridge alignment. The new alignment consists of rotating the highway centre line about a point of inflection some distance from the crossing, which shifts it upstream enough to protect an archaeological site. Both ends of the bridge will move upstream with a maximum shift at the north end of approximately 75 feet, resulting in a skew angle of about 2 degrees in the crossing. Due to the close proximity of this archaeological site to the roadway right-of-way, it is recommended that its boundaries be protected by the erection of a snow fence or some other suitable type of barrier prior to the start of construction. This barrier should be maintained throughout the duration of construction.

The elevations at the ends of the bridge and the grade of the approach fills were also discussed at this meeting. It was decided that the elevations and grades shown on Drawing No. 2 of our Phase 1 (A) report are to be used.

INVESTIGATIONS AND RECOMMENDATIONS

Site

The site topography 75 feet upstream is not appreciably different from the original location. Since the soil borings were quite similar in all test holes, it seems logical that there will not be any serious differences in sub-soil conditions from those observed by the geotechnical consultant on the original alignment. The small skew angle of approximately 2 degrees which results from shifting the centre line will not be serious. We are of the opinion that the change in alignment will have no effect on the cost or performance of the structure.

Geotechnical

Test borings by the geotechnical consultant were taken at the location of piers for a four span structure. However, the material encountered is similar at corresponding elevations in all test holes so there is no reason to believe that a change in pier location will be significant so far as foundation materials are concerned.

The borings show that the piers are underlain by unfrozen, medium density, silty sands and gravels to a depth of about 20 feet. Below this they encountered very stiff to hard, unfrozen sandy and clayey silt with clay content increasing with depth. Bedrock in this general area is horizontally bedded shale but it was not encountered in any of the test holes even though the deepest was carried to elevation 182.

The report recommends driven friction pile foundations for all piers and abutments. The piles for pier foundations are to be driven to a depth of 50 feet, or refusal, while those for abutments are to be driven to a depth of 60 feet, or refusal. They also recommend that abutments be placed on select, well-compacted granular fill with preloading for two summer months before driving abutment piling. Alternatively, the abutment pile loading should be adjusted for anticipated settlement. The recommended design load is 40 tons for an HP 10 x 57 pile.

The original recommendation that embankment side slopes be 2.5 horizontal to 1 vertical has been amended to 2.0 to 1. A copy of the letter discussing this and other points is included as Appendix No. 2.

We agree in general with the recommendations of the geotechnical consultant as amended by the letter. However, we would propose to use an HP 12 x 53 pile. The 12 inch steel pile will be more stable during driving than the 10 inch, particularly in the lengths required.

The original geotechnical report indicated a 15 foot depth of water at the location of test hole D. It has subsequently been determined that a change in drilling crews led to a mistake of 9 feet in sounding at this point. The actual depth of water to river bed at the time of drilling was 6 feet.

The base of pier has been lowered from elevation 232 to 228 in order to meet the pier scour requirements. The total depth of excavation inside a steel sheet pile cofferdam will be approximately 28 feet below water level. With 20 feet of excavation in silty sands and gravels followed by sandy, clayey silt, it seems likely that a tremie seal will be required, at least in piers 2, 3 and 4.

Hydraulic Considerations

The hydrology consultant has estimated a maximum discharge of 90,000 c.f.s. and a maximum bed scour to elevation 240 under the most extreme conditions. On the basis of a pier shaft 8 feet wide they have recommended that the top of the pier base be at or below elevation 236 for the three north river piers. The top of the south river pier base is to be left at elevation 240. A copy of a letter outlining the background for these decisions is attached as Appendix No. 3.

The hydraulic consultant has also recommended that the piers be designed to withstand local scour down to elevation 224 at piers 2, 3 and 4 and 228 at pier No. 1. The present design calls for an 8 foot thick pier base with the top of base at elevation 236 for piers 2, 3 and 4. All loads are now carried by steel piles which will enter the pier base at approximate elevation 228 except the south pier where this would be elevation 232. Most of the local scour will occur around the upstream nose of the pier and the base has been extended forward to help prevent this. The footing extends 18 feet out to the rear of the pier shaft so that the piles can resist overturning from ice. This should help resist scour from vortices developed at the downstream edge of the pier shaft. In addition to these precautions the front and rear ends of the pier base have been rounded. Although the piles carry all vertical loads and could withstand the lateral ice force, we would not expect local scour to completely remove passive resistance from the downstream end of the pier base. In the event that a tremie seal is required it would extend below the base.

The concern over the fact that the pier nose is bevelled rather than rounded is based on ability to shed driftwood. We would agree that large size driftwood will have less tendency to hang up on a rounded nose than on a bevelled one. However, the Willowlake River banks in the general vicinity of the crossing do not show evidence of large driftwood flows. We have used the bevel nosed pier previously, without problem, on rivers likely to carry more large size driftwood than the Willowlake River. The bevelled nose is cheaper to construct than the rounded shape.

The Mackenzie Highway Environmental Working Group have commented on the erosion of the north bank and aggradation of the south bank. They recommend that the toe of the north approach fill should be more than 27 feet from the bank and that rip-rap cannot be considered to be an effective means of protecting the northern approach embankment. They recommend that the south approach be protected by rip-rap up to the elevation of the willow-covered segment of the bar.

At the location chosen for the crossing, it would appear that the toe of slope at the south side should be approximately at chainage 980 + 20. Using 2:1 embankment slopes and 920 feet of bridge, centre-line of bearings to centre-line of bearings, the toe of slope at the north end will be approximately at chainage 988 + 22. Since the top of the north bank occurs at approximate chainage 987 + 90, there will be about 32 feet of clearance between the toe of slope and the north bank. The hydraulic consultant has commented on the bank erosion in a letter, a copy of which is also included in Appendix No. 3.

While the north bank above the elevation of normal high water is subject to scour, it has fairly good natural armour below this level at present. There seems little likelihood of losing all the bank above normal high water in a single flood. Rip-rap placed on the new approach fill would not protect the fill in the event of bank loss below unless the rip-rap was dug down into the existing bank. We would propose to protect the south approach fill by rip-rap carried above and below the intersection of the fill toe with existing ground level. We do not think it is necessary to rip-rap the north approach fill and it would be useless to do so unless the rip-rap is carried below the level of the existing bank. Since all fills are to be made from select granular material, there should be no problem with them draining after a back-up of water due to an ice jam in the Mackenzie. The gradation of the material, particularly near the surface, is important in order to provide stability during the draining process.

We are satisfied that the bed and pier scour estimates are on the conservative side for a river with the apparent characteristics of the Willowlake River.

Environment

The environmental consultant's requirements in general are to protect the fish and wildlife from pollution and disturbance, save nearby archaeological sites and consider future recreational facilities in the area. The principal points regarding fish are that the river bottom should suffer minimum disturbance during construction, that the maximum possible river channel be maintained, that fuel cache and camp be above known high water levels and that fishing by construction workers be controlled. The location of the fuel cache and camp as well as restrictions on fishing can be covered by the general construction specifications. The width of waterway is a matter of design.

The five span continuous steel girder structure will have four river piers. The lengths of span are such that the two north river piers approximately straddle the thalweg. While the environmental consultant has indicated a preference for the four span structure, it would have the disadvantage that one pier would be very close to, if not in, the thalweg.

Each pier shaft is eight feet wide so that four piers represent about 3 1/2 per cent of the waterway during flood. During low water one of the piers would be in the dry so the remaining three would occupy about 4 1/2 per cent of the waterway. This does not appear to represent a significant change in the stream so far as fish migration or channel capacity is concerned and would not increase the velocity except immediately adjacent to a pier. During construction all water piers will be built using steel sheet cofferdams. While some falsework may be necessary the disturbance to the river bottom should be minimal and the main channel in the vicinity of the thalweg should be free over at least 100 foot of span at all times.

The contractor would likely start moving onto the site during July and August and start construction of the south abutment. Pier No. 1 would be in the dry shortly after and construction could start on the cofferdam for its base. Material excavated for the pier base would be used for backfill around the pier shaft. The upper two feet of fill will be select size cobblestone so that about 500 cubic yards of excavated material from each pier will have to be disposed of in nearby borrow pits. A short work bridge would likely be built out to Pier No. 2 so that work could be started on its cofferdam before freeze up. All work on Piers No. 3 and 4 and the north abutment would be delayed until the ice would be strong enough to act as a work bridge. This schedule would involve barge loading and a camp site on the south side of the river with no work on the north side until an ice bridge could be used. The disturbance to the main channel of the river and fish migration should be minimal under these conditions.

The archaeological site known to be near the original location has been the reason for the proposed shift in alignment.

So far as wild life and trap lines are concerned the specifications will have to spell out general and specific limitations. The construction camp for the bridge will never contain "several hundred men." We would anticipate a normal crew of 20 to 30 men with a maximum at any one time of 40.

The contractor, however, must have a certain amount of space for camp site and storage of materials and equipment. While use of the right-of-way on the north side may be adequate he will require at least 5 acres on the south side for equipment repair and storage, aggregate, cement, stockpiling of structural steel, as well as loading space for barges. He will require additional space for a camp site which could be chosen with future recreational needs in mind. The former Department of Public Works camp site could be re-activated as a construction camp. However, the area is at an approximate elevation of 286 and camp buildings would have to be elevated by about 5 feet to clear the observed high water level on the Mackenzie that has resulted from ice jams during the last 30 years.

A clearing about 300 feet downstream of the bridge site adjacent to the river bank contains about 7 acres. This area could also be subjected to flooding in a similar fashion to the camp site but it would serve for storage of gravel, structural steel and other non-polluting construction materials not susceptible to damage should the area become inundated for a short period of time in the spring. Fuel cache, cement and equipment storage would have to be located on higher ground along the highway right-of-way.

The environmental consultant points out the recreational potential of the area and recommends that parking shoulders be provided at the bridge ends and that borrow pits be shaped for future recreational use. We would recommend that parking areas be provided nearby but completely off the highway. Parking shoulders near the ends of a bridge are dangerous, in our opinion, and should be avoided. Borrow pits should generally be located and operated so as to cause the minimum disruption to the area. Unless a specific future use is predetermined, it will be difficult to anticipate the shape that would be most suitable. The proposal to stockpile and replace all organic top soil is excellent since this type of material, necessary for growth, will not be readily available in this area.

In general, the environmental report is not critical of the effect the completed structure will have on the fish and wildlife. Most of the points raised apply to the damaging effects poor construction methods might have on the environment.

LIST OF DRAWINGS

A number of drawings have been included in order to clarify the general appearance and structural details of the bridge. These are:

1. Perspective - Key Map - Site Plan
2. General Layout
3. Typical Pier Details - Abutment and Deck Details
4. Plate Girder

DESIGN

We have carried out the design of a typical river pier and abutment basing pile lengths and capacities on the recommendations of the geotechnical consultant. The pier scour requirements of the hydrology consultant have been met by lowering the three northerly river piers by four feet. The top of the south pier base is at elevation 240 while the other three are at elevation 236. The piers and foundations have been designed to withstand an ice force of 700 kips acting at elevation 275 or alternatively an ice force of 1100 kips acting at elevation 260. This is in accordance with clause 5.1.18.2 of the proposed CSA Standard S6 and assumes four feet of ice. It has been assumed that the crushing strength of ice at the lower elevation could be at 300 p.s.i. at break-up but would be at no more than 200 p.s.i. when the water reaches the higher stage.

We have also refined the plate girder design to the extent of varying the flange sections and checking moments, shears and stresses by computer program. The structural steel should conform to the requirements of C.S.A. G40.21 grade 50A controlled rolled condition satisfying the impact requirement of 20 ft. lb. at -20° F for the standard charpy V notch specimens.

Because of the remote area in which the bridge is located, maintenance items should be minimized. We, therefore, recommend that the weathering steel selected be merely cleaned of all dirt and concrete splatter after construction of the deck is complete and left unpainted. The deck drains and guardrailing should be given a hot dip galvanized coating.

Deck joints should be self-clearing steel finger plate assemblies with suitable splash plates to keep abutment bearings free of accumulated mud and dirt. Conventional steel rockers and fixed bearings are recommended.

QUANTITIES AND ESTIMATES OF COST

The quantities involved in the project have been estimated together with their unit costs in place; these are shown in Table No. 1. The same total cost is arrived at in Table No. 2 on the basis of estimating labour, material and equipment costs for the substructure, superstructure and approach embankments. Since the quantities at this time are approximate, the total costs have been rounded off to the nearest one thousand dollars in each case. The costs of materials and labour have been escalating at a rapid rate during the past year and particularly in the past few months. This is due partly to increased labour rates, partly to an excess of demand over supply and partly to the money market's effect on imported materials. There are no signs at present of an improvement in this situation although we would expect the rate of escalation to decrease. The most economical structure will result if decisions are reached as quickly as possible, materials are ordered with sufficient lead time to permit mill orders and a continuous construction sequence is followed.

TABLE NO. 1

QUANTITIES AND ESTIMATED COSTS

ITEM	STANDARD	DETAIL	APPROXIMATE QUANTITY	UNIT COST IN PLACE	TOTAL
Move In and Move Out					130,000
Cofferdams & Work Bridge			2 Cofferdams 1 Re-use each		250,000
Excavation	Wet	Piers	6,400 cu.yd.	15.00	96,000
Piling	CSA-G40.21W	HP 12 x 53	18,000 lin.ft.	21.00	378,000
Backfill	Common	Piers	4,000 cu.yd.	8.00	32,000
Backfill	Select	Piers	800 cu.yd.	50.00	40,000
Concrete	3,000 p.s.i. 1½" aggregate	Substructure including forming, pouring & finish	3,100 cu.yd.	145.00	450,000
Concrete	4,000 p.s.i. 3/4"aggregate	Deck including forming, pouring & finish	1,050 cu.yd.	240.00	252,000
Re-Bar	CSA-G30.1	Substructure and Deck	216 tons	810.00	175,000
Structural	CSA-G40.21-50A & CSA-G40.21W	Girders, Bracing, Bearings & Miscellaneous	508 Tons	1,250.00	635,000
Guardrail	CSA-G40.21-44W	Galvanized	1,900 lin.ft.	30.00	57,000
Embankment	Granular	Bridge Approaches	25,000 cu.yd.	8.00	200,000
Rip-Rap	1500 lb.max.	South Approach	2,000 sq.yd.	40.00	80,000
				TOTAL:-	$2,775,000

TABLE NO. 2

LABOUR, MATERIAL, EQUIPMENT
ESTIMATE OF COST

Move-In and Substructure:		
Labour (including subsistence)	$600,000.00	
Material	685,000.00	
Equipment	150,000.00	
	$1,435,000.00	$1,435,000.00
Superstructure & Move Out:		
Labour	$250,000.00	
Material	760,000.00	
Equipment	50,000.00	
	$1,060,000.00	$1,060,000.00
Embankment and Rip-Rap	$ 280,000.00	$ 280,000.00
	TOTAL:-	$2,775,000.00

There is no additional cost to cover the overload requirement since the total load on the structure under this circumstance is less than the design load.

SCHEDULE FOR DESIGN & CONSTRUCTION

The estimate of cost has been based on the assumption of an uninterrupted construction schedule. The contract should be awarded in the spring of 1974 so that the general contractor can organize transport, equipment and materials in time to ship them to the site before low water. The most critical item will be concrete aggregate for the substructure since this will involve processing as well as shipping. Supplies of suitable aggregate are available at Wrigley but they must be processed for barging to the site before low water in late August or brought to the site for processing there. In the event that the contract is awarded too late to use barges to the site, it will require transportation by winter road. This will mean that the construction period will not be uninterrupted and will increase costs.

We would, therefore, recommend the following schedule:

1.	December 1973 - February 1974	Design and preparation of contract drawings and documents
2.	February 1974	Tender period of one month
3.	April 1974	Contract Award
4.	May and June 1974	Organize material, equipment and transport
5.	July and August 1974	Process aggregate and move in to site
6.	September 1974 to March 1975	Construct sub-structure and fabricate and transport structural steel
7.	March and April 1975	Erect girders
8.	May 1975 to August 1975	Form and pour deck and complete fill and rip-rap and move heavy equipment out
9.	September and October 1975	Erect guardrail, paint and general cleanup
10.	January 1976	Remove camp and remaining equipment by winter road

The successful contractor may vary this schedule to the extent that the completion date could be earlier. However, the access to the site is limited by the time of year. Furthermore, the access to pier locations is best during the winter and girder erection will be most economical using the ice.

CONCLUSIONS

Using the schedule outlined above, which in our opinion is the most economical, the estimated cost of the Willowlake River bridge is Two Million, Seven Hundred and Seventy-five ($2,775,000.00) Dollars. This assumes a contract award in April 1974 and an eighteen month organization and construction period.

The estimate is almost seven hundred thousand dollars higher than that given in our Phase 1 (A) report. A number of factors have influenced this increase in estimated cost among which are:

1. Labour costs appear to be rising more quickly than we had assumed.

2. Material costs will reflect shortages as well as the rise in labour costs. These have risen drastically since the energy crisis starting in October 1973. The rate of increase will hopefully level off in 1974.

3. The depth of pier bases as a result of scour predictions and length of piling recommended in the geotechnical report are both more than previously assumed. While these changes are not drastic, they represent increased costs.

4. The cost of concrete aggregates will be higher with each contractor responsible for providing his own. Our Phase 1 (A) report assumed that a separate aggregate contract might be let in advance so that supplies would be available no further away than Wrigley.

We are still of the opinion that the concrete substructure is the best for this site. The principal reasons for this are the potential scour depth and relatively high elevation at which the ice exerts its maximum overturning effect. In addition to the large moments which must be resisted, it may well be impossible to construct any pier base without using a concrete seal in the bottom of the cofferdam. If aggregate is required for seal concrete it may as well be available for the piers.

A copy of a letter from Acres Consulting Services Ltd. is attached to this report as Appendix 2. The letter outlines the possible problems that could occur in dewatering cofferdams.

Our Phase 1 (A) report, Appendix II, estimated the extra cost of an all steel structure as $450,000.00 more than a bridge with comparable spans but constructed with composite concrete deck and conventional concrete substructure. Because of the potential depth of scour and the high overturning ice forces, we would expect this differential would not reduce even though our estimated cost of the conventional structure has increased. In addition to this structural steel is a less stable priced commodity than concrete at this time.

We are also of the opinion that delays in construction will not reduce costs since they are related primarily to labour and material.

Respectfully submitted,

T. LAMB, McMANUS AND ASSOCIATES LTD.

RNM:nn

R. N. McManus, P. Eng.

APPENDIX 1

Public Works Canada — Travaux publics Canada

Western Region — Région de l'Ouest

July 30, 1973

T. Lamb, McManus & Associates Ltd.
10214 - 112 Street
Edmonton, Alberta
T5K 1M5

RE: WILLOW LAKE RIVER BRIDGE - MACKENZIE HIGHWAY MILE 395

We have now established new schedules for the development of highway design documents covering miles 346 - 550 of the Mackenzie Highway. Final highway design covering the section adjacent to the Willow Lake crossing have been scheduled for submission by August 21, 1973. It is our intent to incorporate as much detail as possible on the Willow Lake structure within this submission. As a minimum we must have the grade line, abutment locations, approach fill slopes rates and riprap locations and levels resolved by August 15. Would you therefore, immediately implement Phase 1B of our Agreement following, as recommended, alternate 5 of your report on Phase 1B utilizing the 5 span, reinforced concrete deck approach. If at all possible, we would appreciate completion of the report on Phase 1B by August 21, since the design package for the section of highway including this structure is scheduled for that date.

These instructions to implement Phase 1B are being provided without the benefit of the Environmental Working Groups reaction to the Phase 1A report. Their response to Phase 1A will be communicated to you when it arrives; however, we do not anticipate that their comments will require redirection to persue a design approach other than that generally portrayed in alternate No. 5

10th Floor — 10ᵉ niveau
One Thornton Court — Un Thornton Court
P.O. Box 488 — B.P. 488
Edmonton 15, Alberta — Edmonton 15 (Alberta)

.......2

The geotechnical report for this structure site has been provided to you on the past few weeks. Resolution of your points raised in review of same have been passed to Acres for their follow-up. If you find that additional field data is required, would you please outline same to this office within the next 3 weeks since we are gathering requirements of this nature in anticipation of organizing a "helicopter support" drilling program for the area commencing September 15, 1973.

N. A. Huculak
Regional Highways Engineer
Western Region

APPENDIX 2

August 7, 1973

2641

Department of Public Works
Government of Canada
10th Floor - One Thornton Court
P.O. Box 480
Edmonton, Alberta

Attention: Mr. N. Huculak

Dear Mr. Huculak:

Re: Willowlake River Bridge
Mile 394 - Foundation
Report

We thank you for your letter reference 9305-52-309 dated July 19, 1973, and the enclosed copy of a letter from T. Lamb, McManus and Associates Ltd.

Though the scope of the study carried out was not specifically intended to establish the design parameters for the cofferdam, we concur with T. Lamb that dewatering the cofferdams will create a problem unless a concrete seal is used in the base or the cofferdam is taken down through the 20 foot layer of sand or gravel in the river bed to the underlying silty clay.

We would estimate that the permeability of the sand and gravel deposits is of the order of 1.0 cm/sec. with an internal angle of friction of 35°.

The six inch stone size recommended in the report constitutes the minimum which would be acceptable. No information was available on the maximum water velocities to be expected at the time the report was written and, as noted in the report, heavier stone will be required for high velocity areas.

The 2.5 to 1 embankment side slopes were recommended by virtue of the relatively weak foundation materials with high moisture content on the bridge approaches. Where the embankment is founded on sand and/or gravel as is the case with the end slopes of the embankment at the bridge abutments we would agree that the slope may be constructed at 2 to 1.

ACRES CONSULTING SERVICES LIMITED.

Suite 900 - 125 - 9 Ave. S.E.,
Calgary, Alberta
T2G 0P6

Mr. N. Huculak
Page 2
August 7, 1973

Without further investigation it is not possible to determine exact reasons for the high fluctuation in moisture content in 394-S-F. However, the soils at depth are eratic in the area in question and this may account for the fluctuation. Alternatively the high moisture content may be due to an isolated ice lens in that particular stratum.

We trust that this is the information which you require. If however, you require anything further, we would be pleased to supply it.

If you wish a meeting to take place between our soils experts and Mr. A. Morrison of T. Lamb, we would be pleased to arrange it. However, Dr. G. Watson will not be in Canada until the end of August and Dr. D. Matheson is on an extended assignment in our Niagara Falls office. The latter could be made available if you so wish at relatively short notice.

Yours very truly,

L.O. Gloin

L.O. Gloin
Executive Engineer.

DAJ/jp

Copy Given to T.L. Mulk. by hand Aug. 10
Jon

ACRES CONSULTING SERVICES LIMITED.

APPENDIX 3

Bolter Parish Trimble Ltd.
CONSULTING ENGINEERS

11805 - 149 STREET
EDMONTON, ALBERTA
T5L 2J1
TELEPHONE 452-7810

November 14, 1973

File 115-3

Mr. N. Huculak, P. Eng.,
Regional Highways Engineer,
Department of Public Works,
One Thornton Court,
EDMONTON, Alberta.

Dear Mr. Huculak;

RE: Willowlake River, Mackenzie Highway.
Response to E.W.G. September, 1973 Report.

The following is in response to M.H.E.W.G. Item 6(0)7.(i) on page 17.

Item 6(a) - Design Highwater Level Somewhat High.

Drawing 115-3-55 indicates possible Willowlake River high water and ice run at elevation 277 approximately 21 feet above September 22nd water elevation and 12 feet above the June 15th, 1972 water elevation (Field Survey Datum).

H.D.A.C. (May 23) have estimated a water elevation of 269.5 based on low flow in the Mackenzie and a 50 year flood in Willowlake River.

It is assumed that the statement 'somewhat high' is based on comparing the above two values. They are not however comparable since the H.D.A.C. estimate is used to determine maximum velocity (and scour) whereas elevation 277 is an estimate of the high water level during an ice run on Willowlake River when the Mackenzie is moderately high. Water levels can be expected higher than elevation 269.5 depending on the Mackenzie River levels and can be as high as our estimated elevation 298 during an ice jam on the Mackenzie (confirmed by E.W.G. Items 6(f) page 14).

Water levels, ice conditions and corresponding velocities can vary greatly for a given flood on Willowlake River depending on the level of the Mackenzie River.

Continued.......

We believe that H.D.A.C. will concur that their estimate of 269.5 was not intended as a design flood and ice run level but only to determine maximum velocities and scour potential.

Our estimate of elevation 277 was based on the stage discharge and slope of the Willowlake River upstream at the Water Survey of Canada gauge. This was applied at the mouth of the river assuming that the Mackenzie was at a level producing neither a M1 or M2 drawdown at the bridge site. This results in a 16 foot rise in stage on the Mackenzie and a 21 foot rise in stage at the bridge site. The 16 foot rise in stage on the Mackenzie due to flow without ice would be an extreme flood, however it is considered a moderate level when considering ice jam possibilities.

Item 6(b) Velocity Comparisons.

Design velocities do not always occur at design highwater elevations because of such effects as aggradation and degradation of bed, constriction or rapids downstream, or because of a choice in a range of variables (such as channel roughness) in order to make conservative estimates of possible conditions. Bridge designers are usually concerned with the 'highest' design highwater and the 'highest' design velocities. The 'highest' design velocities will thus occur at a lower elevation than the 'highest' design highwater. This factor possibly has caused some misunderstanding of our reported values at this and other stream crossings.

Our computed mean velocities for a design flood of 90,000 c.f.s. occurring at elevation 277 are 4.95 f.p.s. (4.9) in the natural channel and 5.2 f.p.s. (5.3) through the bridge opening. Values in brackets are H.D.A.C. values.

Item 6(c) Pier Scour.

This item was considered outside our terms of reference since it required the bridge designers pier configuration, location, etc. which were prepared subsequent to our reports. As requested, we will now undertake this analysis when we receive the required information from the bridge designers. The following are our findings and recommendations for the Willowlake River Piers based on the Typical Pier Details and General Layout (5 span bridge) preliminary drawings (undated, un-numbered) received from T. Lamb, McManus and Associates Ltd. October 26, 1973.

(i) Elevation of top of Footings.

The top of the footings must be below the estimated general bed scour level (Elevation 240 to 238, our letter of September 14, elevation 238 by H.D.A.C.).

Continued.....

We recommend that the footing top be at least one half the pier shaft width below the estimated general bed scour level (1). The footing tops should thus be placed no higher than elevation 236 for the three main river piers.

The first river pier on the south side is on the inside of the bend on a point bar in the natural channel which builds up as the north bank recedes. The channel cross section is triangular in shape as expected since it is on a bend. On a straight reach the footing tops should always be at the same elevation, however, on the inside of a well established bend as in the case here, the top can be raised since less scour is expected. Allowance however, has to be made for additional scour expected due to blockage of overbank flow by the bridge fill. The estimated general scour at this pier is elevation 254 and the allowance for scour due to overbank flow is 14 feet. It is thus recommended that the footing top for this pier be no higher than elevation 240.

(ii) Local Pier Scour.

Based on the blunt bevel-nosed pier proposed, a shaft width of 8 feet, and assuming the footing top is below the general bed scour level, the following calculations were made:

(a) R.T.A.C. Method (2)

The relatively blunt bevel-nosed pier will have a multiplying factor between the round-nosed and square-nosed pier. Using a factor of 1.75 the allowance for local scour due to the pier is 14 feet. Scour allowance for a blunt-nosed pier would be 16 feet and a round-nosed pier 12 feet.

(b) Shen Method (3)

U = Approach Velocity (Main River Piers) .... 10.4 f.p.s.
b = pier shaft width = 8 feet
ν = kinematic viscosity = $\left(\frac{70}{T}\right)^{0.89} \times 10^{-5}$ where T = °F

For T = 60° $\nu = 1.17 \times 10^{-5}$
and $R = \frac{Ub}{\nu} = \frac{10.4 \times 8}{1.17 \times 10^{-5}} = 7.1 \times 10^{6}$

dse = equilibrium scour depth = $.00073\ R^{0.619}$ (Based on Eq. 21, maximum of Eq. 21, 25, and 26)
$\therefore$ dse = 13 feet

Continued......

(1) Neill, C.R., River Bed Scour 1964, Canadian Good Roads Association Technical Publication No. 23, Pg. 30.
(2) Guide to Bridge Hydraulics, Roads and Transportation Association of Canada, 1973, Edited by C.R. Neill
(3) Method proposed by Hsieh W. Shen, 1969, Colorado State University, paper presented to A.S.C.E. Hydraulics Division. Method quoted by H.D.A.C.

(c) H.D.A.C. Value

The equilibrium scour depth quoted is 16 feet.

(d) Summary

It is recommended that the piers be designed to withstand local scour down to elevation 224.0. The footings must be placed on piles and the piles designed to withstand undercutting of the footings.

(iii) Footings

Footings can limit the amount of local scour if placed below general bed scour levels and constructed to proper dimensions (Ref. (1) p.p. 30 - 31 and Ref. (2) p.p. 97). Piers require a footing or apron of at least a pier shaft width all around the pier measured horizontally from the shaft in order to limit the scour. The area of worst local scour usually occurs around the upstream nose and it is recommended that the footing-apron on the upstream half of the pier be constructed 1.5 x the pier shaft width (12 feet) (Ref. (2) p.p. 113).

The possible hydraulic conditions at this site are complex because of the proximity to the Mackenzie River. It is therefore recommended that the provision of the footing-apron be considered as a safety factor and that no reduction in the local pier scour allowance be made. In addition, driftwood or ice jamming at the nose can increase the effective pier width and the scour depth making such a safety factor desirable.

(iv) Shape of Nose in Plan

It is recommended that a semicircular pier nose in the plan be considered in lieu of the bevel-nosed pier proposed. 'General experience is that a pier nose semicircular in plan, and vertical or only slightly raked in profile, is best for discouraging driftwood accumulations' (Ref.(2)p.p. 97).

Item 6(d) Bed Scour.

This item was reported on in our letter of September 14, 1973. General agreement seems to have been reached that the design general bed scour level could take place down to elevation 240 to 233.

Yours very truly,

BOLTER PARISH TRIMBLE LTD.

R.P. Parish

R.P. Parish, P. Eng.

RPP:nb
c.c. Mr. R.N. McManus, P. Eng.
T. Lamb, McManus & Associates Ltd.

Bolter Parish Trimble Ltd.
CONSULTING ENGINEERS

11805 - 149 STREET
EDMONTON, ALBERTA
T5L 2J1
TELEPHONE 452-7810

November 14, 1973

File 115-3

Mr. N. Huculak, P. Eng.,
Regional Highways Engineer,
Department of Public Works,
One Thornton Court,
EDMONTON, Alberta

Dear Mr. Huculak;

RE: Willowlake River, Mackenzie Highway.
Additional Comments, E.W.G. September, 1973 Report.

The following comments from our firm were not requested by E.W.G., however they may assist the Department of Public Works in their response to E.W.G.

Item 6(f) and 6(0) 8(i) North Bank Erosion

Our January report stated that this bank is receding and moving north. Based on our examination of the aerial photos the H.D.A.C. estimate of 29 feet in 25 years appears reasonable and within the range we expected from our original examination. Our examination was relatively unsophisticated and resulted in a range of possible recedence. Perhaps even with the sophisticated method used by H.D.A.C. their estimates resulted in a range and it would be pertinent to know if the 29 feet was a minimum, average, or maximum that may have occurred.

If indeed rip-rap properly designed and installed is not effective in controlling the recedence then the only alternate is to place the toe as far back on the bank as it is likely to recede over the life of the structure. However, we believe that properly designed protection can be effective in controlling the loss of the abutment fill and in addition the entire north bank could be prevented from receding, if it was deemed desirable, by properly designed channel training works.

Continued.....

The choice of protecting the abutment fill, controlling the north bank recedence, or placing the fill back from the bank is an economic one as long as such choices are compatible with the environmental constraints. Another choice exists involving placing the toe back an intermediate amount to allow for some erosion with a view to protecting the abutment and/or controlling the recedence at some future date thus postponing capital costs. Possible loss of the abutment fill should be considered with this choice and the abutments should be founded on piles and designed to withstand the loss of the fill. Environmental damage caused by loss of fill can be kept to a minimum if the fill is constructed with granular material as recommended in our letter of January 25th.

In our January report it was assumed that setting the fill back an intermediate amount thus postponing capital costs would be the economic choice. If required, we could design the north fill slope protection and/or channel training so that cost comparisons can be made with additional bridge costs.

Item 6(g), 6(j), 6(k) and 6(0)8(ii) Location of South Fill and Slope Protection.

We concur with E.W.G. that the south approach fill should be protected by properly designed slope protection. This requirement was indicated in our January report. Such a design would include an apron or equivalent to protect the slope protection from scour caused by the constriction of overbank flow. The final design of slope protection works would show details of the extent, configuration, thickness, filter material, specifications and alternates etc., for the slope protection and aprons.

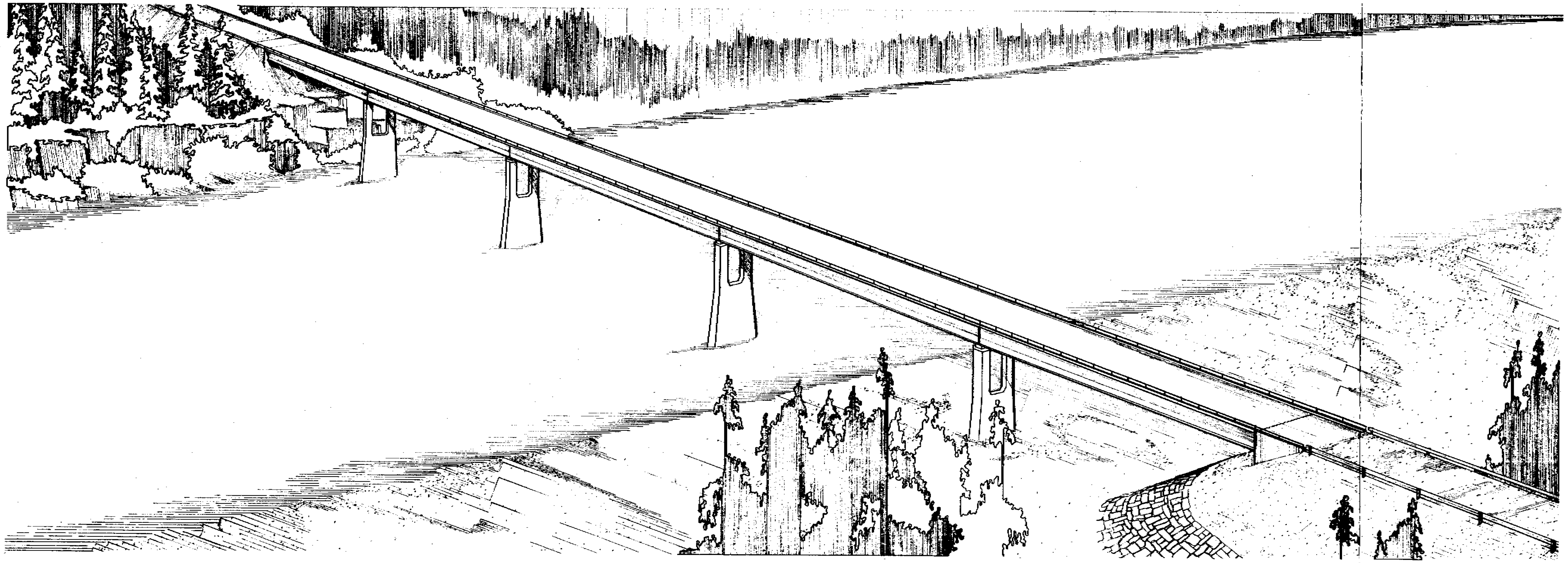
Yours very truly,

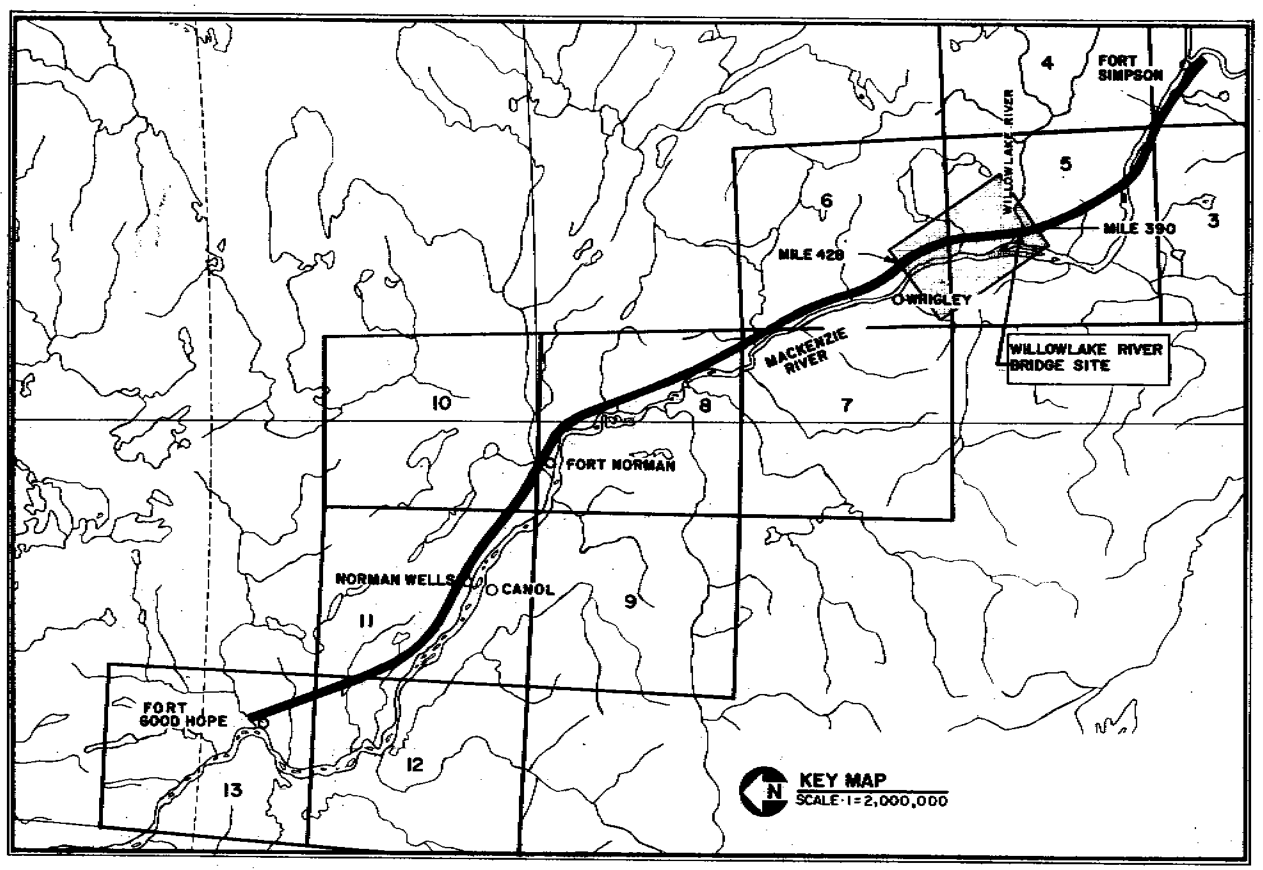
BOLTER PARISH TRIMBLE LTD.

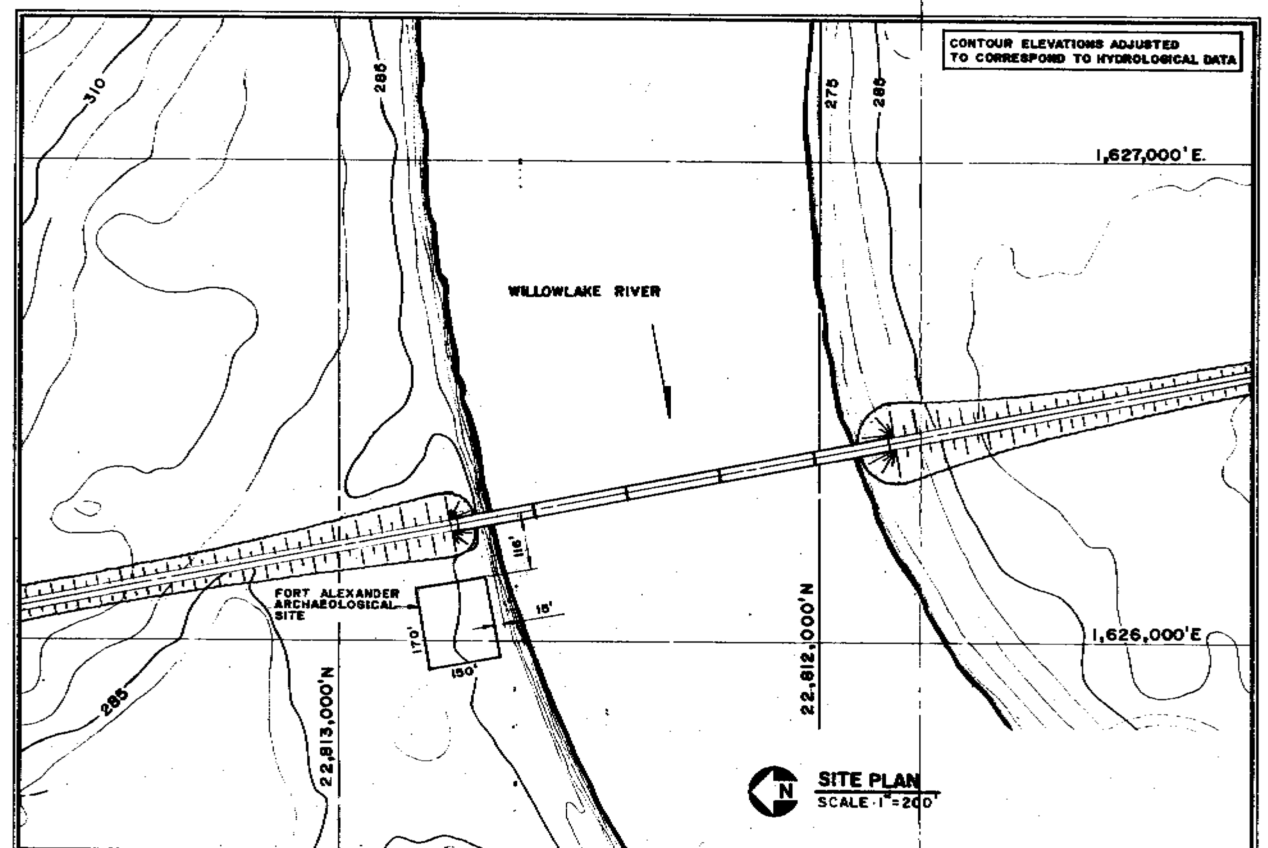
R.P. Parish

R.P. Parish, P. Eng.

RPP:nb
c.c. T.Lamb, McManus & Associates Ltd.







Department of Public Works
Ministère des Travaux publics

T. LAMB, McMANUS & ASSOCIATES LTD.
CONSULTING ENGINEERS
EDMONTON CALGARY WINNIPEG

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

revisions date

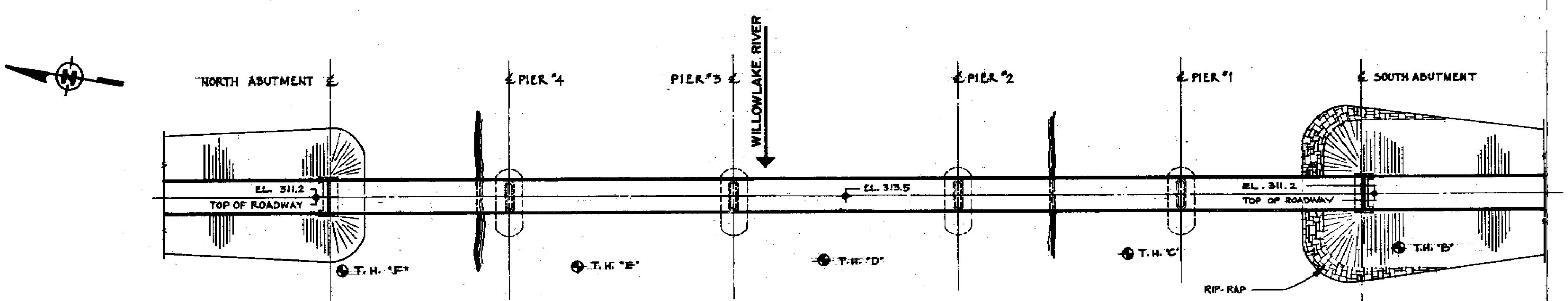
project title titre de projet

MACKENZIE HIGHWAY
WILLOWLAKE RIVER BRIDGE
MILE 395·0

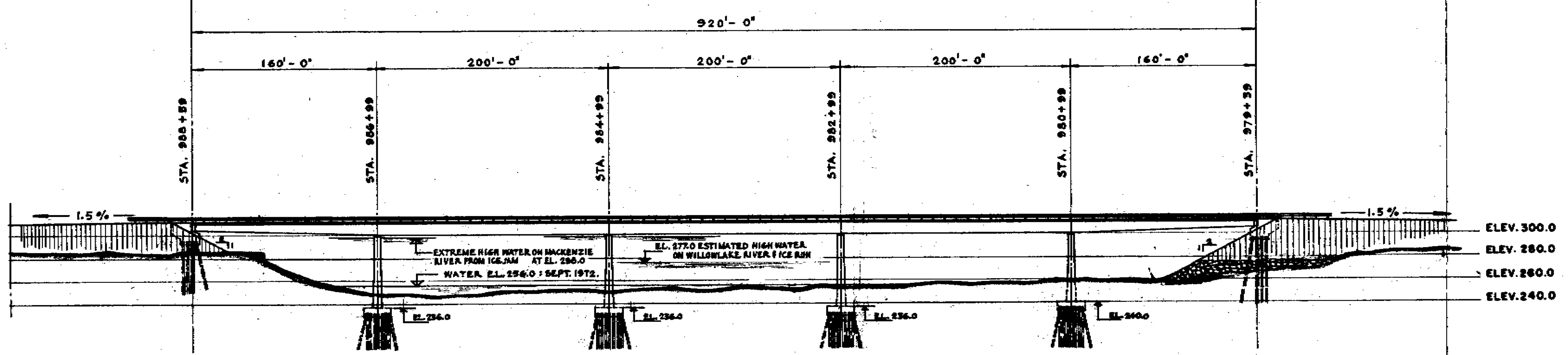
drawing title titre du dessin

PERSPECTIVE
KEY MAP
SITE PLAN

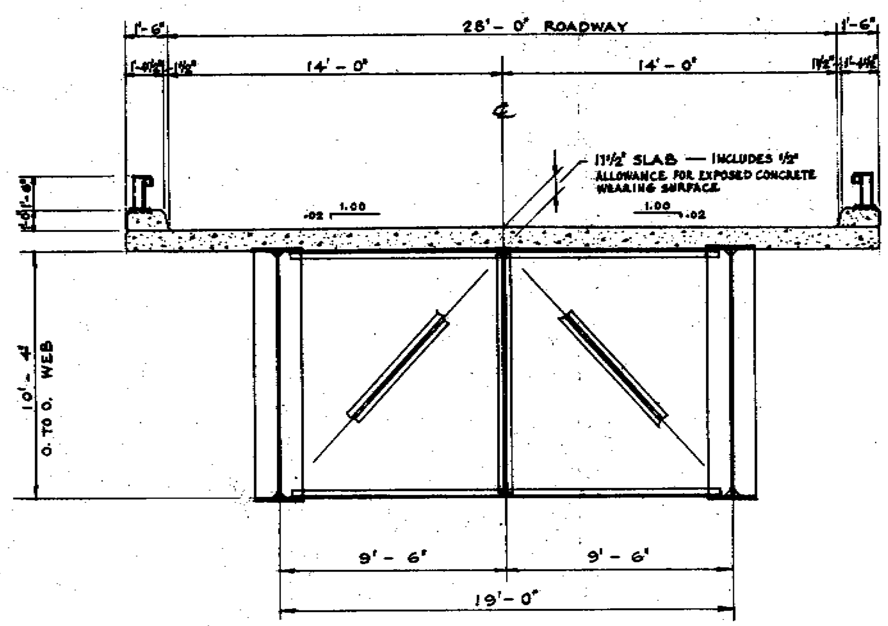
designed by	R.M. MORISON	conçu par
date	DECEMBER 5, 1973	
drawn by	d.g. adrian	dessiné par
date	DECEMBER 5, 1973	
reviewed by		examiné par
date		
approved by		approuvé par
date		
Tender		Soumission
D.P.W. Project Manager		Administrateur de projets M.T.P.
project number	85032	no. du projet
drawing no.	1	dessin no.



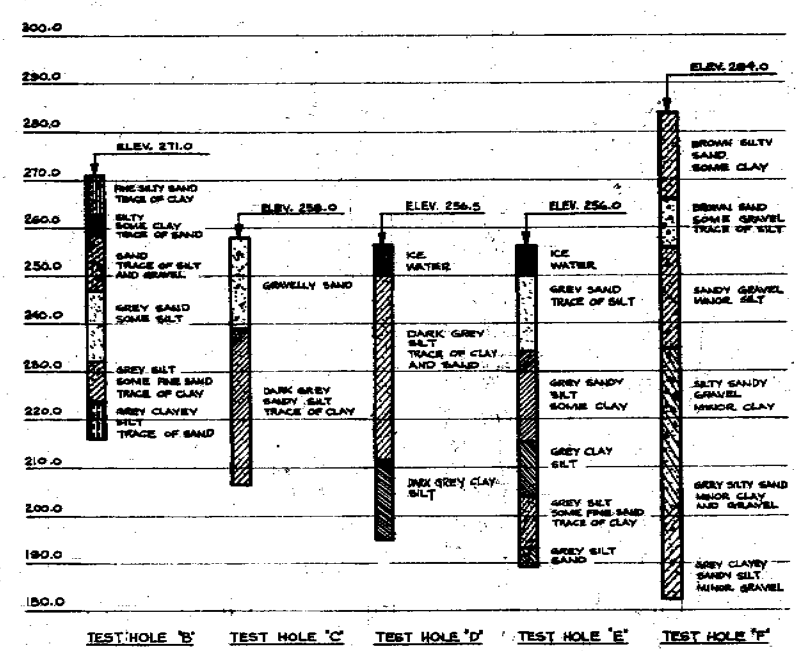
PLAN OF BRIDGE
SCALE, 1" = 60'-0"



ELEVATION OF BRIDGE
SCALE 1" = 60'-0"



CROSS SECTION
SCALE, 1/4" = 1'-0"



TEST HOLE DATA

NOTES

I SPECIFICATIONS

DESIGN IN ACCORDANCE WITH C.S.A. STANDARD S6 – 1974 "DESIGN OF HIGHWAY BRIDGES AND THE A A S H O" STANDARD SPECIFICATION FOR HIGHWAY BRIDGES 1973.

I DESIGN LOADS

DEAD LOAD OF SUPERSTRUCTURE, — 6.5 KIPS PER FOOT OF BRIDGE
— FUTURE ASPHALT SURFACING ALLOWANCE — 30 P.S.F. OF DECK SURFACE

LIVE LOAD OF SUPERSTRUCTURE: —
NORMAL — STANDARD HS 25 (AASHO) PLUS IMPACT AT NORMAL DESIGN STRESS LEVELS. EACH GIRDER DESIGNED FOR 1.21 LANES OF TRAFFIC.
OVERLOAD PROVISION — SINGLE HS 40 TRUCK TRAVELLING IN THE CENTRE OF THE BRIDGE PLUS IMPACT WITH STRESS LEVELS TO 125% OF BASIC UNIT VALUES. THE REAR AXLES OF TRUCK AND TRAILER TO BE DUAL AT 4 FT. MINIMUM ON CENTRE WITH THE MINIMUM DISTANCE BETWEEN THE REAR TRUCK AXLES AND TRAILER AXLES TO BE 24 FEET.

ICE LOADING — HIGH LEVEL ICE FORCE OF 700 KIPS AT ELEVATION 275.0 DETERMINED ON BASIS OF 48 INCH THICKNESS AT A CRUSHING STRENGTH OF 200 P.S.I..

BREAK-UP — ICE FORCE OF 1100 KIPS AT ELEVATION 260.0 DETERMINED ON BASIS OF 48 INCH THICKNESS AT A CRUSHING STRENGTH OF 300 P.S.I..

LATERAL ICE FORCE — 15% OF VALUE ACTING ON LONGITUDINAL AXIS OF PIER.

WIND LOADING — 50 LBS PER SQ. FT. OF EXPOSED AREA IN ELEVATION VIEW.
— 600 LBS PER FOOT OF BRIDGE FOR WIND ACTING NORMAL TO BRIDGE CENTRELINE.

I MATERIALS

REINFORCING — INTERMEDIATE GRADE DEFORMED BARS IN ACCORDANCE WITH C.S.A. SPECIFICATION G30.1.

CONCRETE: —
SUBSTRUCTURE — CLASS I, 1½ INCH MAXIMUM SIZE AGGREGATE, AIR ENTRAINED, HAVING 28 DAY STRENGTH OF 3,000 P.S.I..
DECK — CLASS II, ¾ INCH MAXIMUM SIZE AGGREGATE, AIR ENTRAINED, HAVING 28 DAY STRENGTH OF 4,000 P.S.I..

STRUCTURAL STEEL — STEEL FOR GIRDERS, BRACING, ROCKERS, EXPANSION ASSEMBLIES AND PIER NOSE PLATES TO BE C.S.A. G40.21 GRADE 50A WITH SPECIFIED LOW TEMPERATURE IMPACT REQUIREMENT. STEEL TO BE LEFT UNPAINTED.

H – PILING — C.S.A. G40.21 — W

HANDRAIL AND DECK DRAINS — HANDRAIL AND DECK DRAINS TO BE C.S.A. G40.21 GRADE 44-W STEEL WITH GALVANIZED SURFACE.

RIP-RAP — CLASS 2 WITH NOMINAL SIZE OF 20 INCH AND MAXIMUM SIZE OF 30 INCH OR 1500 LBS.

Department of Public Works
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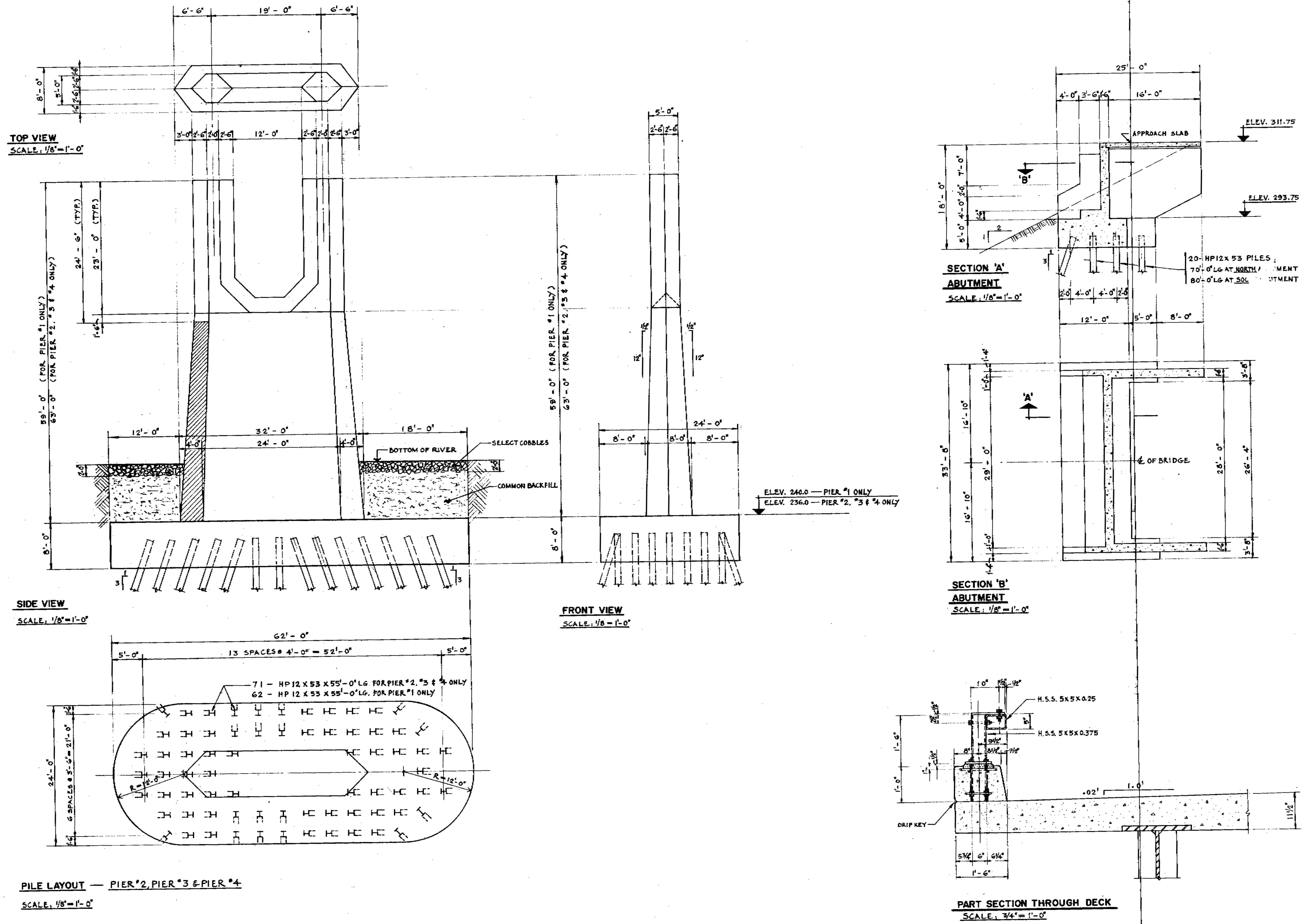
T. LAMB, McMANUS & ASSOCIATES LTD.
CONSULTING ENGINEERS
EDMONTON CALGARY WINNIPEG

THE ASSOCIATION OF PROFESSIONAL ENGINEERS OF ALBERTA
PERMIT NUMBER P128
T. LAMB, McMANUS & ASSOCIATES LTD.

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 WILLOWLAKE RIVER BRIDGE MILE 395.0
drawing title / titre du dessin	GENERAL LAYOUT
designed by / conçu par	R. M. MORISON
date	DECEMBER 5, 1973
drawn by / dessiné par	C. CHOU
date	DECEMBER 5, 1973
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.	2



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CONSULTING ENGINEERS
EDMONTON CALGARY WINNIPEG

PROFESSIONAL ENGINEER — R. M. MORISON — ALBERTA

THE ASSOCIATION OF PROFESSIONAL ENGINEERS OF ALBERTA
PERMIT NUMBER P128
T. LAMB, McMANUS & ASSOCIATES LTD.

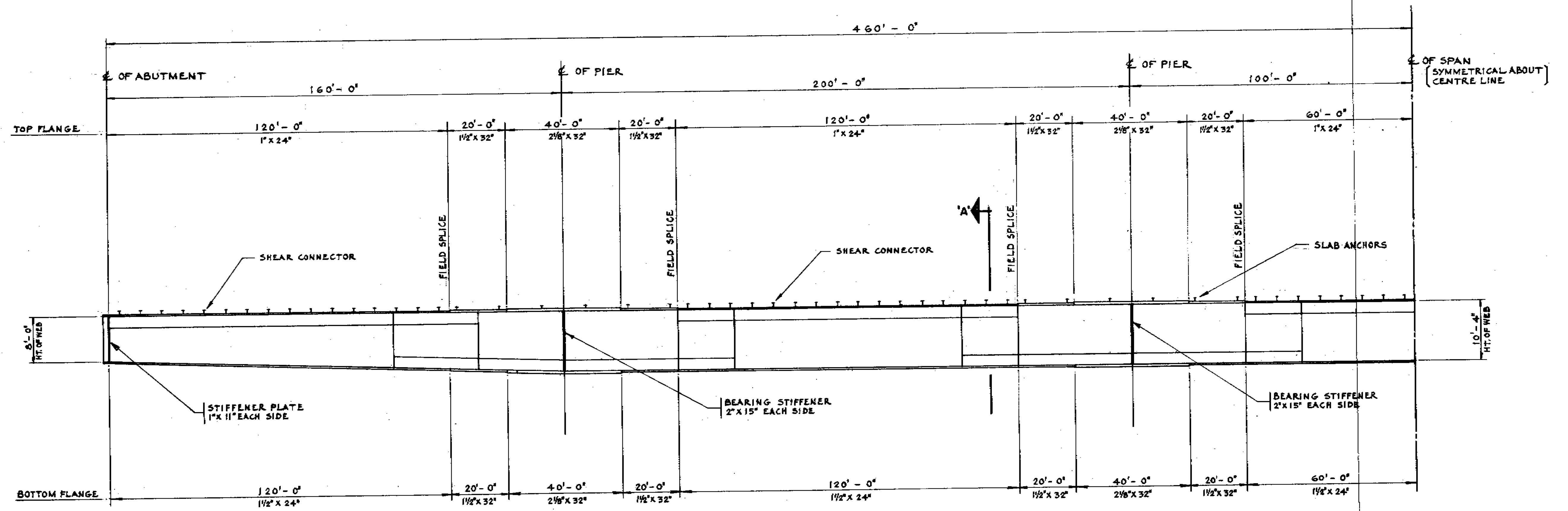
A detail no. / détail no.
B location drawing no. / sur dessin no.
C drawing no. / dessin no.

revisions / date

project title / titre du projet
MACKENZIE HIGHWAY
WILLOWLAKE RIVER BRIDGE
MILE 395·0

drawing title / titre du dessin
TYPICAL PIER DETAILS
ABUTMENT AND DECK DETAILS

designed by R. M. MORISON conçu par
date DECEMBER 5, 1973
drawn by C. CHOU dessiné par
date DECEMBER 5, 1973
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. de projet
85032
drawing no. dessin no.
3



ELEVATION OF HALF OF PLATE GIRDER (OUTSIDE FACE)

SCALE, 1" = 20'-0"

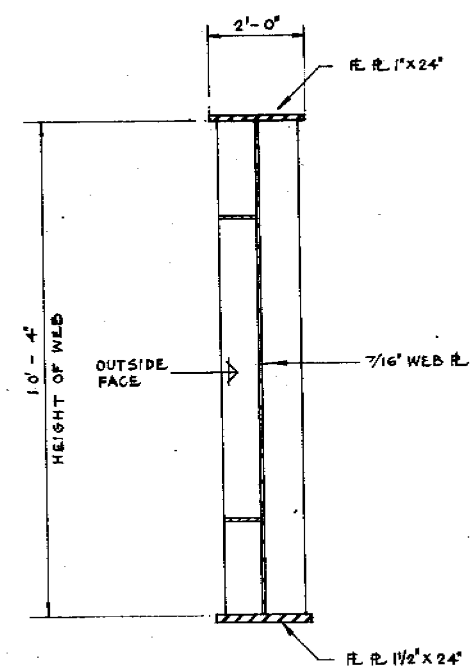
NOTE

FIELD SPLICE TO BE MADE USING HIGH TENSILE BOLTS. NO FIELD WELDING WILL BE PERMITTED.

NOTES

GIRDER IS SYMMETRICAL ABOUT CENTRE LINE.

INTERMEDIATE STIFFENERS ON INSIDE FACE OF GIRDER, NOT SHOWN.



SECTION 'A'

SCALE, ½" = 1'-0"

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Ministère des Travaux publics

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CONSULTING ENGINEERS
EDMONTON CALGARY WINNIPEG

PROFESSIONAL ENGINEER
R. M. MORISON
ALBERTA

THE ASSOCIATION OF PROFESSIONAL ENGINEERS OF ALBERTA
PERMIT NUMBER
P128
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B location drawing no. sur dessin no.
C drawing no. dessin no.

revisions date

project title titre du projet

MACKENZIE HIGHWAY
WILLOWLAKE RIVER BRIDGE
MILE 395·0

drawing title titre du dessin

PLATE GIRDER

designed by R.M. MORISON conçu par
date DECEMBER 5, 1973
drawn by C. CHOU dessiné par
date DECEMBER 5, 1973
reviewed by examiné par
date
approved by approuvé par
date
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D.P.W. Project Manager Administrateur de projets M.T.P.
project number no. du projet
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drawing no. dessin no.
4