GEOTECHNICAL INVESTIGATION

PROPOSED BRIDGE SITE

HELAVA CREEK, MILE 616.4

MACKENZIE HIGHWAY

E-2510

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OCTOBER 16, 1973

R.M.HARDY & ASSOCIATES LTD.

CONSULTING ENGINEERING & TESTING ● GEOTECHNICAL DIVISION

File No. E-2510

October 16, 1973

Mr. F. E. Kimball, P.Eng.,
Manager of Northern Roads Program,
Department of Public Works of Canada,
One Thornton Court,
Edmonton, Alberta.

RE: Geotechnical Investigation
Mackenzie Highway, Proposed Bridge Site
Helava Creek, Mile 616.4

Dear Mr. Kimball:

We are pleased to submit a report on the site of the proposed bridge across Helava Creek. As you are aware, the location of the bridge site has been changed subsequent to our drilling program being completed. The revised bridge site is approximately 300 feet upstream of the original site.

Should you wish for any explanation or amplification of any part of this report we will be pleased to be at your service.

Respectfully submitted,

R. M. HARDY & ASSOCIATES LTD.,

Per:

G. McCormick, P.Eng.

GM/jc

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CONSOLIDATING THE SERVICES OF: R. M. HARDY & ASSOCIATES LTD., MATERIALS TESTING LABORATORIES LTD. & NON DESTRUCTIVE INSPECTION LTD.
BURNABY CALGARY DAWSON CREEK EDMONTON LETHBRIDGE PRINCE GEORGE RED DEER WINNIPEG

INTRODUCTION

At the request of Mr. F. E. Kimball, P.Eng., Manager of Northern Roads Program, Department of Public Works of Canada, Western Region, R. M. Hardy & Associates Ltd. undertook a geotechnical investigation along part of the proposed location of the Mackenzie Highway. This report deals only with that part of the investigation appertaining to the proposed bridge at Helava Creek.

The location of this bridge site is shown on mosaic sheet No. 49 of a set of mosaics prepared by the Department of Public Works for the Mackenzie Highway work. The site is covered by aerial photographs No. A22773-239 and 240 (scale 1" = 1000'). The present proposed crossing is located about 300 feet upstream of the original crossing which was the subject of the investigation carried out as part of our drilling program. In addition to the mosaics and aerial photographs, R. M. Hardy & Associates Ltd. was provided with sketch plans and profiles showing the revised crossing. These drawings are entitled "Plan and Profile Showing Proposed Drainage Structure at Helava Creek, Station 1818+85" and "Revision, Helava Creek" and were used as the basis for Plate 1, Appendix A.

A report entitled "Geotechnical Investigations, Mackenzie Highway, Mile 544 to 635" has been previously submitted to the Department. The geotechnical conditions are discussed in Volume I while Volume II contains information on permafrost of a more general nature. We recommend that these volumes be read in conjunction with this report.

TOPOGRAPHY

The general direction of the drainage in the area is southwesterly towards the Mackenzie River. The valley walls of Helava Creek are relatively steep but the vertical height from water level to surrounding ground is only about 13 feet. On the southerly approach, the ground rises from the creek as the alignment crosses an alluvial fan deposit. On the northerly approach, the ground is fairly level for about 500 feet. The existing profile is such that approaches to the bridge can be constructed without the necessity for cutting while maintaining a gradient of less than 5%.

The valley walls of Helava Creek show signs of erosion but it is not anticipated that the erosion will affect the bridge structure. The width of Helava Creek at the water line is approximately 30 feet.

SOIL PROFILE

The soil profile at the revised crossing is similar to the soil profile at the original crossing on the northerly side. However, on the southerly approach the new alignment crosses an alluvial fan deposit instead of the glacial lake basin at the original location. Alluvial fan deposits consist of poorly sorted silt and sand, usually wet or ice-rich, and are commonly less than 15 feet thick.

The changed soil conditions are not believed to be of any signficance regarding the bridge structure. However, we do recommend against a cut through the alluvial fan deposit on the southerly approach.

The soils on the northerly approach consist of glacial lake basin deposits overlying basal till. The glacial lake basin deposits consist of silts and clays which generally contain high excess ice content. Surface cover is peat which generally varies in thickness from one foot to several feet.

The excess ice in the soils and the approaches to the bridge site will lead to settlements of the approach fills but such settlements will not be serious.

Because our nearest test holes to the revised site are several hundred feet away, we are basing our recommendations on our general knowledge of the area and examination of aerial photographs. The Department's staff reports that the creek bed consists of rock and gravel.

DISCUSSION AND RECOMMENDATIONS

The effect of a stream on the permafrost profile is shown on Plate 2, Appendix A. This chart shows that the thaw bulb beneath a small creek can penetrate to considerable depth so that, for bridge building purposes, the presence of permafrost beneath the stream bed can be ignored. However, it should be noted that the permafrost profile beneath the sides of the stream bed plunges at an extremely steep angle. As is well known, the flow of water in northern streams varies tremendously throughout the year. Very large flows can be experienced during the spring runoff so that some scour can be expected. The amount will depend on the flow of water, the constriction imposed on the stream by the bridge, and the width of the piers. Some erosion of the bank is also possible.

Because of the soil and permafrost conditions in the valley walls and the approach area to this bridge site, we do not believe it would be advisable to use concrete abutments. We therefore recommend that bridge abutments and piers be supported on driven steel H piles. It is extremely unlikely that timber piles could be driven at this site. Precast concrete piles should not be used due to difficulties with transportation and also because the length of the precast piles will have to be determined in advance. Steel pipe piles are an alternative possibility. However, it is probable that they would not be able to withstand driving stresses through the underlying gravel.

Steel H piles which are to be placed on the banks where they will not be affected by scour should be driven a minimum of 30 feet below existing grade and designed on the basis of an allowable skin friction of 400 psf (on the gross perimeter) with the top 10 feet of pile being assumed to carry no load.

Steel H piles driven in the stream bed should be driven a minimum distance of 20 feet below the bottom of anticipated scour and should be designed on the basis of the "Table of Penetration Resistance" following. Design parameters are summarized on Plate 3, Appendix A.

Driving steel H piles will require considerable energy. The weight of the pile driving hammer should be at least twice the weight of the pile being driven. If a diesel hammer is used the weight of the hammer should be at least equal to the weight of the pile. To prevent damage to the points of the piles they should be reinforced with flange plates for a distance equal to 1.5 times the size of the pile. Alternatively, the point can be reinforced with a driving shoe. Piles should be driven to practical refusal or refusal according to the following table of penetration resistances assuming that the hammer delivers an energy of 15,000 ft. pounds per blow.

TABLE OF PENETRATION RESISTANCE

Description	Inches Per Blow
refusal	.00-.05
practical refusal	.05-.25
high resistance	.25-.50
medium resistance	.50-1.25

In order to ensure that refusal has been reached, driving should be continued for at least 100 blows after refusal is first recorded.

Piles driven to refusal in the stream bed, as defined above, may be designed for the full structural strength of the pile section acting as a column. The design load will depend upon allowable stresses in the pile, column length and the arrangement of lateral bracing. Piles driven to practical refusal, as defined above, should be designed for two-thirds of the value permitted for the pile as a structural column. Consideration should be given to using battered piles on the outside of the pile bents in order to provide lateral resistance.

If a drop hammer is used in driving the piles, care should be taken that the energy delivered to the pile is not greater than 15,000 ft. pounds per blow unless calculations show that the pile can safety take higher impact stresses. Bedrock may be encountered before the design depth is reached. In such a case, the piles should be driven into the bedrock at least 10 feet.

One of the problems facing bridges in this area is the possibility of log jams occurring which can cause partial or complete failure of the bridge. Log jams are only likely to occur where trees travelling down the river have a greater length than the clear span of the bridge. We suggest that the height of trees growing adjacent to Helava Creek upstream of the bridge should be checked and, should it be observed that there is a possiblity of large trees being washed downstream, such facts should be borne in mind by the bridge designer. If piles are used to support a vertical face of embankment fill the lateral force against the piles can be computed by assuming that the backfill is a fluid with a density of 60 pounds per cubic foot where the backfill is not compacted.

Embankment constructed below the highest expected flood level should be protected with riprap. As suitable rock may not be available, sandbags filled with concrete may have to be used.

Respectfully submitted,

R. M. HARDY & ASSOCIATES LTD.,

Per:

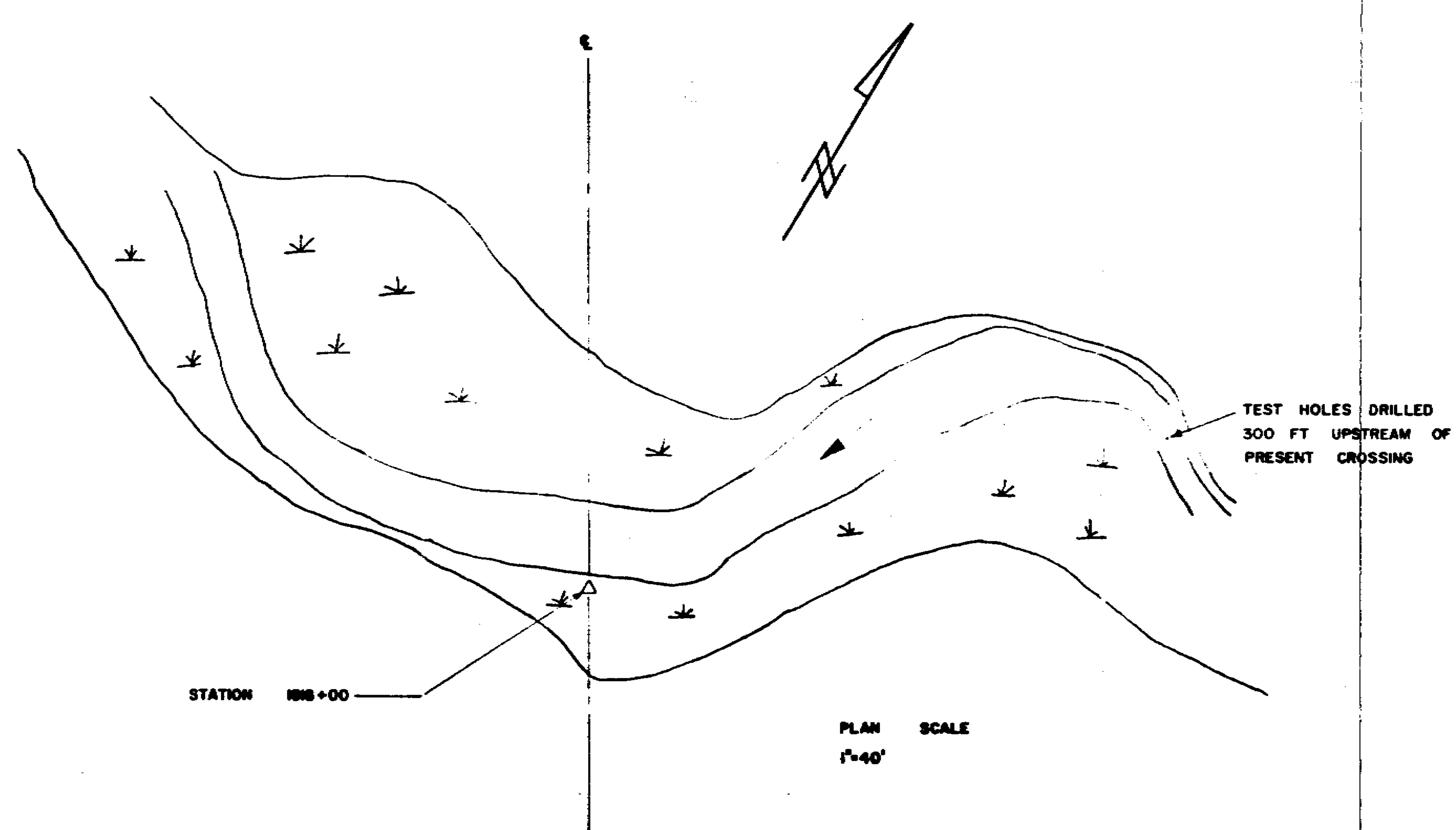
G. McCormick, P.Eng.

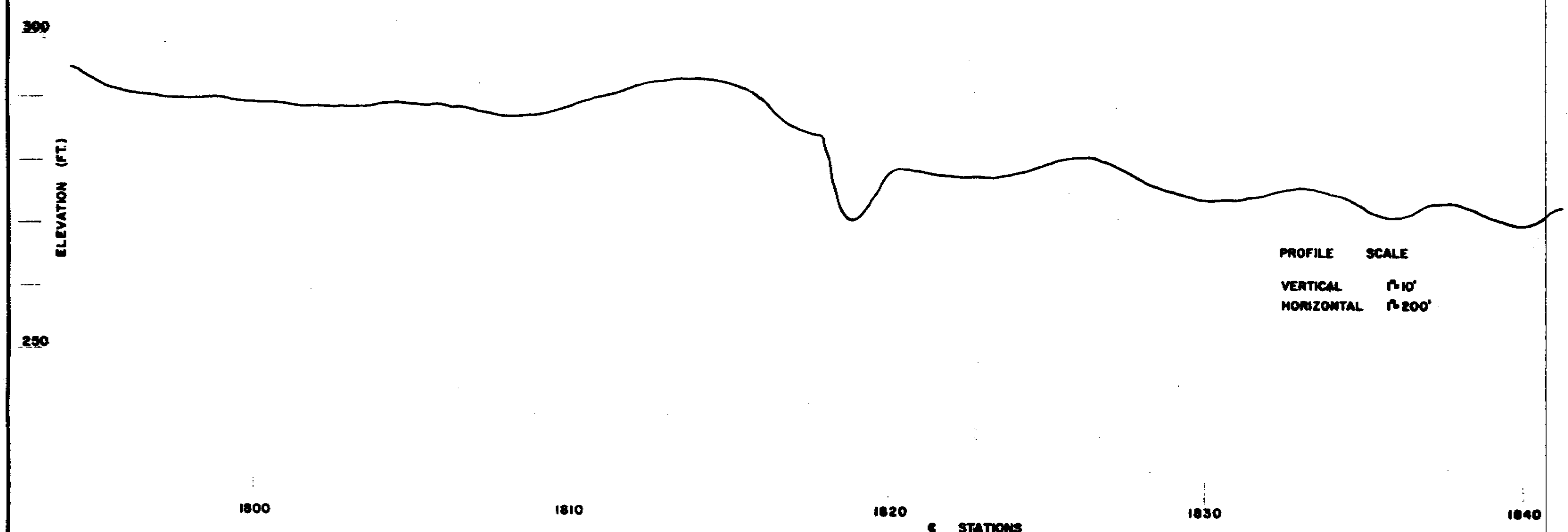
GM/jc

THE ASSOCIATION OF PROFESSIONAL ENGINEERS OF ALBERTA
PERMIT NUMBER
P [illegible]
R. M. HARDY AND ASSOCIATES LTD.

APPENDIX A

Section
Chart





NOTE

THIS DRAWING HAS BEEN REDUCED TO 50 % SIZE

No.	REVISION	DATE	BY

D.P.W. DWG "PROPOSED DRAINAGE STRUCTURE AT HELAVA CREEK"

REFERENCES

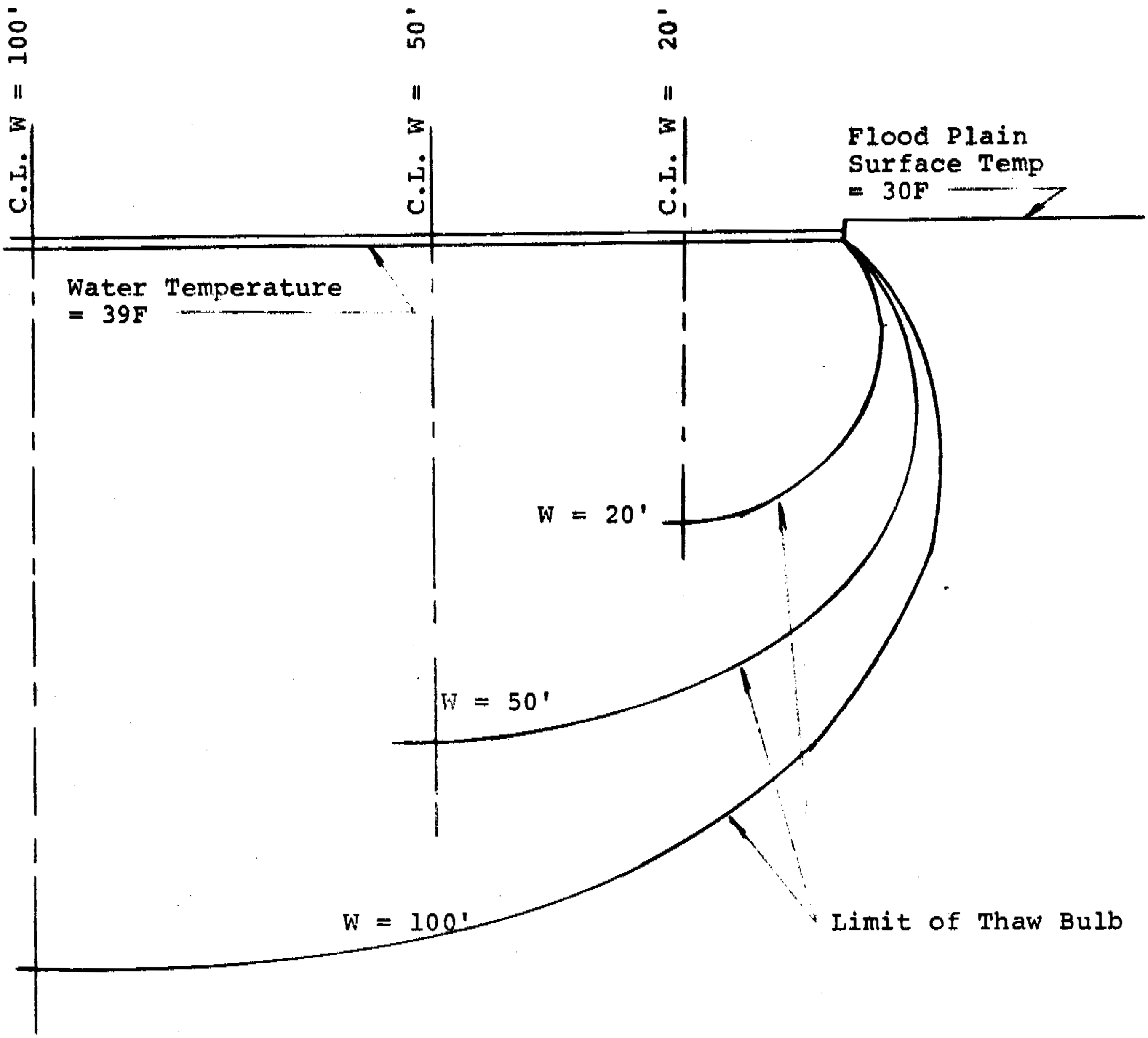
R.M. HARDY & ASSOCIATES LTD.
CONSULTING ENGINEERING & TESTING

DEPARTMENT OF PUBLIC WORKS
MAKENZIE HIGHWAY
HELAVA CREEK

SCALE SHOWN DATE OCT. 9 '73

No. E 2510-102 REV. 0

PLATE I



Scale: 1" = 10'

W = River Width
C.L. = Center Line

G.Mc September 14/73 E-2510

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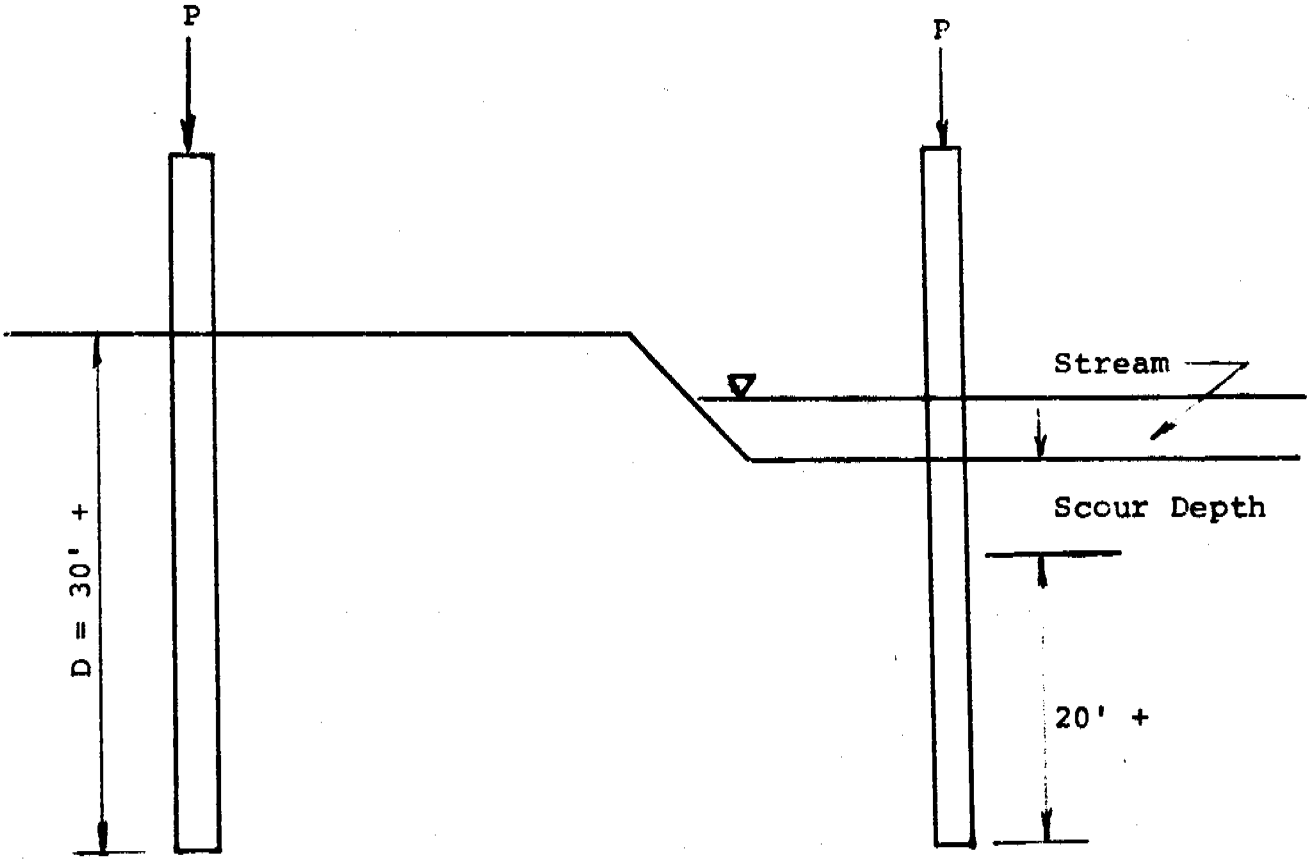
THAW BULBS BENEATH RIVERS
NORMAN WELLS AREA



$$\text{Gross Perimeter} = \frac{4H}{12} = \frac{H}{3} \text{ ft.}$$

Piles on dry land to be designed on the basis of an allowable shaft friction over effective length of embedment of D-10 with D minimum = 30 ft.

Piles in stream bed to be driven to 20+ feet below scour depth and designed on the basis of penetration values (see text).

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MACKENZIE HIGHWAY
BRIDGE PILES
NORMAN WELLS AREA

SCALE	DATE	MADE G.Mc	CHKD.	JOB: E2510	PLATE