

GEOCRES No. 41J-32

DIST. 18 REGION

W.P. No. 931-73-00

CONT. No.

W. O. No.

STR. SITE No. N/A

HWY. No. 17

LOCATION BLIND RIVER - FEASIBILITY
STUDY

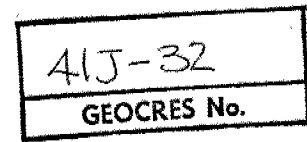
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SOIL MECHANICS SECTION
GEOTECHNICAL OFFICE ENGINEERING SERVICES BRANCH



FOUNDATION INVESTIGATION REPORT

W.P. 931-73-00 SITE NO. N/A

DIST. NO. 18 HWY. 17N

BLIND RIVER DIVERSION

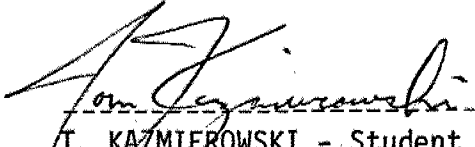
Route Planning Study


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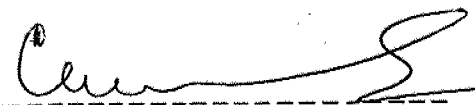
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ABSTRACT

Information relative to subsurface conditions was required to aid in the evaluation of various alternate route proposals for a by-pass around Blind River.

Geological studies aided by borehole exploration techniques indicate that it is feasible to construct a causeway within the North Channel to the grades proposed. The geotechnical requirements to ensure a stable causeway and the anticipated performance characteristics were reported in a memorandum dated July 25, 1975. This report concerns itself with the investigational processes and results obtained therefrom.

The report is intended to provide a framework within which to plan and execute further geotechnical studies of a more detailed nature when and as required.

FOUNDATION INVESTIGATION REPORT

W.P. 931-73-00

BLIND RIVER DIVERSION - Hwy. 17

Route Planning Study

1. INTRODUCTION

1.1 General

A Route Planning project is underway to study the feasibility of diverting Hwy. 17 around the town of Blind River and providing for better operational characteristics on the by-pass route. Several study route alternatives have been developed but the evaluation of all of the alternatives is not yet complete, particularly for routes designated C-1, C-3 and F (see plan, Drawing No. 9317300-A).

A terrain evaluation has been carried out for the overland portions of these routes using air photo interpretation techniques. However, information on subsurface conditions is lacking at those locations where the routes cross water bodies.

1.2 Terms of Reference

This Section was requested in March, 1975 to provide data relative to subsurface conditions for route C-3--the "Causeway" alignment across

the North Channel. A geological study was undertaken as part of a geotechnical study package. Due to ice-thickness deficiency, the borehole exploration programme was deferred from March to late May, 1975, at which time some exploratory boreholes were drilled from a raft rigging (see Photo 1).

The "Causeway" constitutes a major percentage of the total length of Route C-3 and therefore subsurface conditions are critical for cost-evaluation purposes. The subsurface conditions at the crossing of Route C-1 and Blind River, however are not critical for cost evaluation purposes since the cost of structures at this crossing location will likely constitute a minor percentage of the total route cost. Therefore, the decision was made to not investigate the Route C-3 water crossing by exploratory boreholes at this stage of the planning study.

For the Route F and Lake of the Mountains water crossing, Ontario Hydro were contacted for information relative to subsurface conditions encountered by them for the construction of the power transmission line located in the vicinity of the proposed study route. The information received indicated no problematic subsurface conditions, insofar as the scope and purpose of this planning study are concerned. Therefore, exploratory boreholes were not drilled at this crossing location.

At both the Route C-1 and Route F water crossing locations, soundings and shallow probings of bottom materials were requested of and carried out by the Regional Engineering Surveys personnel, Northwestern Region.

2. SITE DESCRIPTIONS

2.1 Site - General

The study area is located in and around the town of Blind River in Cobden and Striker Townships, District of Algoma.

The area is effectively drained by Blind River which enters Lake Huron via two outlets as a result of damming. The two dams are operated and maintained by the Ministry of Natural Resources. One of the dams is located in the town area at the mouth of Blind River where the major flow occurs. This is a control dam, and was reconstructed during 1973-1974. The other dam is located about 2 miles to the west of the dam just described. This structure is a plug dam, and its minor discharge empties into the so called west arm of Blind River. A 3 feet diameter culvert passing through the dam maintains some flow downstream in order to help improve water quality. No major siltation problems have occurred at these dam sites.

The following descriptions are made based on ground reconnaissance and are intended to supplement terrain evaluations already carried out by air photo interpretation techniques.

2.2 Site - Route C-3

The proposed causeway route (Route C-3 Sta 258+00± to Sta 320+00±) crosses the North Channel. The east end of this causeway is situated on Caribou Point which is a heavily vegetated low lying area with large boulders scattered at the surface. From Caribou Point, the causeway route cuts across the Channel and passes through Susanne Island (rock outcrop - see picture 2) and continues to Comb Point (rock outcrop and medium to dense vegetation).

2.3 Site - Route C-1

Route C-1 crosses Blind River (Sta 120+00± to Sta 148+00±) and cuts through two islands (from Sta 122+00± to Sta 134+50± and from

Sta 137+00± to Sta 145+00±). On the west island, bedrock outcrops occur on the northern and eastern portions, while on the east island, bedrock outcrops occur mainly on the western side and to some extent on the northern side. The remaining portions of the islands are covered with vegetation. Tree stumps and sunken logs are visible in the surrounding waters.

Easterly from the Blind River crossing towards its junction with Hwy. 557, Route C-1 enters a low lying and somewhat marshy area. Several hundred feet to the north of this junction, glacial material composed of sand interbedded with gravel is exposed in shallow cut slopes of Hwy. 557.

Further north on Hwy. 557, the ground surface observed is predominantly rolling terrain, heavily wooded, with scattered cottages and cabins. Glaciofluvial sand and gravels graded with boulders and cobbles are the main soil types. Outcrops of bedrock are also visible at places.

2.4 Site - Route F

The site where Route F crosses Lake of the Mountains (Sta 130+00 to Sta 146+00) is located just north of the northwest-southeast trending Ontario Hydro Transmission Line (see Photo 3). The west shore of the Lake in this vicinity is a flat lying, heavily vegetated area, and is marshy along the edges. Large rock outcrops, boulders and granular types of material are visible on the east bank.

3. GEOLOGICAL ASSESSMENT

3.1 General

The following geological assessment of Routes C-3, C-1 and F is based on visual observations and background information provided in the Ontario Department of Mines Geological Report, Number 20.

Geologically, the study area is located in the Precambrian Shield region of crystalline rocks which are over 1 billion years old. The area has been deeply buried in the earth's crust and the rocks have been highly metamorphosed.

Structurally, the area is part of the southern anticlinal limb of the Blind River reverse - S fold which strikes just south of east.

3.2 Faults

Major faults pertinent to this study are:

3.2.1 Murray fault: This is a reverse fault striking in an easterly direction and dipping almost vertically to the south. It runs relatively parallel to Route C-1, approximately 1,000 to 2,000 ft. to the south with a minor N-E striking fault intersecting Route C-1 at approximately Station 75.

3.2.2 Lake of the Mountains fault: This fault strikes northeast and dips almost vertically to the northwest, intersects Route 'F' at approximately Station 110.

There is also a minor fault running parallel to Route C-3 approx. 1000 ft. offshore to the south between Stations 180 and 270.

3.3 Lithology

3.3.1 Route C-3 (Sta 250 to Sta 340): At Comb Point one can observe massive outcrops of quartzite and feldspar-rich quartzite which strike approximately to the east and dip at about 70° to the south. These rocks have frequent joints and fractures, both parallel and perpendicular to the strike; cobbles and boulders are visible throughout. (Possibly as a result of proximity to minor offshore faults). Glacial striations trending in a northeasterly direction can be observed.

Susanne Island (Station 283 \pm) consists of the same quartzite material with intersecting diabase dykes running parallel to strike.

Caribou Point has no visible outcrops, however, the assumed bedrock in the area below the overburden is competent greywacke and argillaceous quartzite.

3.3.2 Route C-1 (Sta 120 to Sta 150): Geologically, this section of the route is situated on structurally competent intrusive quartz diabase dyke as illustrated by massive outcrops on the eastern side of the west island and the western side of the east island within Blind River. These dykes strike east, paralleling the Murray Fault, and dip vertically, with theory suggesting that they have intruded pre-existing fractures. Also visible is large diabase outcrop stretching from the river's east bank.

East of Station 150, Route C-1 leaves the diabase and stretches across sparse boulder greywacke conglomerates of the Gowganda Formation which is the youngest sedimentary formation exposed in the study area. Outcrops of this rock are visible along both Highways 557 and 555.

The boulders and cobbles are rounded consisting predominately of granite, gneiss, and diabase within a fine to medium grained greywacke

matrix. They strike in an east-northeasterly direction. Parts of the northerly ends of the two island also have this type of rock outcrop.

3.3.3 Route F: The pleistocene geology of the area consists of glacial sands and gravels, well-bedded, often showing gradation from boulders and cobbles at the surface down to gravels and sand.

Large massive outcrops of the Gowganda formation with steep faces occur to the north and south of the line between Stations 100 to 130.

On the southern side of Lake of the Mountains where Route F crosses the lake there is an outcrop of the boulder greywacke conglomerate interbedded with pink to reddish coloured feldspathic quartzite. These outcrops strike in a south-easterly direction and dip at a shallow angle to the southwest. They are characterized by jointing parallel and perpendicular to the strike. The joints are nearly vertical in attitude.

4. SUBSURFACE EXPLORATION - ROUTE C-3

4.1 General

Geological and other publications were found lacking in details regarding subsurface soil conditions below the lake bed within the North Channel. It is known, however, that there has been no dredging in the harbour channel in the last ten years or so. Prior to about 1969 navigation requirements called for a 15 ft. depth of dredged channel. Personal communications have indicated the dredged materials to be silts and fine sands. This information combined with the infrequency of dredging when shipping was active in Blind River indicated a high probability of stable lake bed material.

4.2 Test Borings

4.2.1 Mobilization: Four test boreholes were put down from a 12' X 16' raft constructed out of size 200-gallon capacity oil drums. The raft and a B.B.S. #1 drilling unit were supplied by the Canadian Longyear Co. Ltd. (see Photo 1, Appendix II).

Drilling was carried out between May 26 and 31, 1975. In order to avoid high waves and the possibility of damage and loss of equipment resulting therefrom, it was necessary to capitalize upon periods of calm water. Such calm periods usually occurred in the evenings as well as from a few hours before dawn to just before noon.

Once set up at a location and anchored into position with drag-line anchors, it was essential to complete the borehole and sampling operation before demobilization due to high waves, darkness, etc. Once the raft was moved out of position, it was an impossible task to reposition the raft at its original location. Therefore, the depth of boreholes and frequency of sampling for this study has

been controlled by weather conditions, wave occurrence, and available daylight more so than by geotechnical factors.

If further borehole exploration is needed, it is recommended that such work be scheduled for winter when the North Channel freezes over. Working from an ice surface would result in the most economical form of detailed subsurface exploration. The alternative would be to drill from platforms raised above the prevailing lake water level. Such riggings can be very expensive and are justified only for very critical or very lengthy and detailed investigations.

4.2.2 Sampling: The borehole locations are shown in plan on Drawing No. 9317300-A. These locations were selected prior to the investigation and identified in the field by triangulation techniques. The accuracy of coordinated locations is ± 5 ft.

Sampling was carried out with a standard split-barrel sampler meeting the CSA Specifications for the Standard Penetration Test. Cohesive strata were sampled by 2" O.D. thin-walled tubes pushed in manually. Each thin-walled sample taken was followed by an in-situ vane test. Refer to the Borelog sheets for details of sampling and in-situ testing.

Laboratory testing was carried out on selected samples and the results are given on the Borelog sheets and Figures in Appendix I.

5. SUBSOIL CONDITIONS

5.1 Route C-3

The description of subsoil conditions given herein is limited to the causeway portion only of Route C-3. It is believed that Regional Materials and Testing will provide information relative to muck depths and underlying materials in the swampy area between Sta. ± 120 to Sta. ± 160 .

Underlying an average water depth of about 10 ft. is a thin mantle of lake bottom organic material of recent origin. This material consists of bark, sawdust, woodchips and sunken logs, originating from lumber mill operations at the old J.J. McFadden Mill.

The predominant soil along the causeway corridor is non-cohesive in character and comprises compact to dense silts, sands and gravels, variously bedded and intermixed. These deposits contain cobbles and occasional boulders, as evidenced by erratically high 'N' values at certain locations. The bearing capacity of the soil is considered to be adequate to support the proposed causeway embankment loadings.

Near the government wharf, far removed from the causeway alignment, a cohesive stratum of soft to firm, grey clayey silt was encountered about 10 ft. below the lake bottom. This stratum is probably sandwiched between non-cohesive, competent soil strata.

The fact that this soft cohesive stratum was not encountered at the site of the existing Hwy. 17 structure across Blind River within the town (refer Cont. 58-35, see Photo 4) nor along the causeway alignment leads to the conclusion that this stratum is either (i) non-continuous or (ii) dips at an angle of at least $3\frac{1}{2}^{\circ}$ towards the south into the North Channel from its source somewhere between the existing Hwy. 17 bridge and the government wharf.

No serious foundation problems are anticipated if (i) is the case. Some long-term but minor settlement problems can be anticipated along the proposed Route C-3 alignment if (ii) is the case.

If the clay deposit is a wedge-shaped stratum, originating near the town and dipping away and down at an angle of 3-1/2 degrees or better and at the same time increasing in thickness, the effect it might likely have on the behaviour of a causeway would be more or less independent of the actual causeway alignment due to compensating features.

For example, if the alignment is shifted north of the existing proposed Route C-3, the clay layer might be located closer to the surface but it would also be thinner due to its wedge-shape. If the alignment is shifted to the south, the clay stratum might become thicker but it would also be located at greater depth so that the stresses created by a causeway loading would be diminished in magnitude before reaching the clay.

Based on the above reasoning, it is concluded that the evaluation of the causeway scheme should proceed on the assumption of competent lake bottom deposits along the presently proposed C-3 route.

5.2 Routes C-1 and F

For reasons outlined previously under 1.2, no borehole exploration was carried out along these two routes. However, soundings and probings were made and the results are shown on Drawing No. 9317300-B.

Both Routes C-1 and F will possibly encounter clayey soils, in certain areas, at varying depths. At the Dawsey Creek structure site along Sec. Hwy. 555, the subsoil consisted mainly of loose silt with a thin seam of clayey soil at depth. At the Blind River North Bridge site along Hwy. 557 (Cont. 73-84) a confined wedge-shaped stratum of soft layered silty clay was encountered.

It appears from data on hand that quite variable subsoil conditions can be expected along these and other routes.

For comparative evaluation, the risk of encountering very poor subsoil conditions locally or along extensive reaches of any alignment are all about the same. Hence, detailed subsurface exploration is not justified until after selection of a route based on evaluation criteria other than foundation characteristics.

6. DESIGN CONSIDERATIONS

6.1 General

Since the aim of the planning study is to carry out evaluations of various proposed routes, only those foundation characteristics need be discussed which will influence the evaluation process. For the overland portions of Routes C-1 and F, it was pointed out in 5.2 that foundation conditions will probably not be too different on the average, so that settlement and stability problems are likely to be common to both alignments. The greater influence will be that of grade setting and balancing quantities as well as minimizing expensive excavations, subexcavations and fills.

For Route C-3, the Causeway constitutes a major physical, environmental and economic entity and thus has a definite bearing on the evaluation process. Further discussions are therefore confined to the "causeway" concept.

6.2 Causeway

The proposed gradeline is shown on Drawing No. 9317300-B. It has been set to satisfy hydrology requirements as well as geometric design criteria. Navigation clearance requirements (only tentatively known at the present time) necessitate fills 70-80 ft. in height above the lake bottom.

Recommendations pertaining to the materials design aspects of the causeway for cost evaluation purposes have already been made (see memorandum C. Mirza to W.L. Lees of July 25, 1975) and will not be repeated here.

The following comments are offered as an aid to the evaluation process:

- (a) The subsurface conditions along Route C-3, as assessed for this study, indicate no preferential sites or areas for high fills and any associated high-level structures.
- (b) The proposed gradeline westerly from Comb Point should be adjusted (if necessary) to produce sufficient excavation material for use in the causeway fill. This will entail possibly large rock cuts which, if closely confined to minimize property acquisition, could lead to "tunnel" effects environmentally.
- (c) Further detailed rock mapping may be required to estimate overbreak and to assess suitability for submerged rock-fill purposes. Some types of argillaceous rocks and greywackes should be avoided in submerged fill construction--they tend to break down under loading, with time due to stress relief effects as well as due to inherently weak lithological make-up. Selective rock quarrying combined with rock excavation can be a very expensive proposition!
- (d) The causeway opening at the level of the lake bottom should be sufficiently large to prevent undesirable influences on stability as a result of any dredging operations required to keep shipping channels navigable.

6.3 High Level Bridge

Tentative recommendations have already been made in the July 25, 1975 memorandum referred to earlier.

The following additional comments may be helpful during more detailed evaluation studies on Route C-3:

- (a) If the structure is to be supported on piles, pile lengths requirements are likely to be a minimum when the structure is located close to existing rock outcrops.

- (b) Because of the steeply dipping nature of the bedrock, secure anchoring of the piles or caissons will be necessary. This is a costly item.
- (c) Secure anchoring systems may not be required if the structure is located away from rock outcrops. However, greater pile lengths will then be necessary.

6.4 Construction Features

Blasting will be necessary to obtain rock fill from the proposed rock cuts. Some primary crushing may be necessary to keep the void ratio as low as possible, particularly if the rock does not shatter into a well-graded mix.

Because of the southern dip of the rock, the north side of the rock cuts may require minor stabilization by guniting or rock bolting. Protection from rock fall after excavation may be required in the form of wire mesh curtains, etc.

APPENDIX I

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 1

W.P. 931-73-00 LOCATION Co-ords. 16,785,150 N; 1,264,050 E. ORIGINATED BY TK
 DIST. 18 HWY. 17 BORING DATE May 27, 1975 COMPILED BY TK
 DATUM Geodetic BOREHOLE TYPE Diamond Drill - Washboring CHECKED BY *0.4*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N' VALUES		20	40	60	80	100	w_p	w	w_L		
582.3	Raft Deck															
581.0	Water Surface															
1.3						580										
568.7	Lake Bottom					570										
13.6	Wood Chips		1	SS	9											
	Sand, trace of silt.															
	Loose to Very Loose		2	SS	2											
558.3						560										
24.0	Clayey silt, some sand		3	SS	1											
	Soft to Firm															
	Grey		4	TW	PM											
548.3						550										
547.1	Sand & cobble		5	RC	33%											
35.2	End of Borehole															

RECORD OF BOREHOLE No 2

W.P. 931-73-00 LOCATION Co-ords. 16,781,900 N; 1,262,200 E. ORIGINATED BY HS
 DIST. 18 HWY. 17 BORING DATE May 28, 1975 COMPILED BY TK
 DATUM Geodetic BOREHOLE TYPE Diamond Drill-Washboring CHECKED BY o.y.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w			UNIT WEIGHT γ	REMARKS % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
582.0	Raft Deck															
581.0	Water Surface															
1.0						580										
						570										
560.6	Lake Bottom					560										
21.4	wood log															
	Sand, trace of silt		1	SS	15											
	Compact															
	Dense to					550										
	Very Dense		2	SS	61											0 99 1 0
						540										
537.5																
44.5	Silt, some gravel, traces of sand, clay.		3	SS	23											16 7 67 10
533.0	Compact															
49.0	Sand, trace of silt.															
529.0	Very Dense															
53.0	End of Borehole															

ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

RECORD OF BOREHOLE NO 3

W.P. 931-73-00

LOCATION Co-ords. 16,782,900 N; 1,265,350 E.

ORIGINATED BY HS

DIST 18 HWY. 17

BORING DATE May 29, 1975

COMPILED BY TK

DATUM Geodetic

BOREHOLE TYPE Diamond Drill - Washboring

CHECKED BY *o.v.*

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			UNIT WEIGHT γ	REMARKS % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	w_p	w	w_L		
582.2	Raft Deck															
581.0	Water Surface															
1.2						580										
570.2	Lake Bottom					570										
12.0	wood chips															
	Sand, trace of silt.															
	Compact		1	SS	27	560										
	Dense															
			2	SS	40	550										
548.1	Sand, gravel, cobbles		3	SS	100.8"											0 99 1 0
34.1	End of Borehole															

RECORD OF BOREHOLE NO 4

W.P. 931-73-00

LOCATION Co-ords. 16,784,350 N; 1,268,000 E.

ORIGINATED BY TK

DIST. 18 HWY. 17

BORING DATE May 30, 31, 1975

COMPILED BY HS

DATUM Geodetic

BOREHOLE TYPE Diamond Drill - Washboring

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION RESISTANCE PLOT					LIQUID LIMIT — w_L PLASTIC LIMIT — w_p WATER CONTENT — w			UNIT WEIGHT γ	REMARKS % GR. SA. SI. CL.	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100	SHEAR STRENGTH					WATER CONTENT %
												○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					
582.2	Raft Deck																
581.0	Water Surface																
1.2	Lake Bottom					580											
3.0	wood chips																
	Sand, traces of gravel, silt		1	SS	13	570											
			2	SS	10	560										9 88 3 0	
	Compact Gravelly sand, trace of silt. Very Dense		3	SS	131	550										42 54 4 0	
	Dense to Very Dense		4	SS	80	540											
534.2																	
48.0	Sandy silt, trace to some gravel, trace of clay.		5	SS	41	530											
	Dense		6	SS	47	520										18 20 58 4	
			7	WS													
513.2																	
69.0	End of Borehole																

GRAIN SIZE DISTRIBUTION

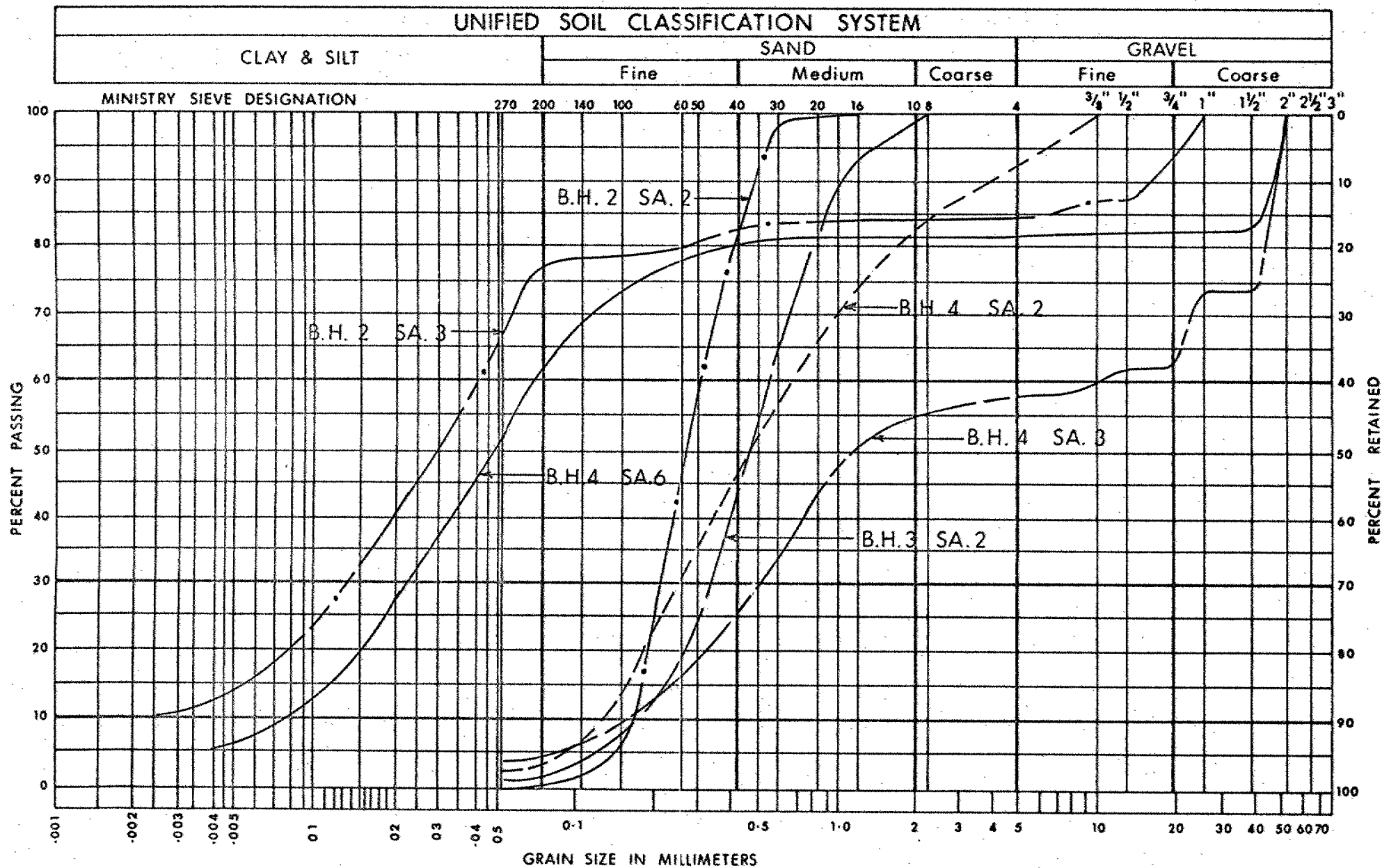


FIG. 1

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

'N'-STANDARD PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>c LB./SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 250	VERY LOOSE	0 - 4
SOFT	250 - 500	LOOSE	4 - 10
FIRM	500 - 1000	COMPACT	10 - 30
STIFF	1000 - 2000	DENSE	30 - 50
VERY STIFF	2000 - 4000	VERY DENSE	> 50
HARD	> 4000		

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.T.	SLOTTED TUBE SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_p	PLASTIC LIMIT
I_p	PLASTICITY INDEX
w_s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$
I_C	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE = $\frac{-\Delta e}{(1+e)\Delta\sigma}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX = $\frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR = $\frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

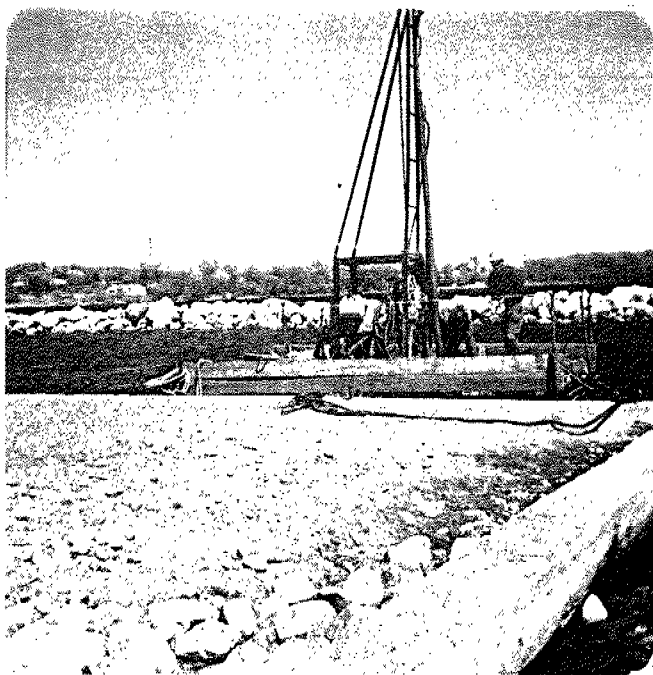
B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

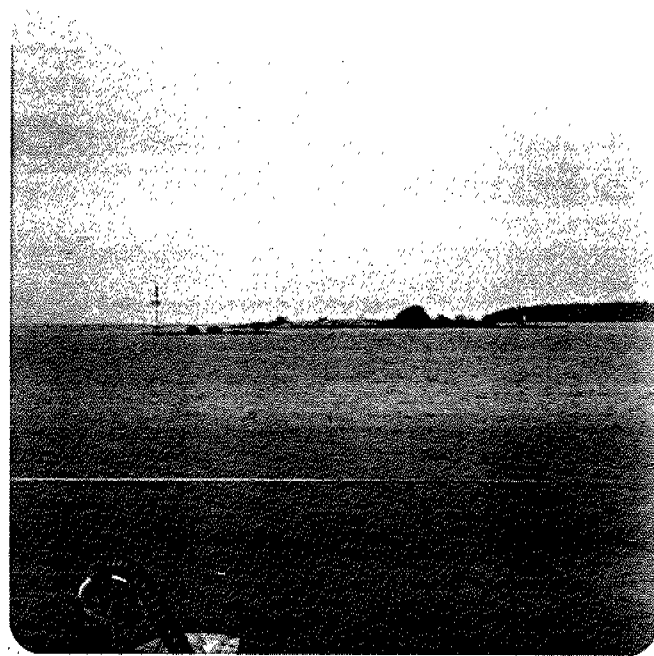
H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

APPENDIX II

PHOTOGRAPHS



1. Raft Rigging used in Borehole Exploration. Raft is positioned at Location of B.H. 1 near Government Wharf.



2. View along proposed Route C-3 "Causeway" alignment. Susanne Island and Comb Point in background.



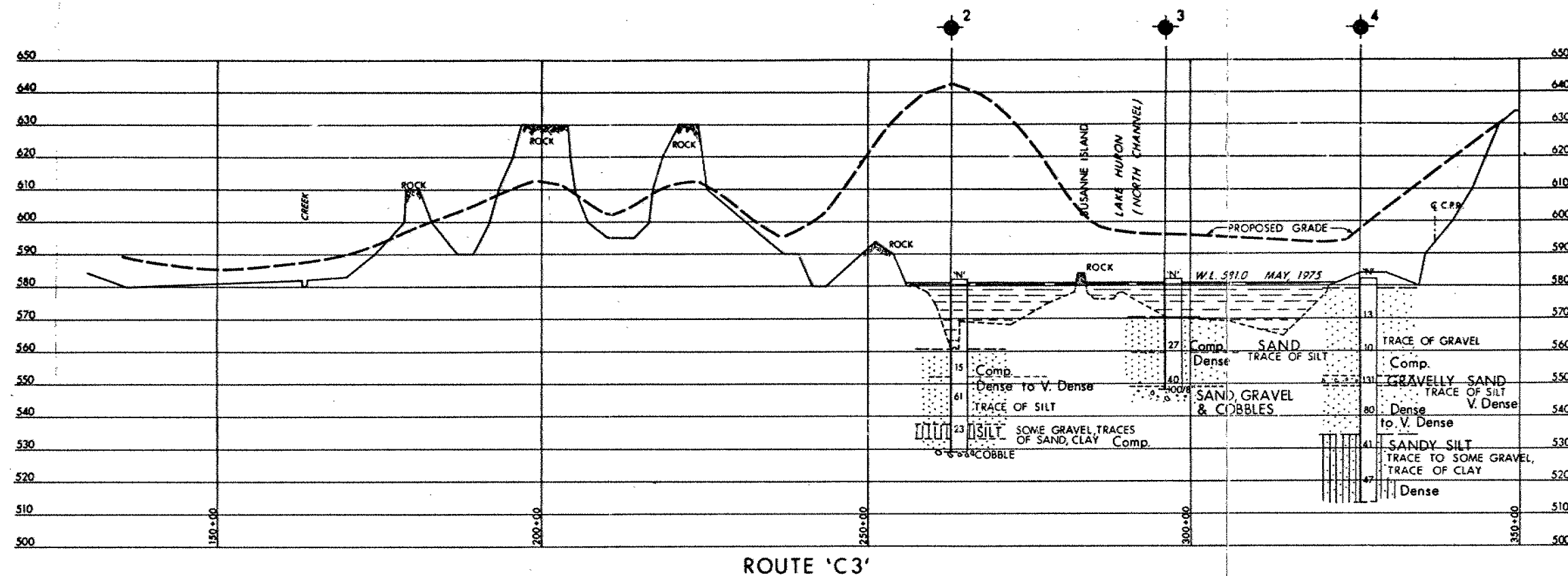
3. View of Route F Crossing of Lake of the Mountains. Note Hydro tower in background.



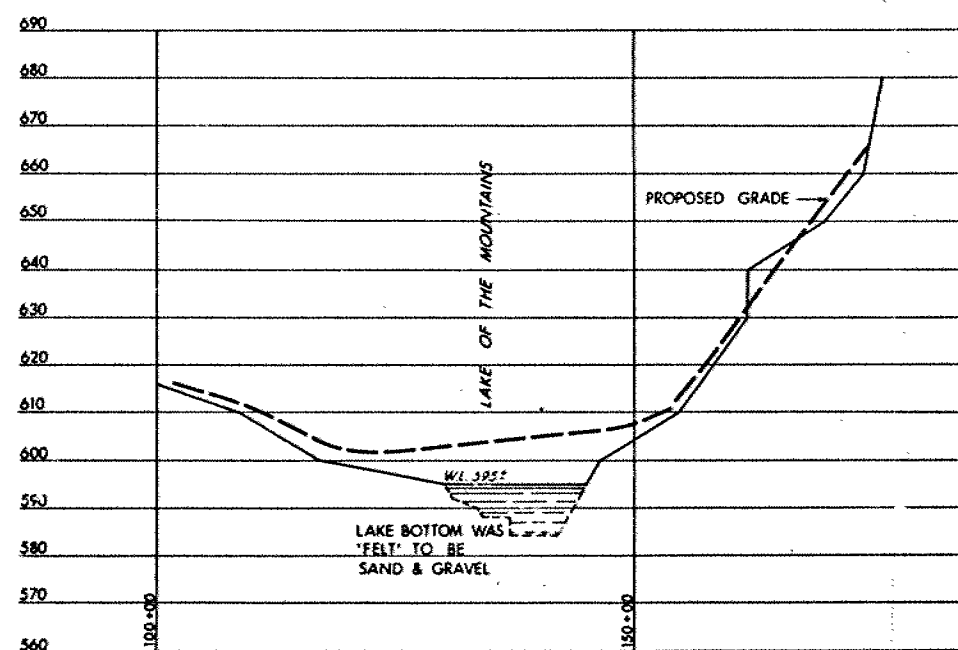
4. Existing Hwy. 17 Bridge in the Town of Blind River over Blind River

DRAWINGS

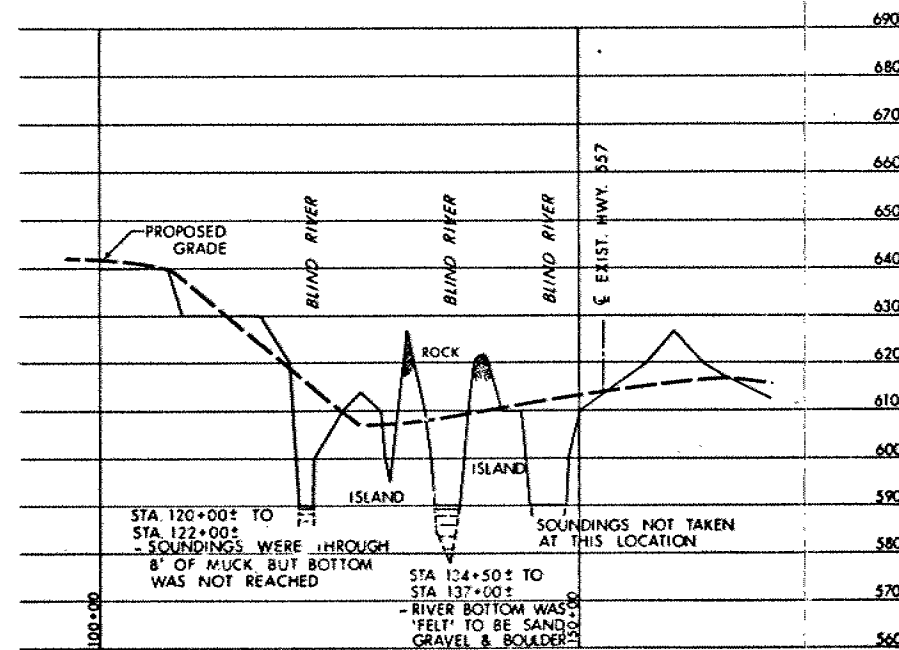
OVERSIZE DRAWING



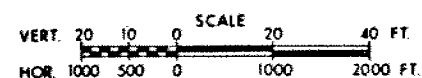
ROUTE 'C3'



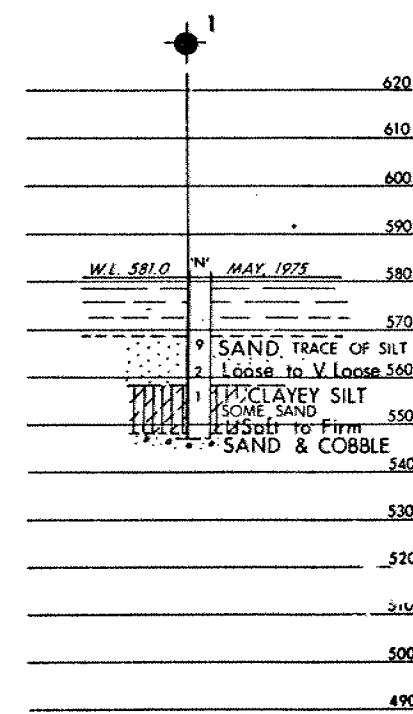
ROUTE 'F'



ROUTE 'C1'



NOTE:
SOUNDINGS BY PROBING FOR ROUTES
'C1' & 'F' TAKEN BY ENGINEERING
SURVEYS, NORTHWESTERN REGION



B.H. 1

NOTE:
B.H. 1 WAS PUT DOWN NEAR THE
SOUTH END OF GOVERNMENT WHARF.
IT WAS PUT DOWN AS A CONTROL
POINT FOR A POSSIBLE FUTURE
GEOPHYSICAL SURVEY

SEE DWG. 9317300-A

KEY PLAN

LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Resistance Test
B/F CONE - Blows/Ft. Cone Test (350 ft. lbs. energy/blow)
- ⊕ Bore Hole & Cone Test
- W Water Levels established at time of field investigation.

NO.	ELEVATION		

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

FEASIBILITY STUDY

BLIND RIVER AREA BYPASS

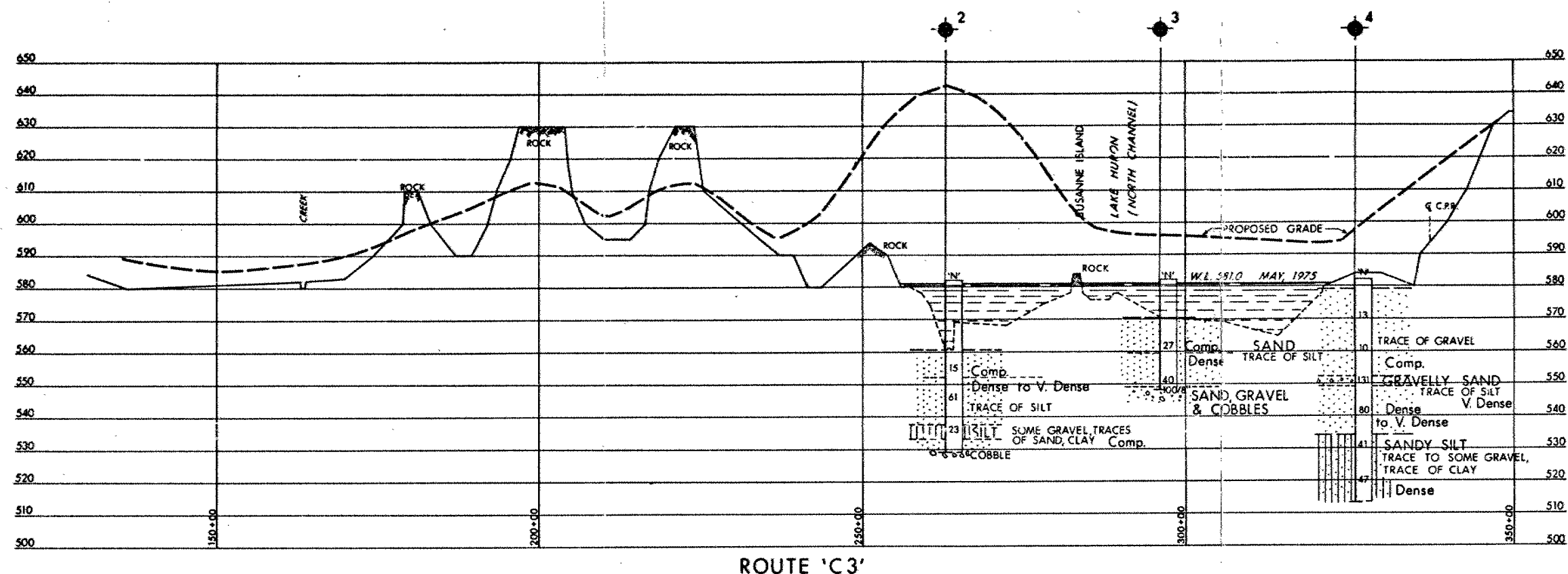
HIGHWAY NO. 17 DIST. NO. 18

CO. DIST. OF ALGOMA

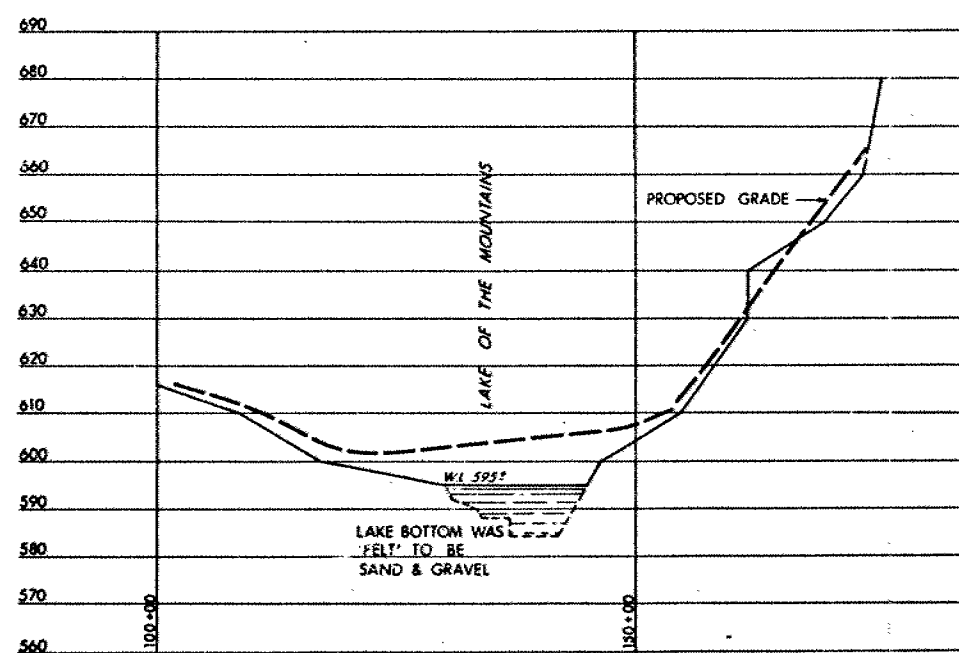
TWP. LCT. CON.

BORE HOLE LOCATIONS & SOIL STRATA

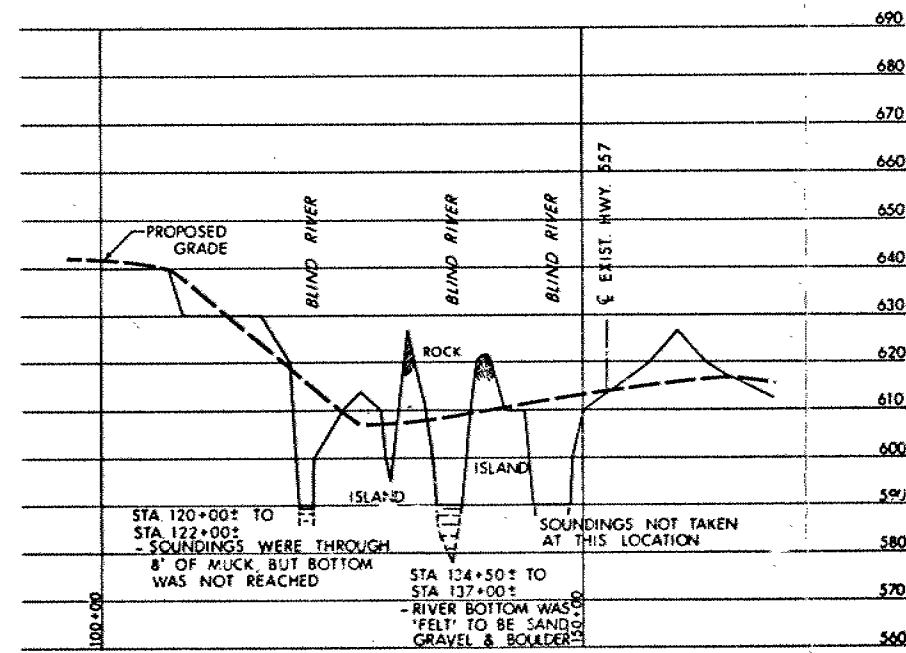
SUBNO H.S.	CHECKED	W.P. NO. 931-73-00	DRAWING NO.
DRAWN O.Y.	CHECKED	W.O. NO.	9317300-B
DATE	JULY 10, 1975	SITE NO.	BRIDGE DRAWING NO.
APPROVED		CONT. NO.	



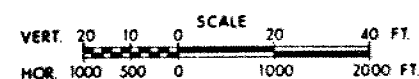
ROUTE 'C3'



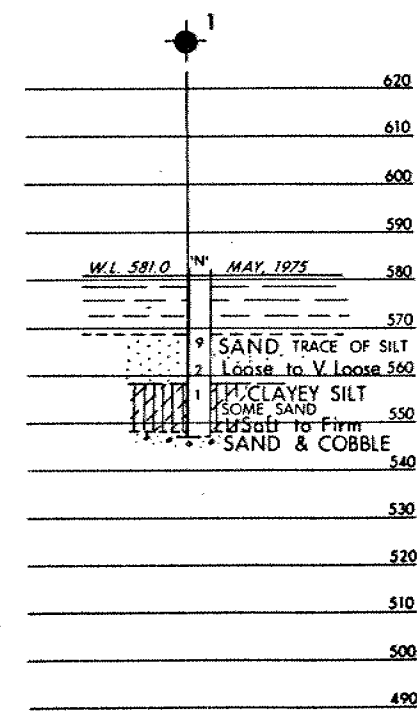
ROUTE 'F'



ROUTE 'C1'



NOTE:
SOUNDINGS BY PROBING FOR ROUTES
'C1' & 'F' TAKEN BY ENGINEERING
SURVEYS, NORTHWESTERN REGION



B.H. 1

NOTE:
B.H. 1 WAS PUT DOWN NEAR THE
SOUTH END OF GOVERNMENT WHARF.
IT WAS PUT DOWN AS A CONTROL
POINT FOR A POSSIBLE FUTURE
GEOPHYSICAL SURVEY.

SEE DWG. 9317300-A

KEY PLAN

LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Resistance Test
B/F CONE - Blows/Ft. Cone Test (350 ft lbs. energy/blow)
- ⊕ Bore Hole & Cone Test
- ⬇ Water Levels established at time
of field investigation.

NO.	ELEVATION		

NOTE -

The boundaries between soil strata have been established only at
Bore Hole locations. Between Bore Holes the boundaries are assumed
from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS-ONTARIO
ENGINEERING SERVICES BRANCH-GEOTECHNICAL OFFICE-SOIL MECHANICS SECTION

FEASIBILITY STUDY

BLIND RIVER AREA BYPASS

HIGHWAY NO. 17 DIST. NO. 18

CO. DIST. OF ALGOMA

TWP. LOT. CON.

BORE HOLE LOCATIONS & SOIL STRATA

SUBNO H.S.	CHECKED	W.P. NO. 931-73-00	DRAWING NO.
DRAWN O.Y.	CHECKED	W.O. NO.	9317300-B
DATE	JULY 10, 1975	SITE NO.	BRIDGE DRAWING NO.
APPROVED		CONT. NO.	

memorandum



To: Mr. D. Aspinwall,
Head, Planning and Design,
Northwestern Region

Date: 81 09 18

ATTENTION: Mr. Dave Hunt

From: Pavement & Foundation Design Section,
Room 315, Central Building,
1201 Wilson Avenue,
Downsview, Ontario

Re: Foundation Assessment of Blind River
By-Pass Route Revisions (Lines ClA - ClD)
W. P. 931-73-00, District 18

INTRODUCTION

As part of the finalization of the route planning project for the Blind River By-Pass, four lines (ClA - ClD) are presently being considered for the actual Blind River crossing. The Pavement & Foundation Design Section was requested to provide a comparative foundation assessment for the possible structures at the four suggested river crossings and identify any major foundation problems which may be encountered.

On 81 08 18, a site visit was made by Mr. D. Hunt and the writer in the company of two M. N. R. personnel. Each of the crossings was inspected and, where possible, shallow hand auger probings were carried out along the shoreline.

This memo will summarize our factual assessment of the alternative lines based on visual observations and inspection, plan and photo interpretation and a review of terrain analysis in the area. Reference is made to the original foundation assessment report written by this Section dated August 15, 1975 which incorporates a geological summary of the area and comments on the original feasibility studies for the alternative By-Pass routes. A complete foundation investigation program will be required upon selection of the final river crossing scheme to ascertain actual subsurface conditions and provide final foundation recommendations for design purposes.

SITE DESCRIPTION AND GEOLOGY

The four alternative crossings are located some 5,000 feet (Line ClA) to 8,000 feet upstream (Line ClD) of the existing Highway 17 and Blind River structure in the Town of Blind River, District of Algoma.

Line ClA is the most southerly crossing of Blind River and traverses two islands referred to as east and west islands with the main river flow between the two islands. Line ClB crosses immediately north of ClA and also traverses the west island in a N-E to S-W direction. Lines ClC and ClD cross Blind River parallel to and some 1,000 and 2,000 feet respectively upstream of Line ClB. In addition ClD traverses a small island referred to as north island.

Geologically, the study area is located in the Precambrian Shield Region of ancient crystalline rocks which consist of a competent intrusive quartz diabase dyke as illustrated by massive outcrops on east and west islands. A major fault in this area, referred to as the Murray Fault, runs relatively parallel to Line ClA some 1,000 to 2,000 feet to the south. The diabase dykes strike east, paralleling the Murray Fault and dip vertically, suggesting they have intruded pre-existing fractures.

Structurally, the area is part of the southern anticlinal limb of the Blind River reverse - S fold which strikes just south of east.

TERRAIN ANALYSIS

The proposed river crossing locations encompass three predominate surficial landforms.

1. Glaciolacustrine
2. Glaciofluvial
3. Bedrock

Glaciolacustrine Deposits

Along the east shoreline of Blind River, Line ClA crosses glaciolacustrine plain deposits consisting predominately of redeposited glacial silts and sands which form plains of low relief and planar topography. Shallow cuts along Highway 555, immediately east of the crossing, consist of loosely compacted silty sands with gravel interbedded with occasional cobbles and boulders. Borings for the Dawsey Creek structure at Highway 555 indicate extensive deposits of very loose to compact silts and sands which appear typical for the immediate east crossing location.

Glaciofluvial Deposits

Surficial deposits along the east shoreline of Blind River for crossings ClB, C, and D are described as glaciofluvial

outwash deposits consisting of loosely compacted silty to gravelly sands overlying bedrock. These deposits are typical for most river valley deposits in the area and are often covered by a thin veneer of recent sandy alluvium. The hand auger probe on the east shoreline at the ClC crossing, advanced to a depth of 10 feet, encountered 0.6 feet of organics underlain by a loose, grey sand to silty sand. Refusal was not met in this probe. Extensive low lying swampy areas consisting of shallow peaty organics exist along the east shoreline of Blind River at the crossings of Lines ClB, C, and D.

At the Blind River North bridge site on Highway 557 (Contract 73-84), a very loose sandy silt to silty sand deposit overlying bedrock was encountered on the east side with a confined wedge of soft to stiff layered silty clay on the west side. Generally, a low local relief with planar topography are associated with this type of landform.

Bedrock Landforms

The west Blind River shoreline for all four lines traverse terrain classified as bedrock knobs. In general, where bedrock is not exposed, a thin veneer of ground moraine till covers bedrock. Also localized peaty organic deposits occur in many of the poorly drained areas between bedrock highs. Exposed bedrock was visible on the west shoreline along lines ClA, B, and C, however, extensive swamp conditions extended along the west shoreline at line ClD. In addition, exposed bedrock was visible at various locations along the three island crossings. A hand auger probe placed between lines ClA and ClB on the west shoreline of the west island encountered one foot of organics underlain by two feet of loose silty sand with refusal met at three feet on probable bedrock.

Relief in this area is moderate to high with the topography classified as rugged.

GROUNDWATER

In consideration of the predominately granular nature of the subsoil at the crossing sites where overburden exists and the low lying relief of the glaciofluvial and glaciolacustrine deposits, groundwater levels are anticipated to be close to ground surface, reflecting Blind River water levels.

DESIGN CONSIDERATIONS

In lieu of specific structure design information (i. e., structure type, span arrangements, profile grades, approach embankment configuration, etc.) and the assessment nature of the anticipated subsoils at the four crossings, only those broad foundations considerations will be addressed which we feel will influence the route evaluation process. Our assessment of foundation considerations for each route are as follows:

Line ClA

Assuming short causeways will join the east and west islands to the river banks and a bridge structure will span the main channel between the islands, no major foundation problems are anticipated. Abutment elements for the structure can be designed with spread footings founded on competent exposed bedrock. Pier elements may require deep foundations (i. e. piles) depending on subsoil conditions within the channel. No stability settlement problems are anticipated for causeways and/or approach fills provided subexcavation of the minor surficial organic material is carried out prior to embankment construction and expected granular material is present at the causeway locations. From a geotechnical point of view, this route appears to provide the most practical crossing location, minimizing both construction and future maintenance costs.

Line ClB

Although similar causeway construction as Line ClA can be used to join the west shoreline to the west island, a more complex causeway/structure combination will be required to join the east shoreline to the west island. Depending on the main channel soil conditions, some form of costlier deep foundations will be required to support structural elements. In addition, larger quantities for subexcavation of organics will be required for causeway construction along the low lying swampy east shoreline. Assuming granular subsoils exist under the proposed causeway locations, minimal stability/settlement distress are anticipated, however, a complete subsurface investigation program will be required to verify this assumption.

This route alternative is the second most favourable from a geotechnical point of view, since it will minimize fill quantities as compared to lines ClC and ClD; however, it will require a more elaborate foundation treatment for the structure as compared to line ClA.

Line ClC

This line will require the largest open river water crossing necessitating an extensive causeway/bridge design scheme. Similar deep foundation treatment and swamp subexcavation on the east bank will be required as with Line ClB. In addition, an extensive rock cut will be required on the west river bank in order to insure low causeway grades, otherwise greater embankment and structure heights with rock-face preparation will be required. This route is the second least favourable crossing location particularly considering the large quantities of fill and rock cut required.

Line ClD

The line traverses north island which will reduce some of the causeway length required. However, very extensive low lying swampy areas occur on both the river shoreline necessitating major subexcavation treatment prior to causeway construction. In addition to the use of deep foundation elements in the river channel for structural support, a large diameter culvert or second bridge structure between the north island and east shoreline may be needed to satisfy hydrological requirements. The crossing presents the least attractive alternative from a geotechnical point of view, as well as posing the greatest impact on the environment, of the four alternative routes.

These foundation considerations are discussed for preliminary planning purposes only, based on visual observations and review of available surficial soil maps. A complete foundation investigation program will be required to provide detail design recommendations upon completion of structural planning functions.

We trust the information provided is sufficient in scope for your immediate requirements, if further discussion is warranted, please feel free to contact this Section.



T. J. Kazmierowski, P. Eng.,
Foundations Engineer

TJK/bd

memorandum



To: Mr. T. Kazmierowski
Pavement and Foundation
Design Section
Central Building
Downsview

Date: 81-08-21

From: Planning & Design
Northwestern Region

Re: Blind River By-Pass Route Revisions
W.P. 931-73-00 - District 18

As agreed during our 81-08-18 field review, please accept this memo as a Regional Request for foundation information, in regard to the above route revision (CIA to CLD previously identified).

Your comments are required for the foundation areas only, of the four crossings, as soil data will be obtained from the Regional Office for approach road alignments.

Could you please provide comments, as based on previous studies and our recent field trip, concerning;

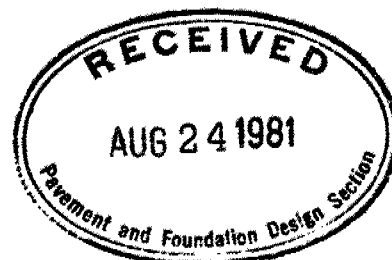
- (i) any major foundation problems which could be encountered, by any route option.
- (ii) a comparative description of the relative merits of the four options.
- (iii) any other items which you feel warrant identification.

Thank you for your assistance on this project. A reply by 81-09-16 would be most appreciated.

Dave Hunt

DAVE HUNT
Senior Environmental Planner

DH:fb





Ontario

ACTION REQUEST

540-1037 (2-72)

DATE

sept 10/75

TO

FROM

TELEPHONE NO.

☐ - PLEASE CALL

4093173-00

☐ - WISHES APPOINTMENT☐ - RETURNED YOUR CALL☐ - WILL CALL BACK☐ - NOTE AND FILE☐ - PROVIDE MORE DETAILS☐ - PLEASE ANSWER☐ - NOTE AND FORWARD☐ - FOR YOUR INFORMATION☐ - DRAFT REPLY FOR MY SIGNATURE☐ - NOTE AND RETURN☐ - FOR YOUR APPROVAL☐ - INVESTIGATE AND REPORT☐ - NOTE AND SEE ME☐ - FOR YOUR SIGNATURE☐ - TAKE APPROPRIATE ACTION☐ - RETURN WITH COMMENTS☐ - PER YOUR REQUEST

COMMENTS:

Since we are far from detailed
 attempt at this stage and time is
 valuable to assume the best for
 each ~~part~~ ^{part} of this stage. Hence

CALL TAKEN BY:

TIME

Mr. Prof. Valdes. I agree with
 an instruction mark.

74-4938



ACTION REQUEST

7540-1037

DATE

Sept 9

TO

Com.

FROM

A. RUTKA

TELEPHONE NO.

☐ - PLEASE CALL☐ - WISHES APPOINTMENT☐ - NOTE AND FILE☐ - NOTE AND FORWARD☐ - NOTE AND RETURN☐ - NOTE AND SEE ME☐ - RETURN WITH COMMENTS☐ - RETURNED YOUR CALL☐ - PROVIDE MORE DETAILS☐ - FOR YOUR INFORMATION☐ - FOR YOUR APPROVAL☐ - FOR YOUR SIGNATURE☐ - PER YOUR REQUEST☐ - WILL CALL BACK☐ - PLEASE ANSWER☐ - DRAFT REPLY FOR MY SIGNATURE☐ - INVESTIGATE AND REPORT☐ - TAKE APPROPRIATE ACTION

COMMENTS:

I hope you didn't get a copy!

CALL TAKEN BY:

TIME

74-4938

Mr. B.J. McKenna,
Reg. Struct. Plan. Eng.,
Northwestern Region,
Thunder Bay.

Hydrology Section

Sept. 4, 1975.

Re: Proposed Blind River Causeway
WP 931-73-00 BW 2602
Hwy. 17 (Feasibility Study)
Dist. #18, Sault Ste Marie

Interim Hydrology

This memo provides further design details for protecting the proposed causeway against wave and ice forces.

Notes

1. Information provided by the Soil Mechanics Section indicates that it may be very difficult to obtain large (3 ton) armour, locally, and that the specific gravity of the local rock is relatively low (140 pounds per cubic ft.).
2. The North-west Region Office advises that the quantity of rock available from excavation along the proposed route is limited.
3. Heavy "trap rock" is probably available from quarries at Thessalon or Spragge.
4. An "earth" core is acceptable, for the causeway, if rock fill is limited.
5. A "flat" slope of 3:1, is preferred on the lake side of the causeway, for the following reasons:
 - (a) allows use of smaller size armour stone units, possibly allowing use of rock from the right of way.
 - (b) ice floes are more likely to ground and form a barrier to further ice thrusts.
 - (c) should scour occur at the toe of slope, the flatter slope provides greater protection against damage and possible failure.
 - (d) the edge of water, particularly for high lake levels, will be at a greater distance from the travelled roadway, thus reducing to some degree, the hazard of spray during storms off the lake.

6. The basic design of the protective stone on the lake side consists of (a) an outer layer of armour stone (2 layers, fairly uniform size) (b) a backing layer of stone (graded) and (c) possibly a filter blanket, depending on the core material.

On the land side, a 3 ft. layer of standard random rip-rap should suffice. A filter blanket may be required, depending on the core material grading.

7. As in our memo of February 18, 1975 the armour stone should be taken to elev. 588 min. In addition, the embankment on the lake side should provide rip-rap to elev. 592 min. to protect against wave run-up.

The minimum grade elevation is open to discussion and is basically a question of safety factors versus cost. A freeboard of 3 ft. above the calculated maximum wave run-up elevation is considered reasonable; thus the recommended minimum grade is elev. 595.

8. The attached sketch shows the general design features as recommended in this memo.

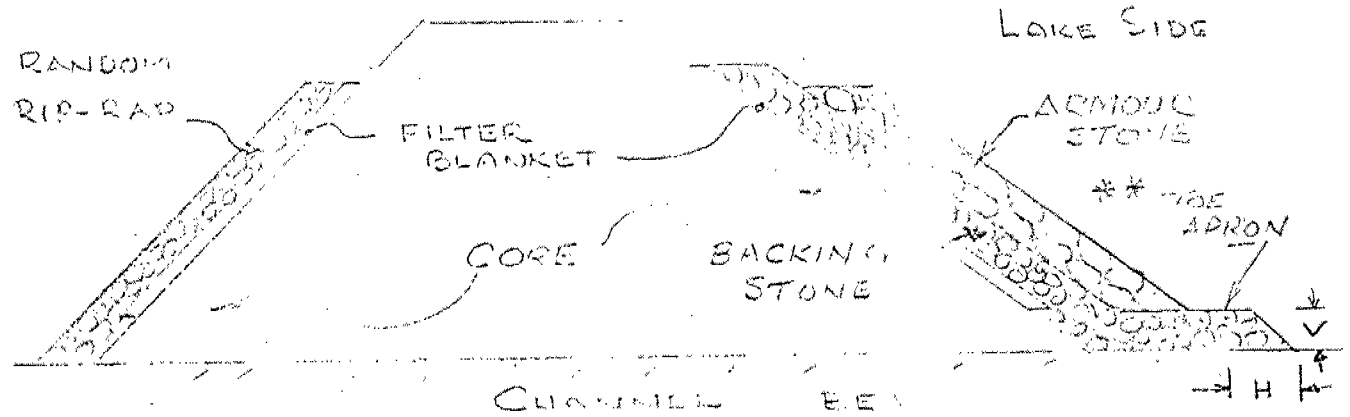


J.W. Carter,
Senior Hydrology Engineer.

JWC/rh
attach.
cc: G. Allen

LAND SIDE

LAKE SIDE



NOTES

ALL ELEVATIONS (S.C.)

MIN. GRADE — 558 (TENTATIVE)

MAX. WAVE RUNUP & TOP BACKING STONE — 592

TOP OF ARMOUR STONE — 588 (ALSO TOP OF RIP-RAP ON LAND SIDE)

* HIGHEST INSTANTANEOUS LAKE LEVEL 582.3

* AVERAGE LAKE LEVEL 578.6

* LOWEST DAILY MEAN LAKE LEVEL 575.5

LOWEST WAVE (TROUGH) 573 ±

AVERAGE CHANNEL BOTTOM (SHE) 567 ±

* AT THESSALON

RECOMMENDED PROTECTION (CONTINUED)

I LAKE SIDE 3:1 SLOPE

1. ARMOUR STONE — UNIFORM SIZE — 2000lb. 5 ft. THICK LAYER

2. BACKING STONE — GRADED — $D_{85} = 200\text{lb.}$ $D_{15} = 50\text{lb.}$

— 2.5 TO 3 ft. THICK LAYER

3. FILTER BLANKET *

LAND SIDE 2:1 SLOPE ACCEPTABLE

1. PROVIDE 3 ft. THICK LAYER RANDOM RIP-RAP PLUS

2. FILTER BLANKET *

II LAKE SIDE 1 1/2:1 SLOPE (FOR COMPARISON, NOT RECOM.)

1. ARMOUR STONE — UNIFORM SIZE — 4000lb. 7 ft. THICK

2. BACKING STONE — GRADED — $D_{85} = 400\text{lb.}$ $D_{15} = 50\text{lb.}$

13.0 TO 3.5 ft. THICK LAYER

3. FILTER BLANKET *

LAND SIDE — SAME AS FOR I

* FILTER BLANKET TO BE SPECIFIED WHEN

CORE MATERIAL GRADING DETERMINED

* * MIN. TOE APRON: H = 6 ft. V = 3 ft.

BLIND RIVER HARBOUR

"MAP # A-1-a"

SCALE: 1" = 1320' ±

North
↑

ROUTE "C-3"

DREDGED
CHANNEL



MAP # 177C / 197

"MAP #A-1-6"

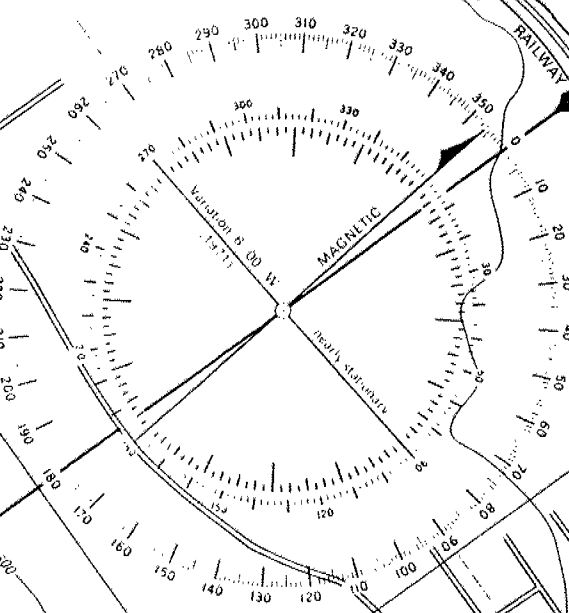
SCALE: 1"=1000'

A

HIGHWAY 17

CANADIAN PACIFIC RAILWAY

NORTH



Blind River

Booming Ground

Dorothy Inlet

Byrner
Ch.
Kiln

G.W.T.

Harriette Pt.

Ruins
Public Whf.

Ruins
Piles
Crib

BLIND RIVER

W.I.Z.Z.A.

R.C.

Dam

Marina

C.B.H.

Tower
100 ft.
(PA)

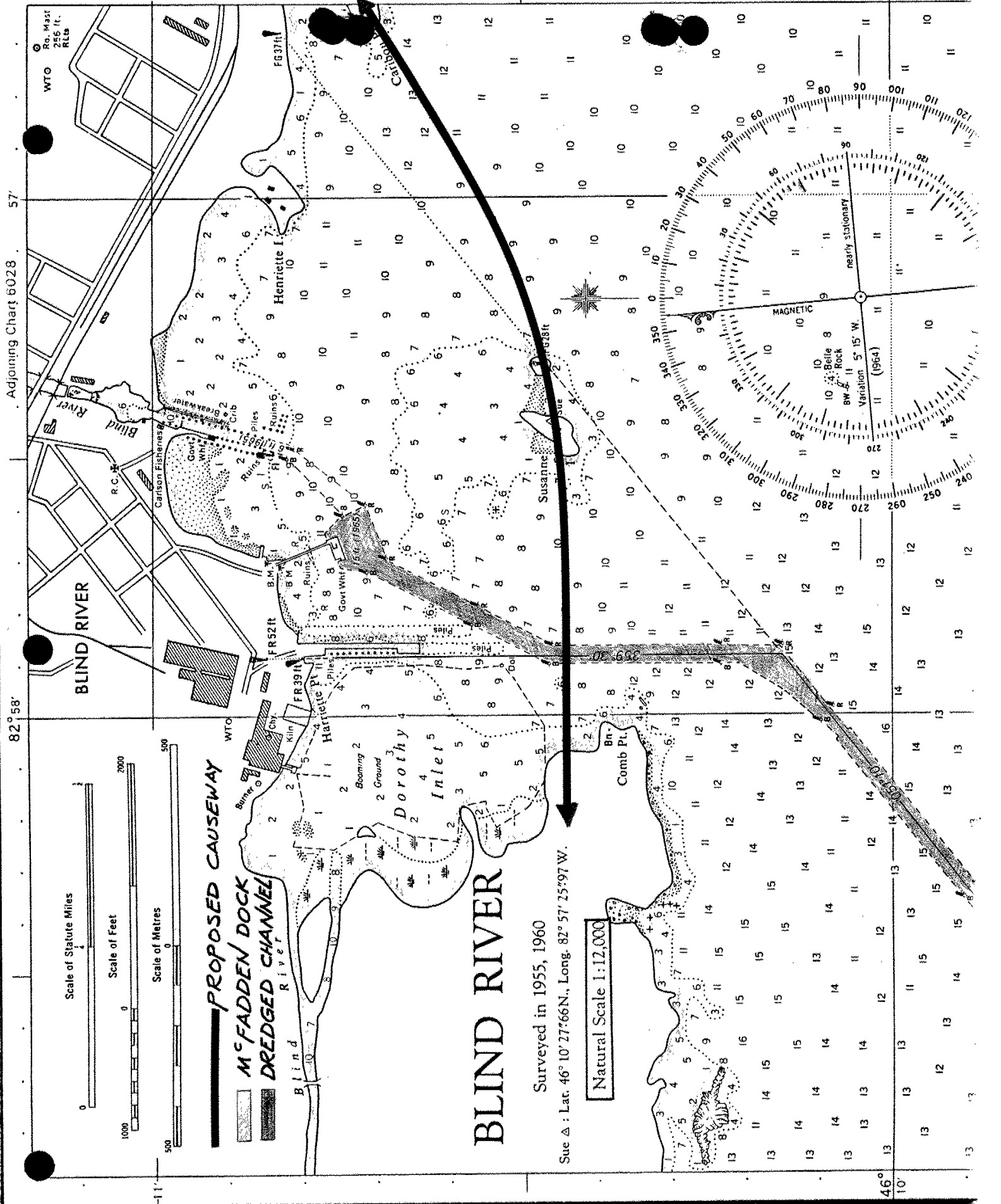
NOT NAVIGABLE

Douce Rk.

NORTH CHANNEL

(LAKE HURON)

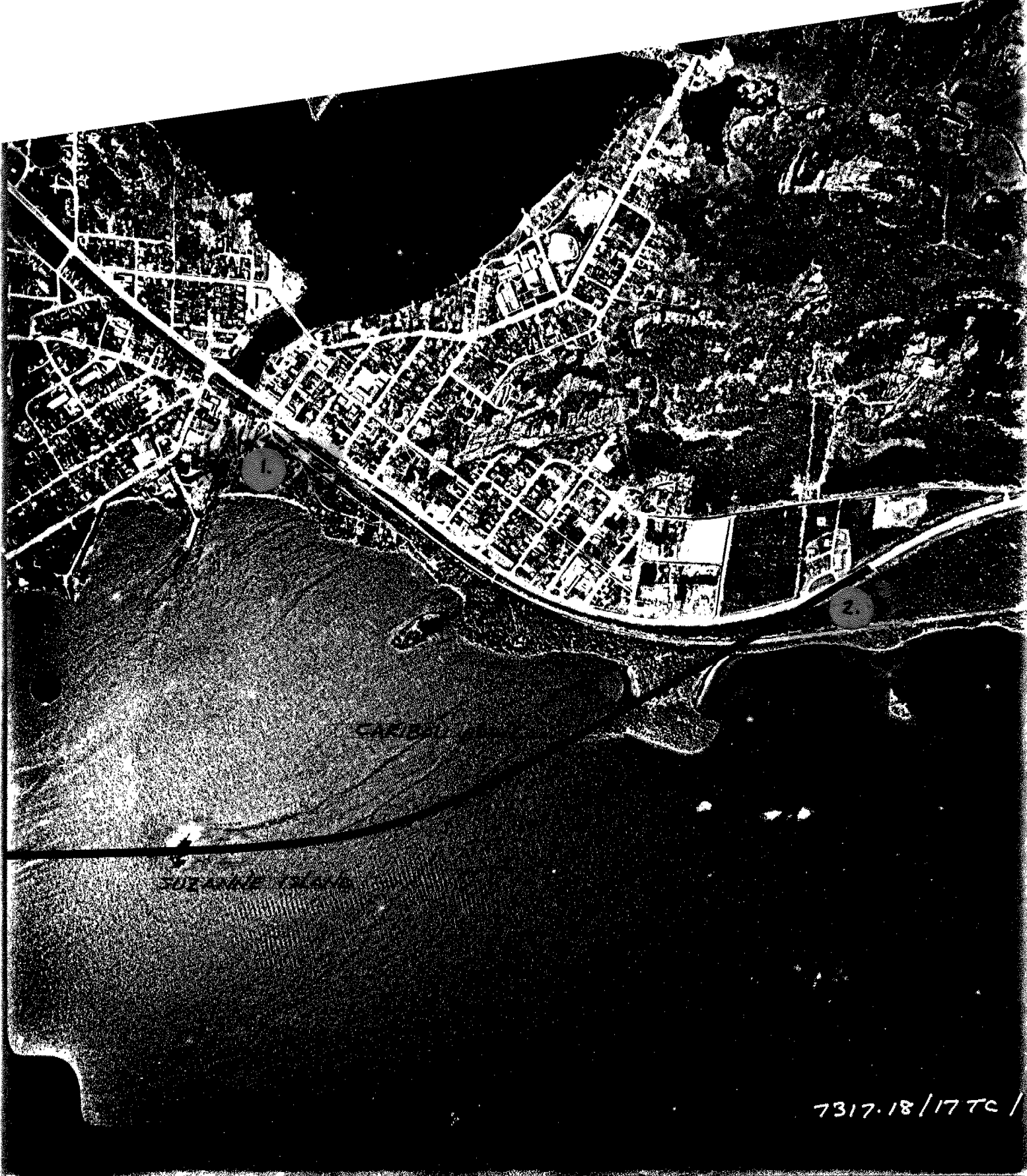
(Use Chart 2,768)



Adjoining Chart 6028

82° 58'

"MAP # A-1-c"



7317.18/17 TC /

SCALE: 1" = 1320'

BLIND RIVER HARBOUR
PROPOSED ROUTE (C-3)

↑ NORTH

"MAP #A-2-a"



POSSIBLE LOCATIONS (:: M.O.E.)
FOR PROPOSED SEWAGE TREATMENT PLANT

83°00'

55'

"MAP # A-1-b-1"



CANADA

ONTARIO - LAKE HURON

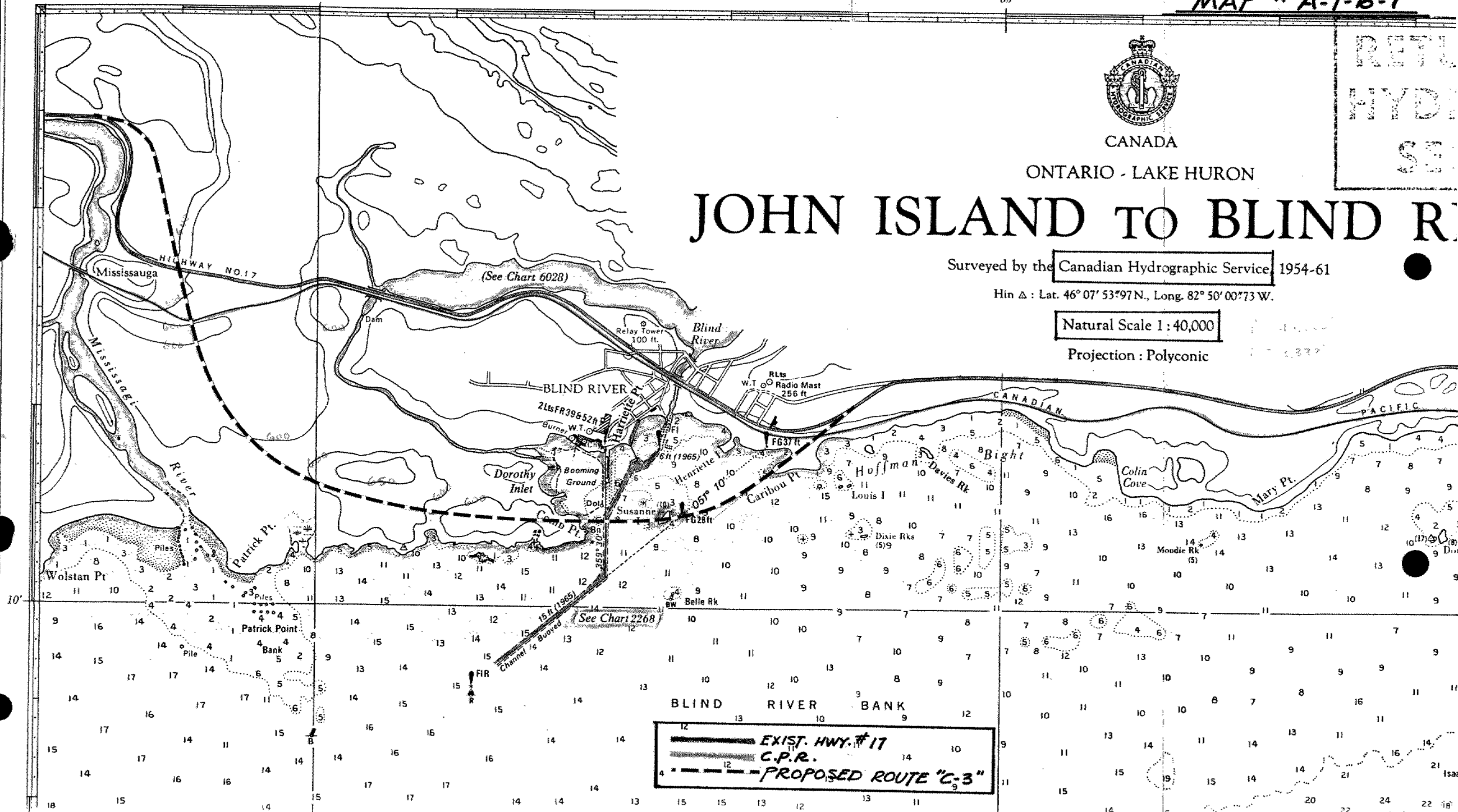
JOHN ISLAND TO BLIND R

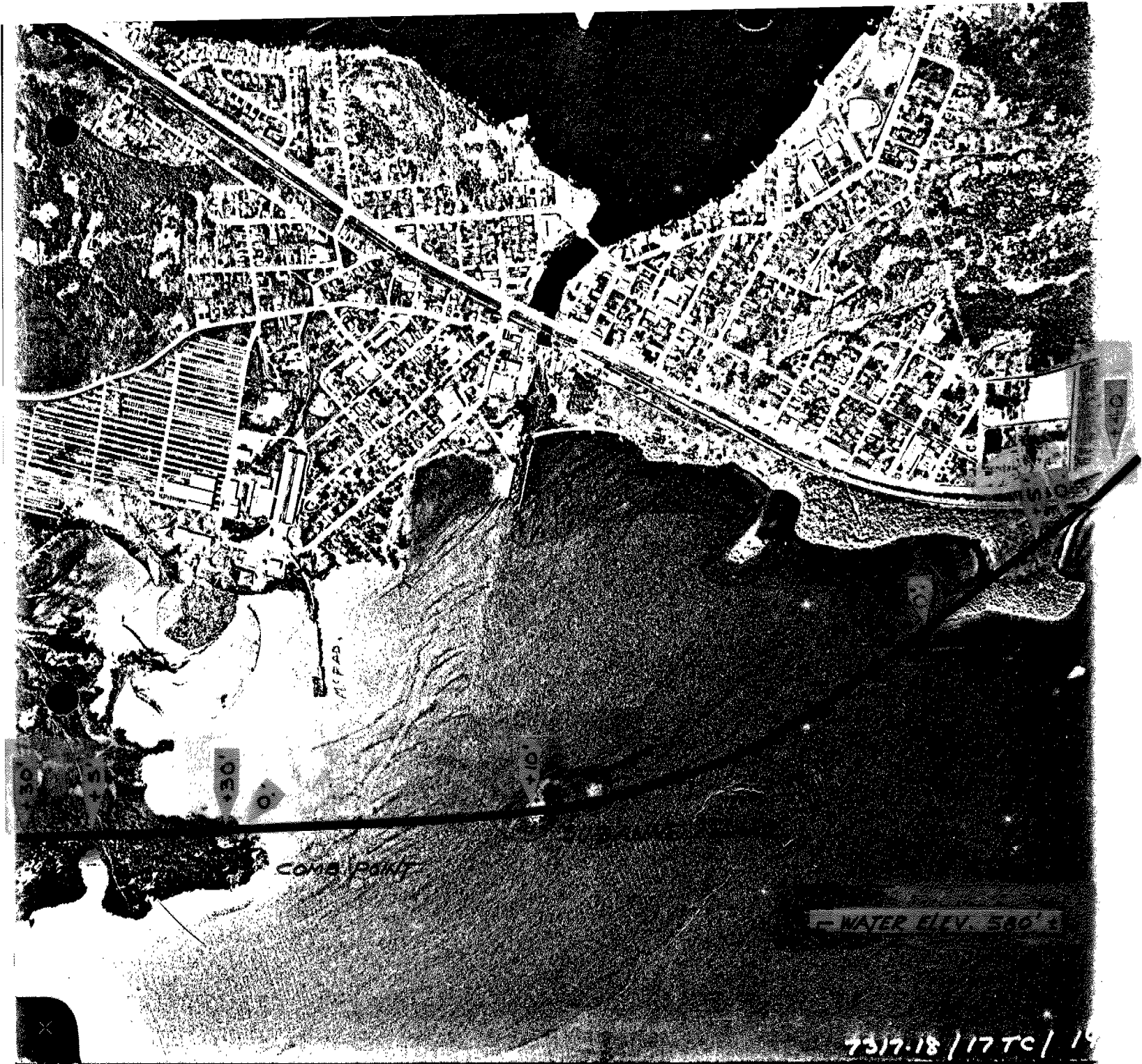
Surveyed by the Canadian Hydrographic Service 1954-61

Hin Δ : Lat. 46° 07' 53" 97" N., Long. 82° 50' 00" 73" W.

Natural Scale 1 : 40,000

Projection : Polyconic





BLIND RIVER HARBOUR

"MAP #A-4-a"

SCALE : 1" = 1320' ±

↑ NORTH

SHALLOW WATER (CAN SEE BOT.)

PROPOSED ROUTE (C-3)

SPOT ELEV.^s ALONG PROPOSED ROUTE

POOR DRAINAGE (HIGH WATER TABLE)

LEGEND FOR AIR-PHOTO -
INTERPRETATION

BLIND RIVER

W.P. # 931-73-22

"A-8-a-1"

ELEVATIONS (ft.)
ABOVE MEAN
SEA LEVEL
1" = 200'

500

600

700

800

900

1000

1100

1200

1300

1400

1500

1600

EL. 1500

SERPENT LAKE

LAKE

EL. 1280

PEPLER LAKE

MATINENDA LAKE

EL. 774

MATINENDA LAKE

MATINENDA LAKE

EL. 764

CHIBLOW LAKE

DAM & POWER HOUSE

CATARACT LAKE

EL. 600

LAKE OF THE MOUNTAINS

BLIND RIVER

TOWN OF BLIND RIVER

DOROTHY INLET
HARBOUR

NORTH CHANNEL

EL. 580

HORIZONTAL SCALE: 1:250,000

5 MILES

OVERSIZE DRAWING

Scale of Statute Miles

Scale of Feet

Scale of Metres

BLIND RIVER

-----PROPOSED CAUSEWAY

WATER DEPTH
CONTOURS

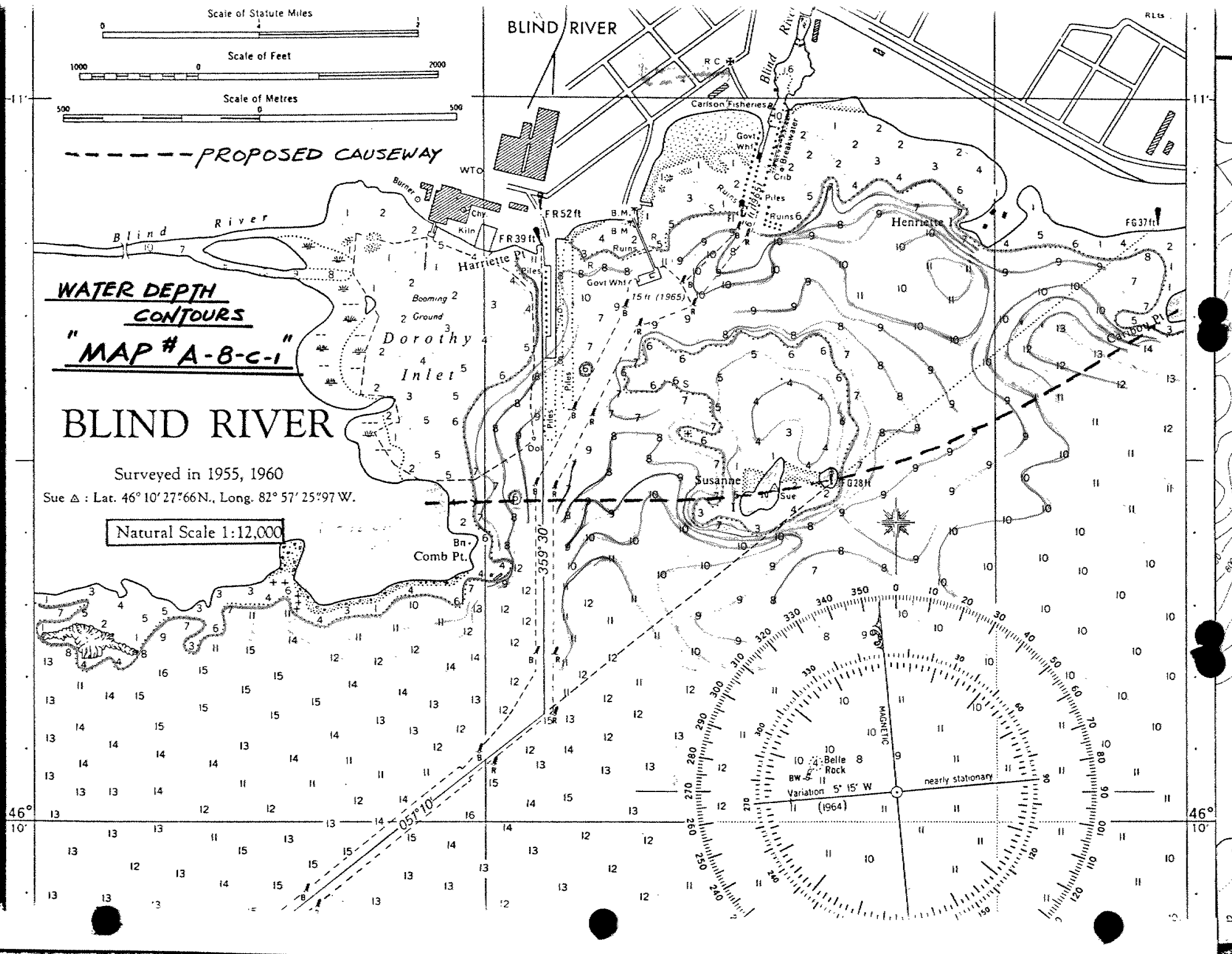
"MAP # A-8-C-1"

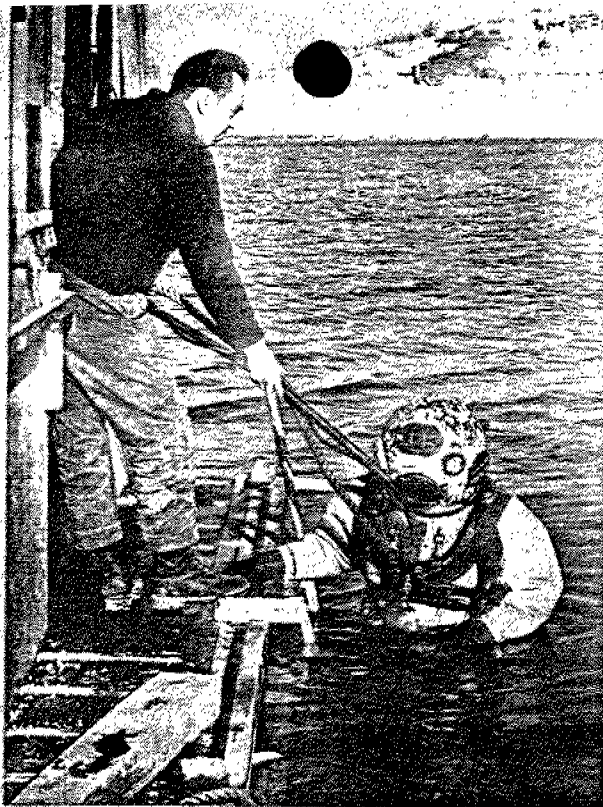
BLIND RIVER

Surveyed in 1955, 1960

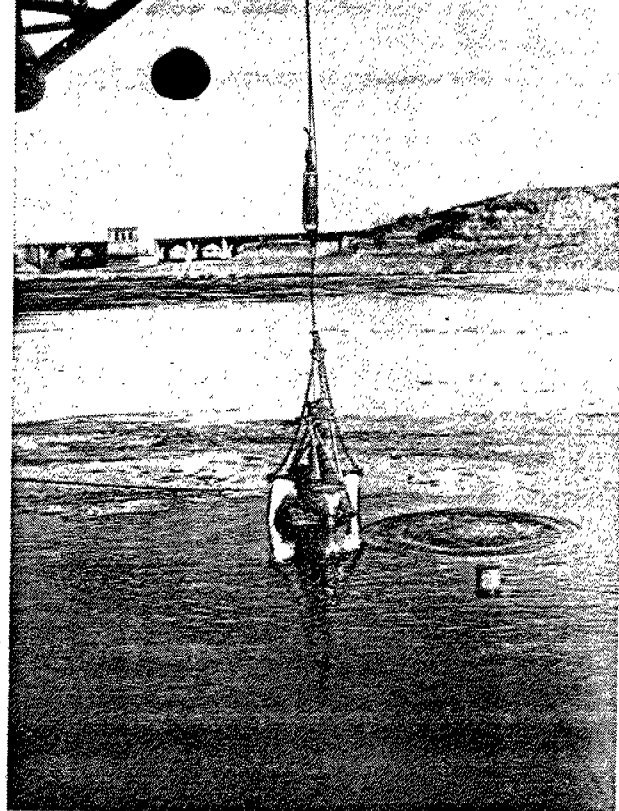
Sue Δ : Lat. 46° 10' 27" 66N., Long. 82° 57' 25" 97 W.

Natural Scale 1:12,000





GOING DOWN—Sealed inside his 200-lb water-tight outfit, a diver prepares to leave scow and descend 28 ft to reservoir bot-



tom. Above, circular pattern of surfacing air bubbles reveals diver's position to crane operator lowering rock with a grapple.

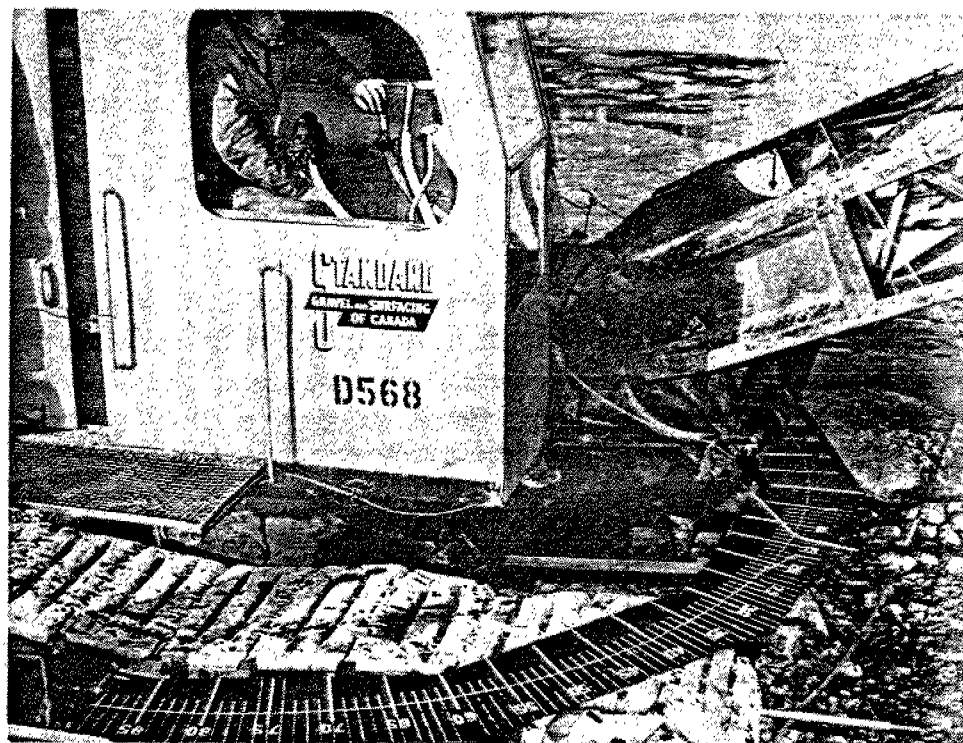
Divers 'Hand-Place' Riprap

DIVERS IN HARD HATS are spotting riprap in 28 ft of water for a Canadian contractor. Working in ice-cold water and in almost total darkness, the divers radio instructions to the surface crew and feel-fit the big rock chunks while a crane grapple sets them.

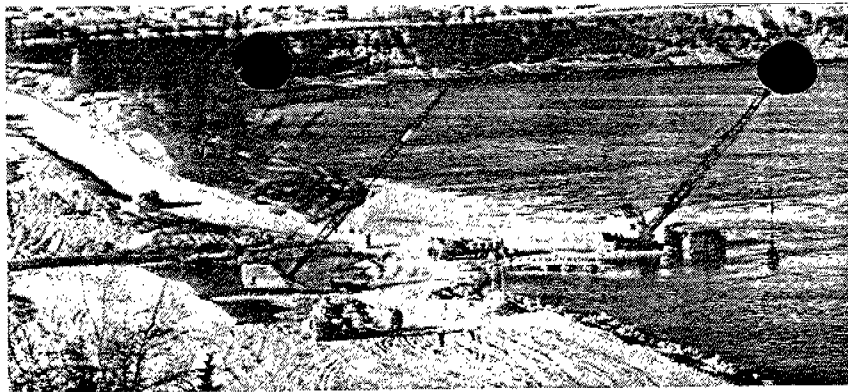
Standard Gravel and Surfacing of Canada, of Calgary and Edmonton, Alta., is building a causeway across Calgary's Glenmore Reservoir for a six-lane bypass highway. A 140-ft channel through the causeway will be bridged to allow water to flow freely in the reservoir. This channel is the site of the unusual riprap work.

Consultants Haddin, Davis & Brown Co., Ltd., of Calgary wanted maximum stability in the channel lining to prevent scour of the loose-silt bottom. They stipulated that each piece of rock ($\frac{3}{4}$ -ton minimum) for the 10,000-yd riprap must be "hand" placed.

continued on page 116



GIANT PROTRACTOR—Wrapped around front of crane, 180-deg graduated arc shows boom angle from centerline. Vertical angle indicates radius for plotting rock positions.



RESERVOIR CROSSING—Riprapping operation proceeds at right while earthmovers build fill on near and far sides of channel. Temporary crossing (center) is culverted.

DIVERS PLACE RIPRAP . . .

continued

Standard sent to Vancouver for experienced divers and brought back an eight-man crew—four divers and four tenders—from Universal Diving, Ltd. The gang has found that working in a prairie reservoir is no picnic.

The crew puts in a 20-hr working day; each diver works alone, and the four rotate on 2½-hr shifts. Even with two or three sets of long Johns and a heavy flying suit under his water-tight outfit, a diver must keep moving to stay warm in the 27-deg water. And his 200 lb of gear, including lead-weighted shoes and belt, makes moving something of a chore.

Crane operator and tenders pay close attention to the diver's bubbles to avoid swinging the heavy rocks over his head.

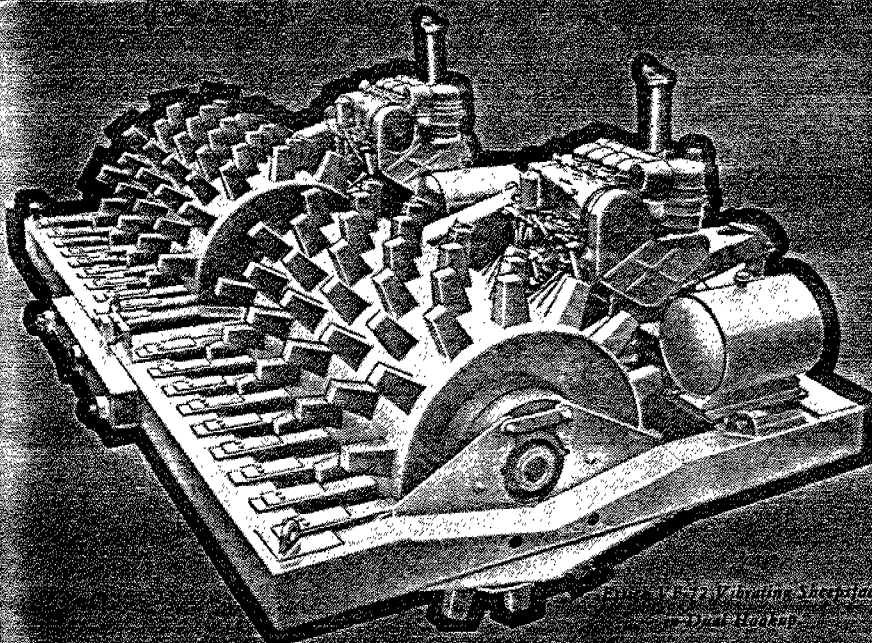
Rocks are lowered on radio command from a diver, who then coaches the crane operator in fine-positioning each piece. The diver has to feel all around to make certain that each rock is in contact with its neighbors because visibility is never more than a few inches. When he is satisfied, he signals for the grapple to be withdrawn.

Diver-inspectors hired by the engineers descend periodically to check the work.

As each rock is put in place by the Northwest crane, a foreman on the divers' scow records its position on a plan-view drawing of the channel. The information he needs is read directly from a huge crane-mounted protractor that gives him the angle position of the boom off centerline, and from the boom-angle indicator, which tells him the radius from crane to rock. As long as the crane crawlers straddle the centerline and the position of the rig along the line is known, the position of each rock can be plotted.

To keep ice out of the working channel Standard laid a 1-in. hose on the bottom after puncturing it with 1/16-in. holes every ft. A 600-cfm Worthington compressor sends air through the hose, and the streams of escaping bubbles agitate the water enough to prevent freezing. A ½-in. cable tied to the hose keeps it on the bottom.

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FOUNDATION ASPECTS OF THE RAINY LAKE CAUSEWAY

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RAINY LAKE CAUSEWAY consists of alternating embankments and bridge structures. Significant physical conditions at the site are water depths up to 50 ft. underlain by about the same thickness of soft to firm clay, and an annual ice cover 3 to 4 ft. thick.

To achieve stable embankments the underlying clay was remoulded by dynamite blasting so that total displacement of the clay would occur during construction. Tests to determine the amount of dynamite required to effect sufficient reduction in the shear strength of the clay are described, and a method of rapid loading of dynamite is given.

To cope with ice pressure the bridge structures are designed to flex and adopt total ice movements. Tests to determine maximum annual ice movements are discussed.

Introduction

The Rainy Lake Causeway is a major link in Highway No. 11 between Atikokan and Fort Frances, Ont. This highway is currently under construction and the causeway is due to be opened shortly. The Highway 11 program has been undertaken by the Ontario Government to provide an alternative route between the Canadian Lakehead and Winnipeg. It will also open up the Rainy River District for tourists at a favorable time when the Mississippi Parkway will be extended into Minnesota, and the traffic over the Lake Superior North Highway can be expected to gather momentum. Rainy Lake extends south into Minnesota and the causeway results in a saving of 40

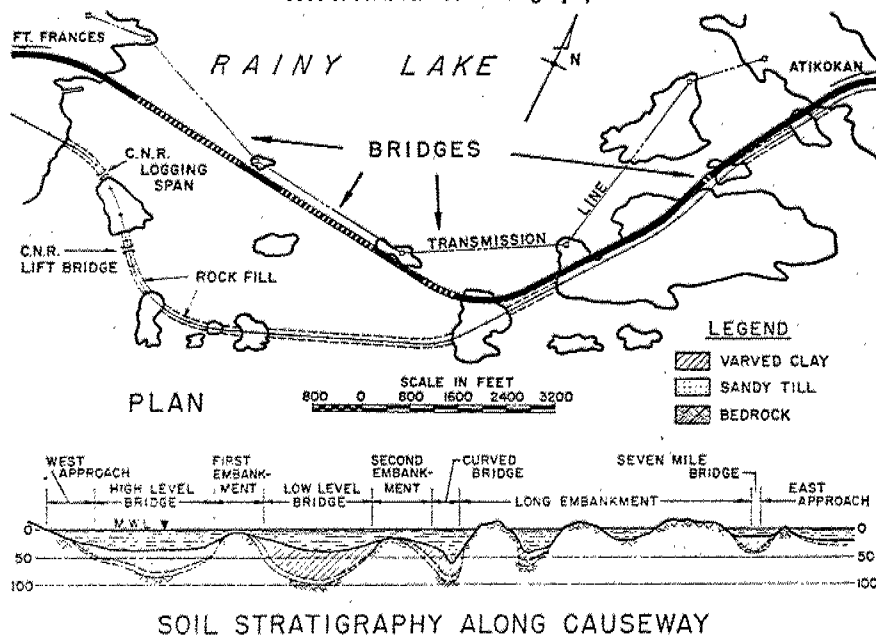
miles which would otherwise be involved if the new highway were carried around the north end of the lake.

For many years Rainy Lake has been used as a transportation route for timber and pulp logs destined to sawmills and paper producing plants located in Fort Frances and International Falls, Minn. In 1900, the Canadian National Railways completed the rail link between Port Arthur, Ont., and Winnipeg. The line was carried across Rainy Lake at Rocky Inlet on a temporary timber trestle. In 1912, the C.N.R. completed a rock fill causeway which incorporates a bascule type lift bridge for navigation purposes, and a fixed bridge span for

logging operations. This causeway is reported to have cost \$2 million. A transmission line crosses the Lake to the north of the railway embankments. It is suspended between towers located on islands in the Lake, and clears the water by about 50 ft. The railway and transmission line crossings are shown in plan on Fig. 1.

The new causeway is 2.8 miles long and will cost about \$4.5 million. It is located between the existing railway embankment and the transmission line, and incorporates an elevated bridge over the navigation and logging channels. It consists of alternating fill embankments and bridge structures named as below, from west to east.

FIG. 1: Plan and Soil Stratigraphy.



West approach embankment	740 ft.
High level bridge	2014 ft.
First embankment	808 ft.
Straight low level bridge	1811 ft.
Second embankment	1002 ft.
Curved low level bridge	453 ft.
Long embankment	4892 ft.
"Seven Mile" bridge low level	138 ft.
East approach embankment	3042 ft.

The actual alignment of the causeway and the locations of the various structures are shown on Fig. 1. The main navigation span and a span for logging operations are located near the middle of the highway level stretch. Figures 2 and 3 show the full length of the causeway.

The causeway provides for 60 m.p.h. two lane traffic and H20-S16 loading. The high level portion gives a clearance of 36 ft. above high water level for shipping purposes.

Site and Geology

Site studies were started in 1957 and, at an early stage, a first order triangulation and a co-ordinate system was established for the soil exploration and the general survey. Bench marks referenced to geodetic survey were also erected on shores and on islands in the lake.

The site investigation included: a comprehensive soil exploration; drilling through the existing embankment; a study of gravel deposits in the area; a general topographical survey, soundings; and an investigation of ice conditions and lake currents.

The geology of the site is typical of the Canadian Precambrian Shield. A high degree of glaciation is in evidence. Sound acidic igneous bedrock, metamorphosed in places, persists throughout. Rock outcrops, practically void of drift, occur frequently and usually have smooth rounded pro-

files and show evidence of striation. Along the causeway alignment the water is 30 to 50 ft. deep. Immediately below lake bottom there is a stratum of varved clay, which is believed to have been deposited during the last glaciation. In places, a layer of compact glacial till occurs between the clay stratum and the bedrock.

Rainy Lake is the beginning of a long waterway extending through Lake of the Woods, Lake Winnipeg and the Nelson River to Hudson Bay. Its level is high at 1107 ft. above sea level. The level of the lake is controlled at the Fort Frances power dam by The International Rainy Lake Control Board.

Soil Conditions

Along the causeway alignment, lake bottom is generally underlain directly by a stratum of soft to firm varved clay which varies in thickness from a few feet in shallow areas, to 30 to 50 ft. under the deeper parts of the lake. The maximum clay thickness recorded elsewhere was 85 ft. A generalized stratigraphic profile along the causeway is given on Fig. 1.

The clay has a pronounced varved structure. Laminae are generally horizontal and are alternately grey and light grey; occasionally they are grey and reddish brown. Their thickness varies from 1/32 in. to 1 in. with an average of about 1/4 in. A photograph of a sample of the grey clay showing the varving is given on Fig. 4. The same figure shows the chunky structures of individual clay lamina in the reddish brown clay, and the inclination of some of the varves to the horizontal. The chunky structure is where the stratum is reddish brown in colour.

The clay generally has a high plasticity. Liquid limits of 50 to 96 were obtained for the grey clay, with corresponding Plastic Limits of 15 to 31. For the reddish coloured clay the corresponding range in Liquid and Plastic Limits was 88 to 134, and 33 to 49, respectively. The natural moisture content of the stratum was found to vary between 40 and 140% with a general value of 60 to 70%. It tends to decrease with depth. Typical pressure void ratio curves for the clay are given on Fig. 5. The Casagrande construction generally gave preconsolidation pressures of the same magnitude as the existing overburden pressure, indicating that the clay is normally loaded. Some preconsolidation is however indicated in the upper part of the stratum in locations where water depths are less than 20 ft.

The undrained shear strength of the clay as measured by vane tests varies from 100 to about 2000 p.s.f. and there is a trend towards uniform increase with depth below lake bottom, where water depths are greater than about 20 feet. A shear strength versus depth plot is given on Fig. 6. Where water depths are less than 20 ft. the clay strength in the upper 10 ft. of the stratum was variable, but the average shear strength is about 400 p.s.f. Undrained triaxial compression tests gave results which are plotted as shear strength on Fig. 6. The sensitivity of the clay was generally in the range of 3 to 6.

The clay stratum is often underlain directly by bedrock. Between the clay and the bedrock in places, however, there is a stratum of grey sandy till containing boulders. The maximum measured thickness of the till is 32 ft. The till is of variable density but it is generally in the

FIG. 2: High Level Structure.

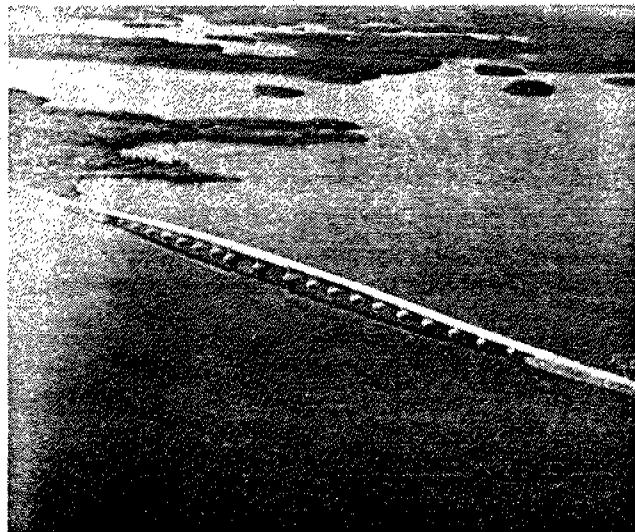


FIG. 3: Low Level Structures and Embankments.

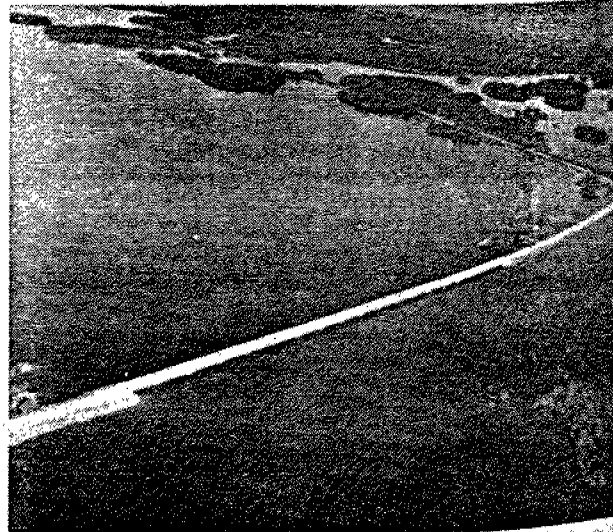
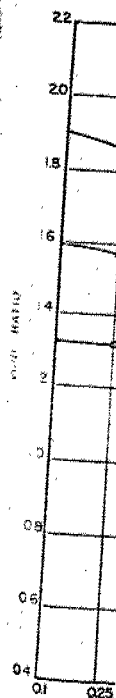


FIG. 5: P
Clay.



loose to compact range. "N" values varied from 1 to 44.

Boreholes through the existing railway embankment indicated that the rock fill had displaced the clay to bedrock over part of the length of the embankment as shown on Fig. 7. Other borings, however, revealed that complete displacement of the clay was not a general condition, and that in some cases the rock fill was "floating" in the clay as shown on the same figure. In some of the boreholes, individual boulders of rock fill were separated by up to one foot of soft clay.

According to construction records for the railway embankment, quarried rock up to 10 ft. in size was placed by end dumping. During construction of the western end of the embankment, difficulties were encountered due to progressive slipping at the end of the embankment. The fill operation became hazardous and it was deemed advisable to support the rail track at the dumping face. For this purpose a large floating gantry was rigged up, which in part straddled the embankment at the end. Penetration tests made at the time indicated up to 60 ft. of soft clay in places. Since completion of the embankment, subsidences apparently caused by both consolidation and progressive displacement of underlying clay have continued and at intervals, most recently in 1949, it was necessary to raise the tracks to grade

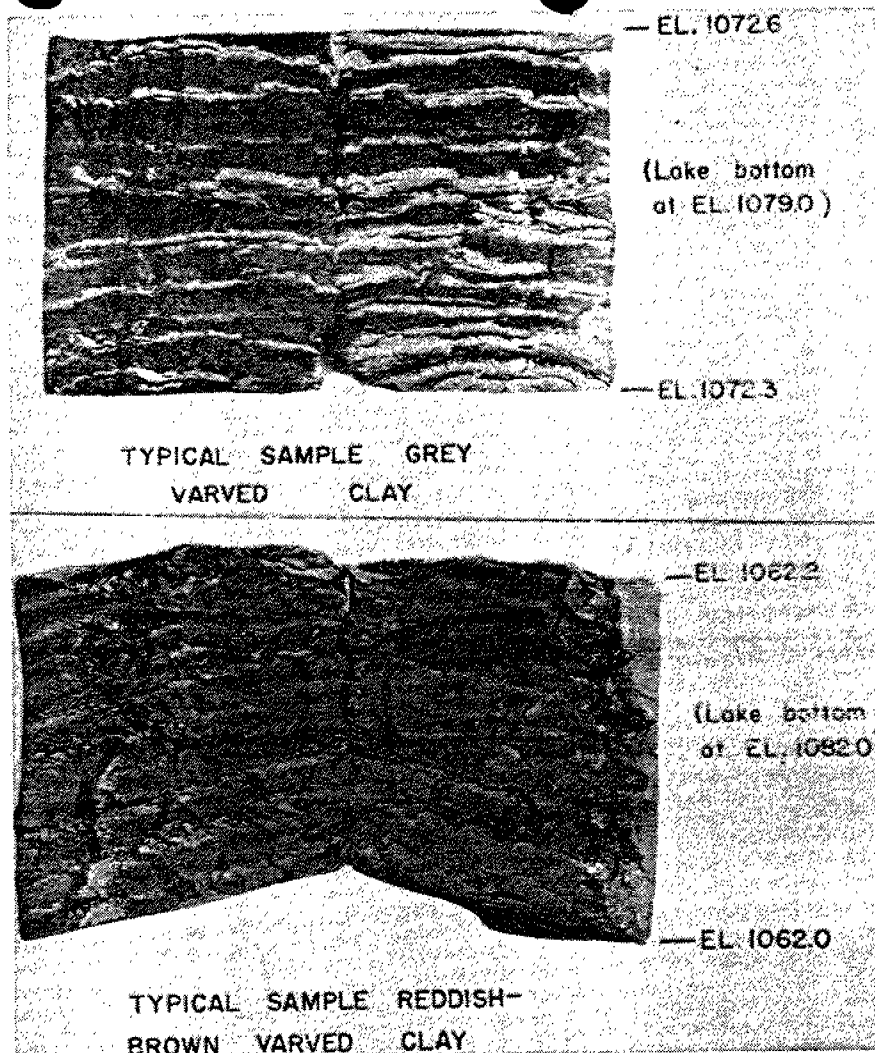
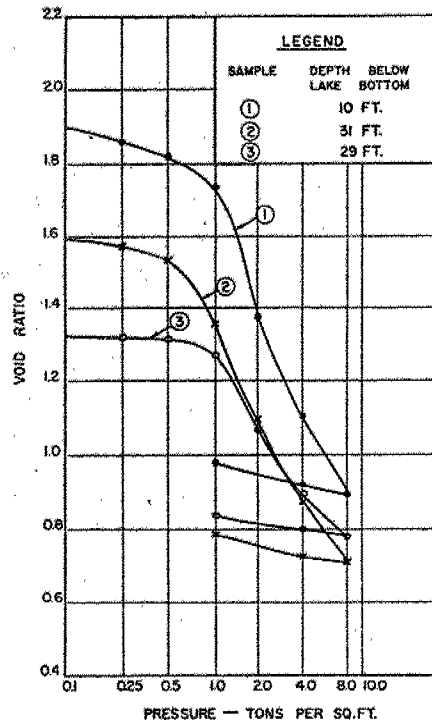


FIG. 4: Samples of Varved Clay.

FIG. 5: Pressure-Void Ratio Curves for the Clay.



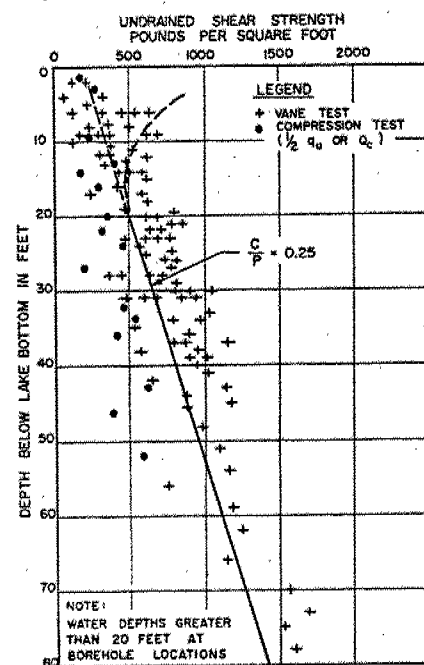
by adding more ballast. In places the total settlement since 1912 amounts to 10 ft.

Ice Conditions

Rainy Lake is ice covered five months of the year. Due to the altitude of the site and the climate, an extremely heavy ice cover is developed, sometimes reaching 4 ft. in thickness. Ice formation always starts in calm bays as board ice. With offshore wind, the ice sheet may proceed on its own lee, towards more open water. If the wind turns, the waves may break up areas of ice cover, although the resulting ice floes at this early stage are not substantial enough to cause damage. Total ice cover usually coincides with the first period of adequate calm and frost which occurs in December. The studies of ice conditions on the lake are described later in the paper.

When a lake gets covered with ice, compressive stresses develop in the

FIG. 6: Strength Versus Depth Profile for the Clay.



ice sheet. The magnitude of compression at any given point is equal in all horizontal directions due to certain plastic qualities of the ice sheet. Cold weather renders the upper layers of the ice sheet stressless and only warm weather creates full ice pressure.

Since water has maximum density at 39.2°F, the temperature profile for the ice-covered lake is always characterized by a gradient which is increasing with depth from 32°F at the underside of the ice sheet, to a possible maximum of 39.2°F. The presence of the coldest water at the underside of the ice sheet has some influence on the uniform growth of the ice sheet. Because of the lower density of this cold water it will flow towards an opening made in the ice sheet, or any hollow in the underside of the ice sheet, providing accelerated freezing in such locations at the expense of the surrounding ice sheet.

In general, the tendency of nature is to maintain the underside of the ice sheet at an even level. However, at a rock fill embankment or a shore of very coarse material, the flow of cold water from the underside of the ice sheet towards the ice free interior of the coarse material will cause thinner ice at the boundary. Horizontal ice movements result from inadequate lateral support along the edges of the ice sheet or from buckling failure of the ice sheet. Any tendency for the ice sheet to move relative to a bridge structure will cause horizontal ice pressure in the sub-structure of the bridge.

In an effort to establish realistic design criteria for ice pressure, an extensive program of field observations of ice movements and related physical conditions was undertaken. It was considered desirable to record ice movements to one tenth of an inch and a fix by triangulation from shore would not be of sufficient accuracy. Control points on the ice were therefore arranged along straight lines across the ice sheet. Each line was defined at the ends by fix-points on shore. The control points on the ice consisted of nails placed on timber horses securely imbedded in the ice sheet. These are shown on Fig. 8. By placing an instrument over one fix-point and sighting the corresponding point across the lake, it was possible to observe movements of the intervening control points. It was not possible to measure the actual movement, only the component perpendicular to the line; but the error in measurement was small.

The ice sheet at the Highway crossing is bounded about 1000 ft. to the

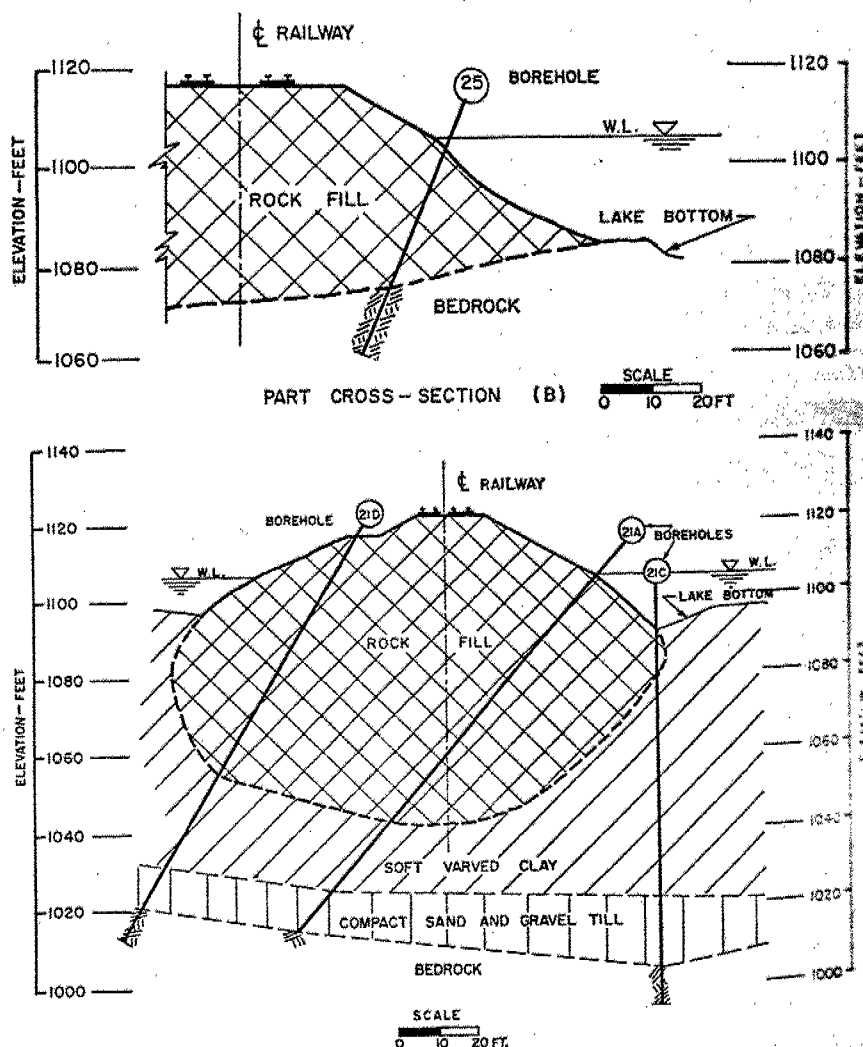
south by the railway embankment, but to the north there is an open expanse of about eight miles. At a very early stage the ice sheet forms a pressure ridge north of the causeway alignment approximately along the middle of the open expanse, and this appears to be an annual occurrence. A photograph of the ridge is shown in Fig. 9. As might be expected, therefore, the ice movements at the causeway centre line are initially found to be towards the north and away from the railway embankment. Northward movements are small, never exceeding ½ in. They cease early in January, presumably because the ice gets too heavy to allow relative movement at the ridge.

For the remainder of the winter the ice sheet moves south towards the railway embankment corresponding to a general expansion of the ice sheet. At the causeway centre line location, movements are about one inch per month reaching a total of three inches at mid-March, about which time the ice sheet starts to melt. The accumulative ice movements at mid-

March are indicated on Fig. 10.

The southward ice movement is believed to be caused by reduced lateral support of the ice sheet along the railway embankment. Control lines along and close to the embankment indicate a total ice movement of 4 to 5 in. At the railway bridges, where the current maintains small openings in the ice sheet local movements may be larger. It is believed that the reduction in support along the rock fill embankment is caused by the retarded growth of the ice sheet along the embankment as mentioned above. At any time the ice thickness elsewhere is found to be very uniform across the lake and it reaches 36 to 40 in. When warm weather arrives at about the middle of March, melting starts, not only on the top and bottom but also inside the ice sheet forming water-filled holes in the ice. The ice loses strength as it gets increasingly honeycombed and by the time the ice sheet breaks up it is very weak. A serious condition of drift ice is thus prevented.

FIG. 7: Cross-Sections through existing Railway Embankment.



Design

The final design was determined by the

1. The east bankment causing the ice to move along the side of the embankment, creating relative motions and destruction;
2. Because of the clay, how stable the embankment was in the long run, and the way in which it would be achieved by the clay. In a safe alignment at a safe way embankment;
3. Maneuvering and the layout between the way and the





FIG. 8: Timber Horses for Measuring Ice Movement.

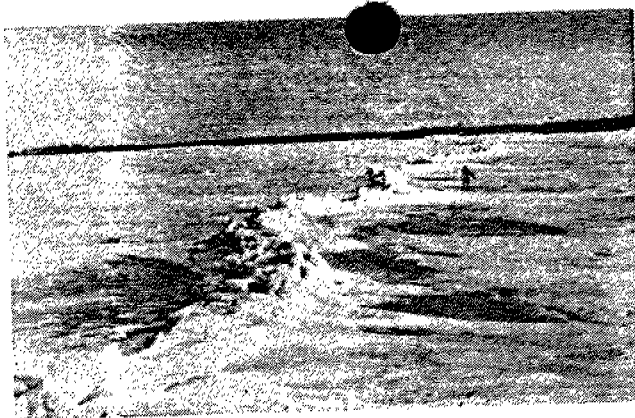


FIG. 9: Ice Ridge on Rainy Lake.

Design

The final alignment was determined by the following considerations:

1. The east half of the railway embankment has performed well. Locating the Highway embankment alongside it, would take advantage of relatively favorable soil conditions and would facilitate construction;
2. Because of considerable depths of clay, however, construction of stable embankments for the Highway in the west half of the crossing would require dynamite blasting to achieve displacement of the clay. In addition, the causeway alignment would have to be kept at a safe distance from the railway embankment;
3. Maneuvering room for navigation and the logging operations between the Highway and the railway was required;
4. The final alignment provides a short 2.8 mile crossing, compared to 3.2 miles for the railway embankment.

The placing of the navigation span and logging span side by side was decided on because it would require a minimum length of elevated bridge. Slight asymmetry in the general profile of the High Level Bridge was adopted to effect an appearance of continuity between high and low level bridges.

The location of the transition points between embankment and bridge were firstly dictated by soil conditions, because it was not considered economically feasible to provide stable embankments where the thickness of clay exceeded 20 to 25 ft. Since the greatest thickness of clay occurred where the water was deep, however, the final project also showed approximate equality in cost of embankment versus structure at all points of transi-

tion. The cost of embankment islands was estimated on the basis of winter haul, since the ice sheet on the lake allows transport of fill over ice roads.

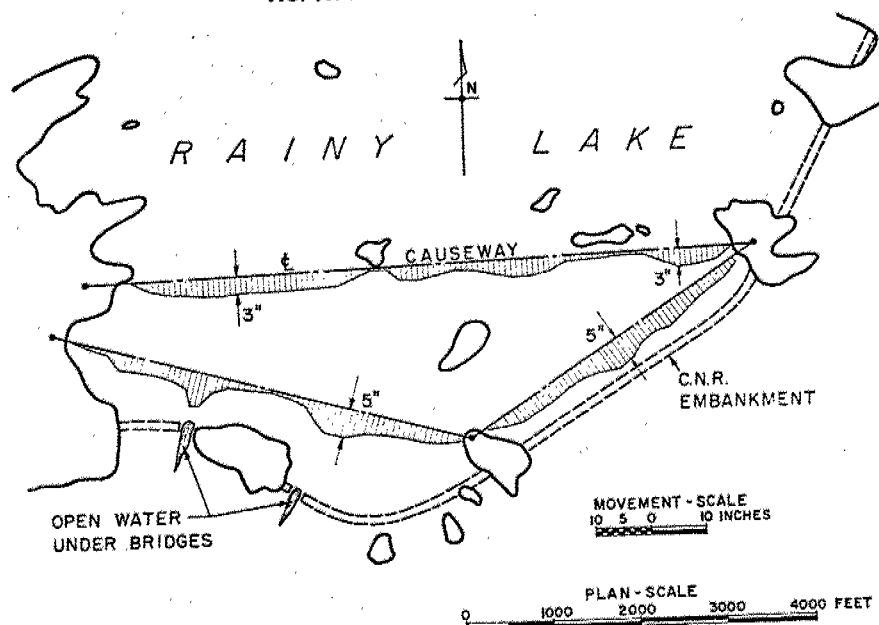
EMBANKMENTS

The causeway is 44 ft. wide at road level and has a free board of 10 ft above high water level. It was assumed that overall side slopes of 1.5 horizontal to 1 vertical would be developed during construction.

Since the clay over much of the site is normally loaded and sensitive, it could not generally safely support more than about 10 ft. of rock fill on its surface. For end-dumped embankments of the heights that were required, sinkage of the rock fill into the clay was therefore expected as a general condition. It was also considered that such sinkage would occur for the most part by progressive sliding at the face during dumping. Preliminary estimates of the depth of sinkage of the rock fill into the clay were made. These were based on the assumption that sinkage would occur until the ultimate bearing capacity of the clay at a given elevation was equal to the applied load of the fill.¹ The actual sinkage of the rock fill railway embankment was checked by borings, and it was found that the sinkages computed as above were generally less than those which occurred in practice.

Based on the estimated sinkages, preliminary settlement computations were carried out to check on the amount of consolidation which would take place in the undisplaced clay under the embankment. These showed that settlements of the roadway at the centre line of up to about 5 ft. could be expected. The analyses also showed that only about one half of the anticipated settlement would be completed in the first five years after construction. These results indicated that with partial displacement only, the performance of the embankment

FIG. 10: Ice Movements on Rainy Lake.



would not be satisfactory. It was, therefore, considered essential to completely displace the clay from beneath the new rock fill embankment, particularly in the areas of the structural abutments. Where the clay was less than 8 ft. thick, it was estimated that adequate displacement could in general be achieved if a continuous rate of end dumping was maintained. In the event that incomplete displacement occurred along the sides of the embankment it was felt that this could be improved by toe shooting after the initial fill pass.

To effect positive displacement where clay depths were greater than 8 ft., however, it was considered essential to lower the shear strength of the clay to about 100 p.s.f., or less, in advance of fill placement. Field tests were carried out to determine

the required degree of remoulding could be produced in the clay stratum by blasting. The tests were carried out from the ice at locations where the in-situ undrained shear strength of the clay varied from 400 to 500 p.s.f. For the blasting, 50% Forcite ditching dynamite was used. The strength of the clay, both natural and remoulded, was measured using a penetration type vane.

In all, test blasting was carried out at five locations.

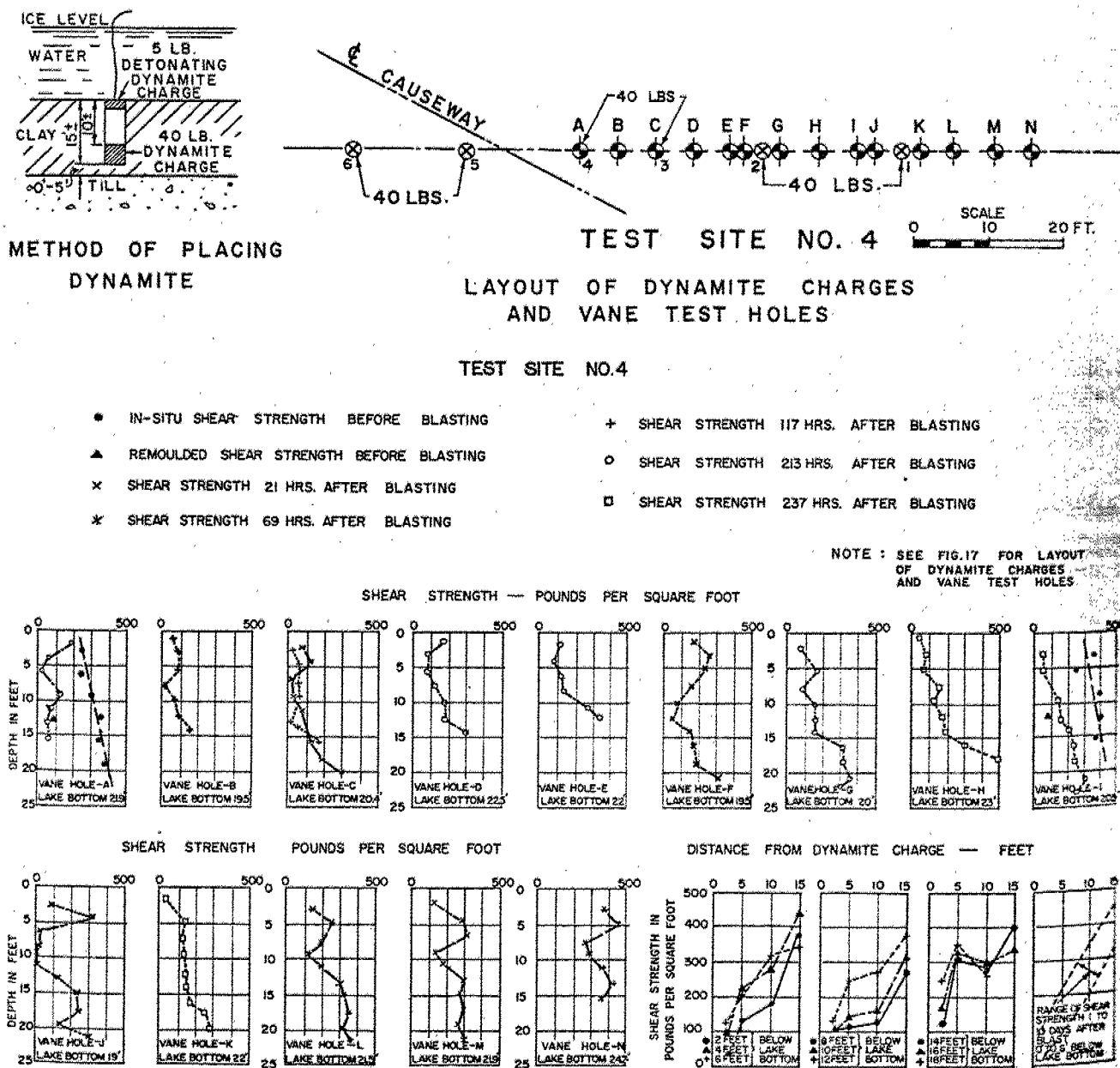
The results of one of the tests, expressed as shear strength profiles are given on Fig. 11.

The rate of regain of shear strength of the remoulded clay was checked by vane tests taken up to 10 days following blasting, and some results are given on Fig. 11. The measurements indicated that there was gen-

erally no appreciable regain in shear strength of the clay for at least several days after remoulding by blasting. The test results also showed that maximum remoulding was effected by the combined effect of a group of concentrated charges placed near the base of the clay stratum. On the basis of the results obtained it was concluded that a powder factor of one pound of dynamite for each cubic yard of clay was necessary to effect a lowering of the shear strength below 100 p.s.f.

Since the remoulding of the clay by blasting was a major part of the embankment construction it was important to find a practical and economical way of loading and detonating dynamite in the clay fast enough to keep up with the fill operation. Several methods were tried, and

FIG. 11: Results of Blasting and Vane Tests, Test No. 4.



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FIG. 12: Jetting of Dynamite into Clay Below Lake Bottom.

after considerable experimentation it was discovered that it was feasible to water jet sticks of dynamite into the clay. This method was only found to be satisfactory when the jetting pipe was between 1½ to 2 in. in diameter and the jet pump powerful enough. The successful test set-up consisted of a 2 in. sectional jet pipe and a 4 in. four-stage centrifugal pump developing up to 200 p.s.i. pressure with an output of 600 gallons per minute. See Fig. 12.

By using ditching dynamite it was possible to effect explosion by propagation and a whole pattern of dynamite pockets with proper spacing and loading could be discharged by a single "primer". It was found that 50-pound pockets of dynamite would explode by propagation if spaced as much as 18 ft. apart. In practice a 10 ft. spacing between 50-pound charges was recommended.

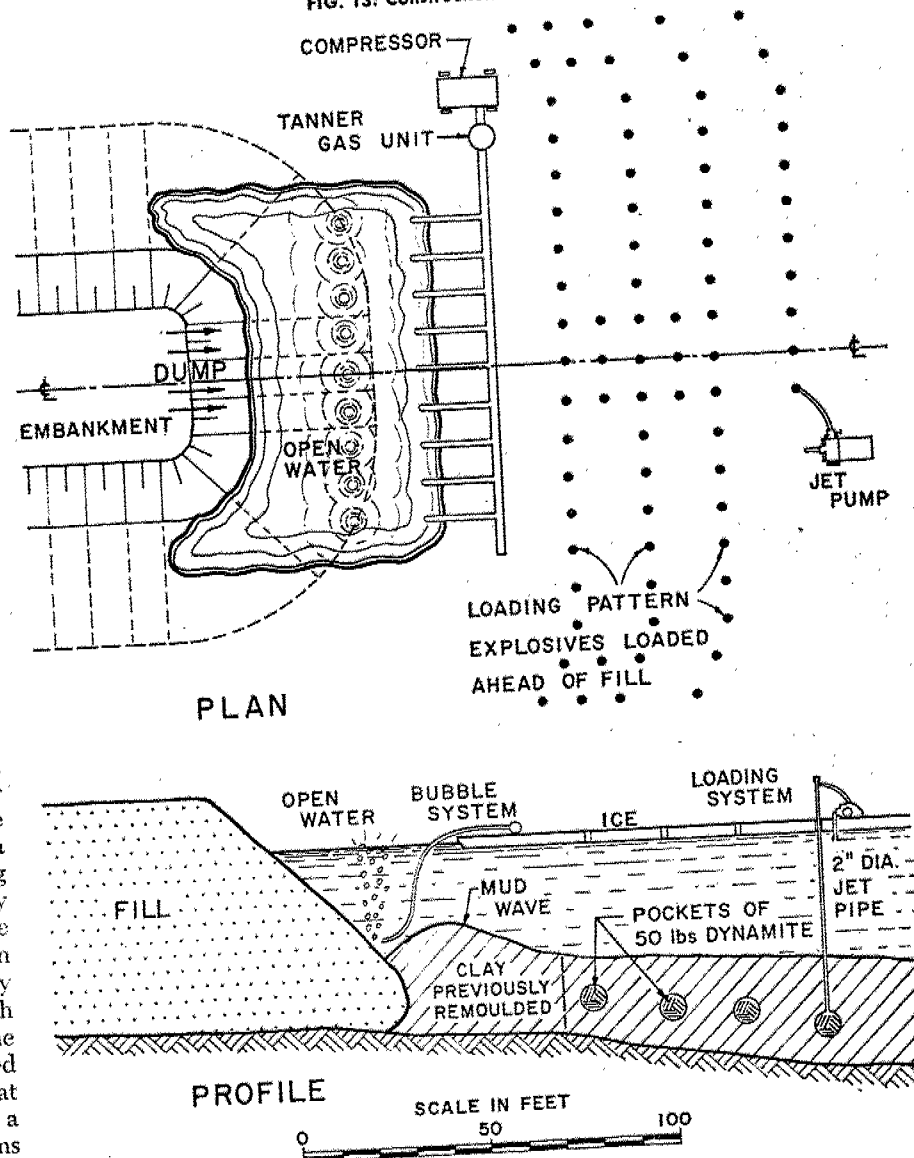
The specifications for embankment construction called for continuous end-dumping of rock fill to final grade. Where the clay layer in the lake bottom exceeded 8 to 10 ft. in thickness the specifications called for remoulding of the clay ahead of the fill by dynamite blasting using a loading of one pound of ditching dynamite for each cubic yard of clay to be remoulded. Dynamite was to be placed in pockets of not less than 50 lb. per pocket at a spacing one way not exceeding 10 ft. and at a depth in the clay equal to two-thirds of the clay thickness. It was recommended that each blast contain a total of at least 3000 lb. of dynamite. As a special precaution, the specifications also provided for toe shooting along

the sides of the embankments if this was deemed to be necessary to complete displacement in local areas after the initial fill operation. Figures 13 and 14 show typical details of embankment construction. When the ice thickness is substantial, the centre of the fill can generally break through the ice during dumping but the sides of the embankment tend to get hung up on the ice sheet. It is, therefore, advisable to clear the ice ahead of dumping. In Rainy Lake it was possible to do this by melting the ice using an air bubbling system shown schematically on Fig. 13.

BRIDGES

Considerations of ice pressure and ice movement had a very important influence on the final design of the bridge structures. Based on ice obser-

FIG. 13: Construction of Embankments.



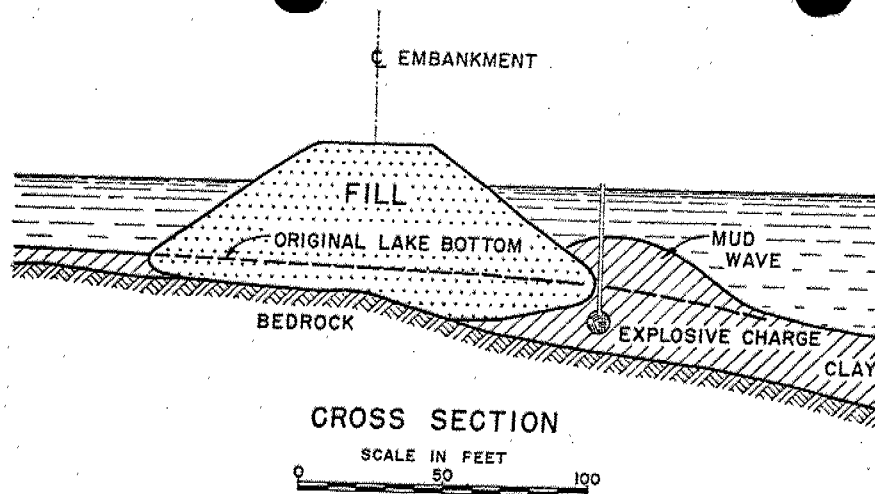


FIG. 14: Shooting at Toe of Fill.

vations, it was concluded that piers or pile bents must either have enough lateral strength to resist an ice thrust of 5L to 10L tons, where L indicates the spacing in feet, or else be capable of adopting most of the annual ice movement up until mid-March.

The following detailed observations were pertinent:

1. In Northern Ontario and Quebec it is usual to use 20,000 lbs/foot for ice thrust in design of rigid structures;

2. Reduction in potential ice thrust due to a small expansion of the ice sheet is negligible. This was confirmed by "liberating" an area of ice from the lake at low temperature and measuring its immediate expansion, which was found to be one inch in 380 ft. At the bridge site, which is located on the periphery of an eight mile stretch of ice cover, the stress relief offered by a few inches of lateral movement will not greatly reduce the latent thrust of the ice sheet;

3. The relative flow of the ice sheet around and past a pile is negligible unless the ice pressure is very high, because such a flow will require very large differential movements in the ice. This was confirmed by jacking between two test piles, see Fig. 15. Pointed or streamlined shape of the pile section at ice level will not improve the flow of the ice sheet. (This is distinct from bridge piers in rivers which are often plow-shaped to prevent jamming of ice floes.);

4. During the spring when the temperature of the ice sheet approaches 32°F throughout, melting of the ice in the pressure zone at front of the piles will arrest the growth of pressure between piles and ice. At this time it was found that the two test

piles could be jacked apart by exerting very low jacking pressure. The test piles are shown on Fig. 15. The brackets extending from the piles are fitted with lugs. The distance between the lugs was measured by micrometer in the manner shown on Fig. 16. By adjusting these readings for temperature changes in the brackets, it was possible to determine accurately variations in the distance between test piles;

5. Variations in annual ice movements are probably small. A comparison of different years indicates that movements depend on the condition of the shore and on a certain constant condition of stress and plasticity in the ice sheet.

Daily records of weather, water level and water outflow from Rainy Lake were available from the year 1947. The temperature curves from mid-December to mid-March for the years on record were compared and it was found that accumulation of any of the pertinent aspects of the curves showed little variation over the years.

During the winter the inflow to Rainy Lake is small due to rocky up-land, and a relatively constant outflow is maintained through the Fort Frances Power station;

6. The introduction of rock fill embankments for the highway route may affect the general ice movement. At the east half of the lake crossing where the highway and the railway embankments are adjacent, the movement of the ice sheet will probably not be altered. At the bridge centre line the main southward movement of the ice sheet is undoubtedly restrained by the islands in the vicinity, since these islands consist of solid rock outcrops with steep shores.

By locating the fill embankments south of the islands they should tend to reduce the southward ice movement. Piers or pile bents would tend to restrain and reduce the "plastic" ice flow;

7. An artificially-created opening in the ice sheet will produce ice movements. Large ice movements caused by man's interference are not likely to happen due to the difficulty of maintaining a substantial area ice free for any length of time.

In places along the centre line the depth to solid bottom reaches 100 ft. The initial study of a bridge on solid piers was not pursued very far because the large weight required to resist overturning from the ice thrust indicated that this solution would be uneconomical.

The following alternatives received closer study:

1. A floating bridge on cellular concrete pontoons, which would adopt the ice movements;

2. A bridge with light superstructure on bents of vertical piles with mounds of fill around each pile bent for lateral support. The supporting fill to be conveyed in place from the completed bridge;

3. A flexible bridge with medium light superstructure on bents of vertical piles flexible enough to adopt the total ice movement.

The flexible bridge scheme was selected because economy was the main consideration and the following design criteria were adopted:

1. Each pile bent to allow a minimum deflection of 4 in. at lake level;

2. Each of the three centre bents in the High Level Bridge which define the navigation and logging spans to develop at least 200 ton resistance to ice thrust;

3. Each other bent in the High Level Bridge to develop at least 100 ton resistance to ice thrust;

4. Each bent in the Low Level Bridge to develop at least 50 ton resistance to ice thrust.

These requirements were met by using vertical concrete filled steel

FIG. 15: Simulated Ice Pressure on Piles.

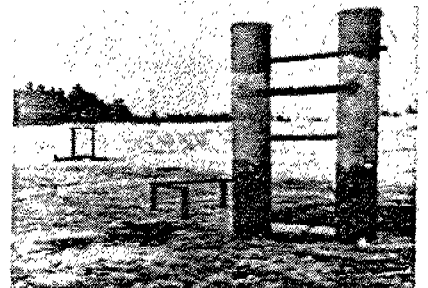




FIG. 16: Checking the Distance Between Test Piles.

be piles at all bents, but varying the diameter of the piles from 16 in. to 24 in., and the number of piles at the bents from 12 to 90.

Almost all pile caps and girders in the superstructure consisted of precast prestressed concrete. For economy and easy erection only two basic units were used throughout.

The pier cap unit was box-shaped with compartments. By filling a different number of compartments with low grade concrete after the pier cap was erected it was possible to provide desired individual loading of the pile bents.

In the design of the pile bents the bearing strength of the soil was taken as 400 p.s.f. and the point of contraflexure was determined by methods suggested by Rowe² and Minikin³. Formulae for the deflection and maximum fibre stress for the piles was developed from first principles.

The flexibility and lateral strength of the completed pile bents were also tested during construction. See outside front cover. The lateral loading tests were carried out during the

winter and the ice sheet was used to jack against. The piers to be tested were completely isolated by cutting a one foot wide slot in the ice around each one. Hydraulic jacks were placed in the slot and the bearing edge of the ice was reinforced by steel beams frozen into the ice prior to the test. The load against the piers was determined by the hydraulic pressure. Movements were measured at several points on the pier by sighting with instruments located on adjacent piers. One of the test piers is shown on the outside front cover. Due to the flexible nature of the pile bents special attention was given to the design of the bearings and the anchor pins were extended to prevent the girders from "pulling out". This may be seen on Fig. 17, which shows a model of an expansion bearing.

ABUTMENTS

At the transition points between the embankments and bridges it was not possible to carry out the clay remoulding ahead of the embankment because this would destroy the lateral support provided by the clay to the piles of the bent adjacent to the abutment. At these points therefore a special treatment was required as shown on Fig. 18. This involved stopping blasting 50 ft. from the pile bent adjacent to the abutment, and the provision of a gravel stabilizing berm as shown.

To facilitate driving of piles for the abutments, the embankment fill was changed to gravel at the abutment locations. The gravel embankments were protected by rip-rap.

ACKNOWLEDGMENTS

The writers are indebted to H. W. Adcock, Assistant Deputy Minister, Engineering, Department of High-



FIG. 17: Model of Typical Expansion Bearing.

ways, Ontario, for permission to publish engineering data from the Rainy Lake Causeway Project and for reviewing the manuscript.

The following organizations kindly made their local experience and records available: The Canadian National Railways; The Government of Canada, Department of Transport; The Hydro-Electric Power Commission of Ontario; The International Board of Control for Rainy Lake; The Minnesota and Ontario Pulp and Paper Company Ltd.; Canadian Industries Limited; Weather Bureau, U.S. Department of Commerce.

Of the great number of people who contributed to the planning of the Rainy Lake Causeway project at the early stages we wish to acknowledge: From the Department of Highways, Ontario, W. A. Clarke; F. C. Brownridge; A. M. Toye; R. Panter; F. I. Hewson; F. B. Whiteley; H. H. Greenly; G. K. Hunter.

From the Canadian National Railways: B. Chappell.

From Geocon Ltd.: J. Morgan.

From Foundation of Canada Engineering Corporation Limited: W. E. Hickey; H. W. H. Casper; R. W. Crudge; G. M. Orzechowski.

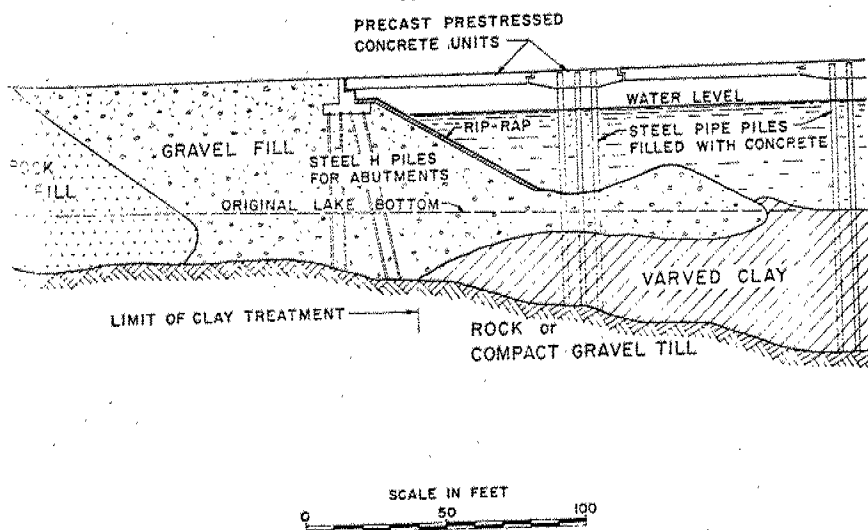
A special acknowledgment is made to W. G. Noden, M.P.P. Rainy River for his interest and assistance.

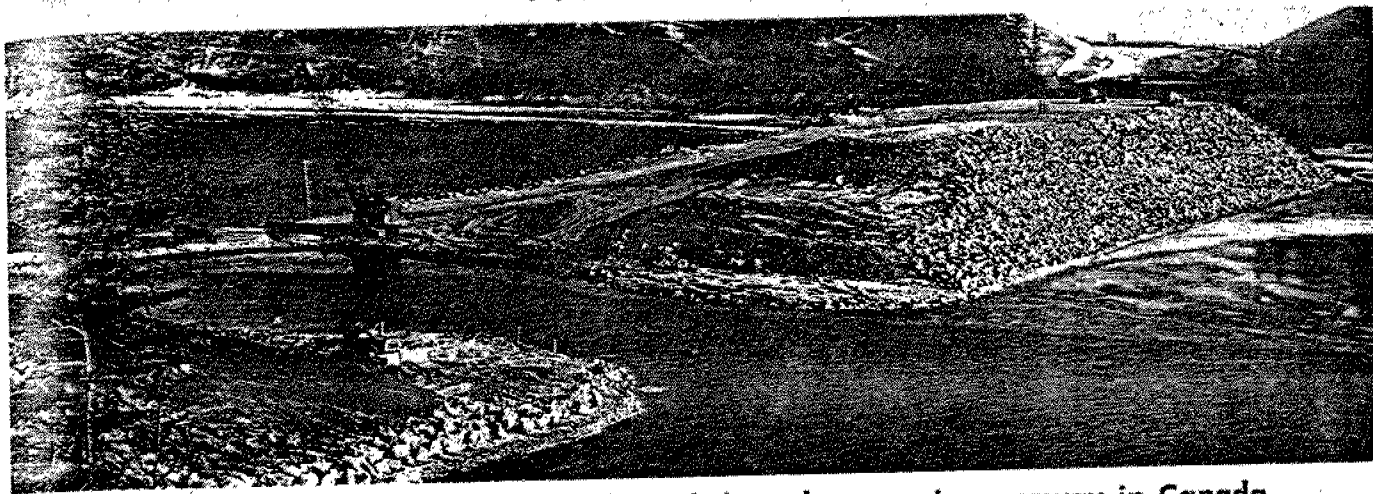
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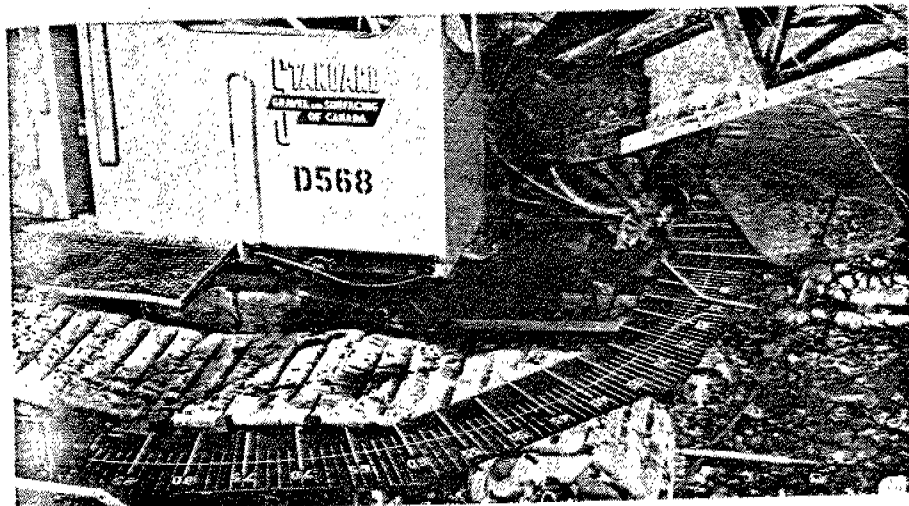
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FIG. 18: Typical Profile at Abutments.

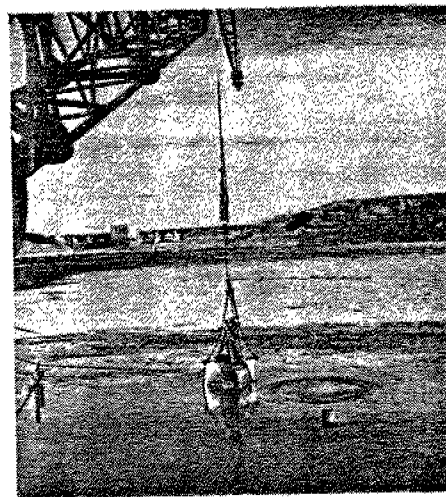




Rip-rap will protect sides and bottom of channel through reservoir causeway in Canada.



Grant protractor helps crane operator to spot rip-rap.



Bubble marks diver's location.

Rip-Rap Hand Placed Under Water

Standard Gravel & Surfacing of Canada, Ltd., is individually placing 10,000 cu yd of rip-rap (minimum size, $\frac{1}{4}$ ton) under water in construction of a 700-ft causeway and bridge across a reservoir in Calgary, Alberta.

"We wanted maximum stability (in the causeway fill)," said a spokesman for consulting engineers Haddin, Davis and Brown Co., Ltd. of Calgary. "There's a high rate of flow through the reservoir and a loose silt bottom. The flow passes through a 140-ft-wide channel in the middle of the crossing."

"After extensive hydraulic model studies, we were convinced that the best answer lay in specifying large, selected rip-rap to cover the channel sides and bottom, and individual placement."

An eight-man crew of Universal Diving, Ltd. of Vancouver puts in a 20-hour day on the project. Each of four hard-hat divers works alone during 2½-hour shifts.

City engineers lowered the level of the water in the reservoir by 14 ft, but divers still must work in about 28 ft of water. Visibility is rarely more than a few inches, and divers must check placement of the rock entirely by feel.

Each rock (some weigh as much as 2 tons) is lowered to the reservoir floor by a rock grapple, then spotted by the crane operator, who follows telephoned instructions from the diver. Every rock touches the next, the only openings in the solid surface resulting from natural irregularities in the shapes of the rocks.

The channel is kept ice-free by compressed air. The air is directed into the channel through a submerged 1-in. hose, graduated to $\frac{3}{4}$ in. by means of a standard reducer. The hose has holes of $\frac{1}{8}$ in. diameter every 4 ft and is weighted with $\frac{1}{2}$ in. cable.

The causeway itself is built of 500,000 tons of screened gravel, $\frac{1}{4}$ in. minimum size. 28,000 cu yd of 4-in. to 24-

in. rip-rap will line the sides of the gravel fill. In addition to the heavy rip-rap on the channel sides and bottom, plans also call for a 9-in.-thick revetment slab of 3,000-psi concrete through the channel area.

A six-lane, single span steel bridge will carry the highway over the channel.

A major problem facing the contractor was finding a way to move fill material from the borrow pit on the west side of the reservoir to the east side, according to Arnold Berg, Standard Gravel's project superintendent. The only connection was over a narrow bridge at the dam site.

The contractor solved the problem by building a road on a temporary gravel fill across the new channel, where the permanent bridge will go. A 54-in. culvert in the fill handles the flow from the north to the south side of the reservoir.

The project is scheduled for completion in October.

214-11. BASIS OF PAYMENT (CONTINUED)

- (a) For the compacting units employed, payment will be made at the applicable unit price per hour as specified on the Tender or at the negotiated unit price per hour for approved special equipment and such payment shall be full compensation for furnishing motive power for the compacting units satisfactory to the Engineer, all labour, repairs, and all work subsidiary and incidental thereto, for which separate payment is not elsewhere provided.
- (b) Payment for water used as directed will be made at the Contract unit price per thousand gallons.
- (c) Payment for shoulder drain excavation will be made at the Contract unit price for "Earth Excavation."
- (d) Payment for shoulder drain backfill will be made in accordance with the specification for the material so used.

When stage, berm or surcharge construction is specified on the Contract drawings, the work shall conform to all requirements specified herein and on the drawings, and payment as herein provided shall be compensation in full for all work required and for all costs arising from, and all delays due to, the controlled placing of the embankment materials.

SECTION 215. ROCK EMBANKMENT CONSTRUCTION.215-01. DESCRIPTION.

Rock embankments shall be constructed by end dumping. The materials shall be well distributed as far as practical, so as to fill the voids and form a solid embankment. The embankment shall be constructed to full width and true to cross-section as the work progresses. No dumping over the side of embankments will be permitted. No rock placed in the embankments shall be greater in maximum vertical dimension than one-third, or greater in maximum horizontal dimension than one-half of the depth of the section of the embankment in which the material is placed.

Advancing ends of rock embankments shall be kept to a concave face with the shoulder corners well in advance of the center. The rock shall be dumped on the surface of the embankment and pushed forward with a bulldozer or like unit. This method will aid in maintaining a surface dense embankment true to grade and with uniform slopes.

Voids on the top of the embankments shall be chinked with small rock fragments.

The rock embankment chinked up shall be up to grade.

John F. Memorial Causeway, Corpus Christie, Texas — Just heard of the name, & have no information.

GA 7103—20.83

E 6

Stability of slopes loaded over a finite area
Karafiath, L. L. / Nowatzki, E. A.

Highw. Res. Rec. No. 323, 1970, pp. 14—25, 11 fig., 2 tab., 5 ref.

In frictional soils, when pore pressures are negligible, failure occurs in shear zones. In such cases the stability of slopes loaded over a finite area can be analysed by bearing capacity methods. Two methods of analysis are presented: an approximate method, based on the assumption that a slip-line field analogous to the Prandtl solution for horizontal ground applies, and a numerical method, based on the numerical integration of the governing differential equations of plastic equilibrium. A formula for the N_q value is given for the slope angle, friction angle, and the angle of the inclination of load and surcharge as principal variables. The concept of stress gradient, which expresses the rate of increase of the bearing stress from the edge of the loaded area, is used to account for the effect of the weight of the soil. The stress gradients obtained by the approximate method are compared with those determined from bearing stresses obtained by numerical integration methods. Results of small-scale experiments are presented showing that the decrease in bearing capacity with slope angle can be reasonably estimated by the approximate method. The methods apply to the stability analyses of highway embankments as well as to problems in land locomotion theory. (AA)

non-cohesive soil / embankment / slope / stability / analysis / bearing capacity / critical surface / model test / theoretical (ED) / E

GA 7012—16.35

E 6 / H 4

The magnitude and cost of minor instability in the side slopes of earthworks on major roads

Symons, I. F.

Rd. Res. Lab. Crowthorne, RRL Rep. No. 331, 1970, 19 pp., 3 fig., 2 tab., 2 ref.

This report gives the results of a study undertaken to assess the magnitude and cost of the problems of minor instability in major roadworks. The results of the study indicate that the instability is mainly confined to short lengths of relatively deep cutting and high embankment and is closely related to the soil types and local drainage conditions. The annual cost of correcting instability on motorways was small and ranged between £180 per km on the earliest lengths to less than £50 per km for the more recent lengths. The annual expenditure on instability in the side slopes of trunk roads was found to be negligible. Whilst it is shown that the problem is not sufficiently serious to warrant any fundamental change in the current methods used for slope design or construction, the study does indicate that particular care is necessary in the design of drainage installations and in the construction of slope profiles in cutting areas.

highway / embankment / cut / slope / stability / drainage / costs / E

GA 31.32

H 4 / D 6

Special tests for design of high earth embankments on US-101

Hall, E. B. / Smith, T.

Highw. Res. Rec. No. 345, 1971, pp. 90—99, 9 fig., 2 tab., 6 ref.

New highway routes through the mountains of California will necessitate the construction of several embankments approaching 400 ft in height. The testing for use in design of high embankments involving soil-rock mixtures cannot be accommodated in conventional triaxial equipment. In an attempt to simulate embankment stress conditions on materials that were of the same types and sizes as contemplated for the prototype, large-scale triaxial tests were conducted on 12-in. diameter by 28-in. high specimens of minus 3-in. material at confining pressures up to 125 psi. Higher confining pressures to 400 psi were also used but these specimens were approximately 6 in. in diameter by 14 in. high, and the maximum particle size was 1½ in. Tests were conducted to obtain both total and effective stresses. Degradation of the materials under compaction, consolidation, and shearing was determined. (AA)

highway / earthfill / rockfill / shear strength / consolidation / triaxial test / failure / equipment / test procedure / construction / case history (embankment / design)

GA 7004—05.52

H 4

Theory and practice of embankment construction
Dammbau in Theorie und Praxis
Striegler, W. / Werner, D.

Wien-New York: Springer Verlag, 1969, 462 pp., 253 fig., 438 ref.

Comprehensive and complete illustration of the complex field of embankment construction. Embankment construction materials: differentiation between rocks and soils and their grouping according to particle-size distribution and cohesiveness. Suitability of rock and soil for embankment construction, design principles. With reference to construction management the most varied influences on the execution of construction work are explained. For questions of the design and construction of embankments the stressing and the bearing capacity of the embankments and their subsoil are examined. This results in the design and making safe of slopes and also stability investigations. The embankment/subsoil system is discussed in detail. The similar problems of embankment-type dams and the special principles governing the design and construction of embankment-type dams are also explained. In accordance with theoretical principles and those relating to building materials, questions relating to the construction of embankments are then discussed, such as preliminary construction work, site installations, methods of founding embankments, excavating, loading, transporting, placement and compaction of the building materials, and sealing of embankment-type dams with natural and artificial materials. In conclusion, hints on the control of the construction and the test methods to be applied.

dam / embankment / stability analysis / construction / text book / well documented (ED)

GA 7004—05.14

E 6

Sliding of embankment and cutting slopes

Les glissements de talus routiers — Etude des désordres observés entre 1963 et 1967

Pilot, G.

Rev. Gén. Routes 40 (1970) No. 451, pp. 41—46, 18 fig.

A working group composed of engineers from the "Administration des Ponts et Chaussées" has reviewed the problem of sliding of embankment slopes. Their study deals with damage noted from 1963 to 1967; 165 index-cards on slidings were established, recording succinctly main data on: localisation, nature of the soils, description, repair methods and cost. After recalling the organization of such a survey, the different aspects of the recorded slidings of embankment are considered and the conclusions drawn from the survey are studied. It has become possible to emphasize the frequency of slidings of certain clays, and under conditions such slidings occur, to study the role of water, and to stress the consequences of too steep embankment slopes. For each case, the measures implemented to repair the damage have been investigated.

cut / embankment / stability analysis / slope stability / landslide / defects / case history

GA 44.44

H 4

Reinforced earth highway embankment — road 39, near Los Angeles

Chang, I. / Forsyth, R. / Smith, T.

Highway Focus, Federal Highw. Admin. Washington 4 (1972) No. 1, pp. 15—35, 9 fig., 6 tab., 6 ref.

A reinforced earth embankment consists of noncohesive soil in which a reinforcement of horizontal metal strips is installed. These metal strips are jointed with steel shells supporting the earth at the front of the structure. The soil of the embankment is required to fulfil the following conditions: the angle of internal friction has to be larger than 25°, the soil particles must not be smaller than 0.074 mm nor larger than 250 mm. A roadway project is described in which a retaining body of 21 m in height, made of reinforced earth, is provided. It is founded on the ground surface with a slope of 1.75 : 1. The soil of the structure consists of sand and gravel, the reinforcement of thin steel strips protected against corrosion, a semi-elliptical shell, 25 to 40 cm high on the fill face. The article describes the stability problems of the subsoil, the design and the calculation of the reinforced earth structure and the measuring devices for checking the method. The movements of the structure are measured, as well as the earth pressure inside the earth body and the movements and stresses of the reinforcements and of the shells in front of the structure.

earthfill / sand / reinforcement / retaining wall / gravel / stability analysis / active earth pressure / stress distribution / deformation (embankment / soil)

GA 56.21

B 3 / E 6 / H 4

An investigation into the failure of a high road embankment at Rickiv, Pietermaritzburg, South Africa

Maurenbrecher, P. M.

Nat. Inst. Rd Res. South Africa, Pretoria, Internal Rep. RS 10/71, 1971, 13 pp., 17 fig., 4 tab., 15 ref.

This report relates the site investigation carried out by the National Institute for Road Research and the Natal Roads Department, at Rickiv fill failure. The background to the failure and the geology of the area is given. Only after a further failure occurred in the natural slope adjoining Rickiv was a satisfactory explanation found as to the cause of the failures. It has also been found that the whole escarpment between Pietermaritzburg and Hilton is subject to gigantic slips. The report finally assesses the stability of the reconstructed fill at Rickiv and gives further suggestions to ensure stability. (AA)

highway / embankment / failure / landslide / site investigation / slope stability / analysis / case history

GA 47.36

H 4

Rockfill

Penman, A. D. M.

Bldg. Res. Stn Garston/Curr. Pap. No. 15/71, April 1971, 10 pp., 6 fig., 1 tab., 26 ref.

Review of the development of construction procedure. Large post-construction settlements of dumped rockfill induce the choice of hard material without fines, and sluicing. Causes of settlements are discussed: Growing load of particles crushes the rock at the contact points and enlarges the contact areas; reorientation of particles. Dependence of settlements on load, particle size, shape of particles, and compressive strength of rock, which is influenced by the water content, effect of compaction. Development of compaction equipment, results of test fills. Settlements of the crests of 7 dams as well as of one road embankment related to time since completion. Rockfill is also advantageous for road embankments, allowing economy of material because of steep slopes and shortening of construction time (no pore pressures). The author recommends measurement of settlements as a basis for design because laboratory tests with scaled-down particle size give strengths that are too high.

dam / rockfill / construction / consolidation / water content / grain size distribution / compaction / equipment / settlement records / state of the art review (ED) (embankment / time factor)

GA 41.53

H 4

Construction of embankments

Highw. Res. Board, Synthesis Highw. Practice No. 8, 1971, 38 pp., 27 fig., 3 tab., 59 ref. More embankment problems are the result of poor foundations than of faulty placement of the embankment itself. In the design of embankments, geometric design criteria and safety standards usually have precedence over other considerations such as soft foundation and poor materials. The design of high embankments should consider the quality of the fill materials because the weight of the embankment is critical. Standard specifications of highway agencies continue to require fill material to be placed in relatively thin lifts and compacted by rolling with suitable equipment. There is, however, a trend toward minimizing procedural specifications and placing greater reliance on density requirements. Moisture content is a continuing problem, particularly with silty soils and swelling clays. Current procedures for building rock embankments are generally satisfactory. A move to thicker lifts may be justified with vibratory compactors. Density requirements, except specifications based on statistical quality control, are considered to be minimum standards that must be exceeded by all field test results. Statistical concepts for density requirements help to evaluate the significance of an occasional bad test. Frozen soils cannot be compacted satisfactorily. Unless the frozen layer of material can be removed from both the embankment and the cut or borrow area, operations should be suspended. For successful earthwork loading was determined for various depths of fill, pipe sizes, and bedding conditions.

highway / rockfill / earthfill / construction / compaction / standard / density / control / statistical analysis / state of the art review (ED) (embankment / specification / design)



Memorandum

To: See Attached List.

From: Route Projects Planning Office,
3rd Floor, West Tower, Downsview.

Attention:

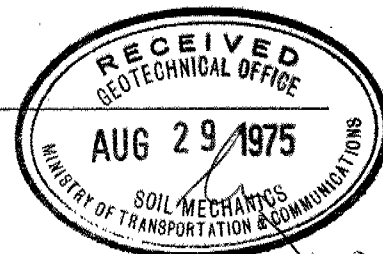
Date: August 22, 1975.

Our File Ref.

In Reply to

Subject:

Highway 17, Feasibility Study
Blind River.



Recently Mayor Venturi of Blind River met with the Minister of Natural Resources and the Deputy Minister of Transportation and Communications and made various proposals regarding the above study. The outcome of the meeting being that we have been asked to investigate further route alternatives; this means that the evaluation by the Internal Team will have to be delayed beyond September, as originally planned, until later in the year.

Our present intentions are to undertake a preliminary analysis of the suggested routes as to whether they should be included in the evaluation, report our findings to Mr. Gilbert, hopefully in four to six weeks and from there continue with the study. You will be informed at that time of our findings and of the revised date for the Internal Team evaluation.

The minutes of the above meeting are enclosed for your information.

Geoffrey Allen,
Project Planner.

GA/js

Encl.

File WP 931-73-00

Files. ✓



Memorandum

To: File

From: Route Projects Planning Office

Attention:

Date: August 15, 1975

Our File Ref. 263-163-3

In Reply to

Subject: Highway 17 Blind River
Feasibility Study
Notes on Informal Meeting
with Mayor O. Venturi of Blind River and the Minister
of Natural Resources regarding the above study

Time & Place - 4:30 p.m. Wednesday 6th August 1975
Minister's Office, 6th Floor, Whitney Block

Those in attendance

Mr. L. Bernier-Minister of Natural Resources
Mr. B. Gilbertson-M.P.P., Algoma Riding
H. Gilbert-Deputy Minister, MTC
C. Meyers-Senior Project Planner MTC
G. Allen-Project Planner MTC

Mayor Venturi presented several proposals for consideration involving both the Ministry of Natural Resources and the Ministry of Transportation and Communications; these are outlined below:

1. The western portion of route C3 be re aligned to cross the Mississagi River to the south of the C.P.R. crossing and from here either (a) follow the south bank of the river to Iron Bridge or (b) cross the C.P.R. and the Mississagi River once again and connect with existing Highway 17 at the closest feasible location.

He was of the opinion that alternative (a) would easily connect with a southern bypass of Iron Bridge and also serve the proposed nuclear power plant should it be located in the Dean Lake area.

Both alternatives would bypass the existing boundaries of Mississagi Indian Reserve.

2. The town would be satisfied with a vertical navigational clearance of much less than 60' as presently recommended by the Federal Dept. of Transport.

3. He indicated his displeasure at the expense involved in the proposed re construction of Highway 555 which is under a connecting link agreement. He wondered whether some defrayment of expenses could be obtained by the Town selling the Ontario Government land it owned on the eastern side of the Mississauga delta area for use as a provincial park.

4. He wondered whether funds would be available from the Federal Government towards the cost of the causeway portion of the route C3 due to its secondary function as a breakwater for the Blind River harbour

A general discussion of the above points followed resulting in Mr. Gilbert promising that the proposals would be investigated by this Ministry

The meeting closed at 5:30 p.m.

Geoffrey Allen

GA/kt

cc: E. McCabe
C. Meyers
P. Dickey
L. Shorr
D. Aspinwall
J. Thompson

G. Allen
Project Planner

Route Projects Planning Office,
3rd Floor, West Tower, Downsview.

August 22, 1975

The attached memo has been sent to the following:

L. Shorr)	
W. Lees)	
D.G. Lowman)	
H. Welker)	N. W. Region.
L. Poste)	
B. McKenna)	
R.W. Belle)	
H. Munford)	
D. Aspinwall	Mun. Eng.	Sault Ste. Marie.
B. Darch)	
C. Meyers)	R. P. P. O.
P. Dickey	Environmental Office.	
N. Close	Maintenance Management.)
C. Sherwood	Regional Transportation Planning.	
A. Laughren	Priority Development.) Downsview
C. Mirza	Soil Mechanics.	
J. Carter	Hydrology.)



Memorandum

To: B. J. McKenna,
Regional Structural Planning Engineer,
Northwestern Region,
Thunder Bay.

From: Route Projects Planning Office,
3rd Floor, West Tower, Downsview.

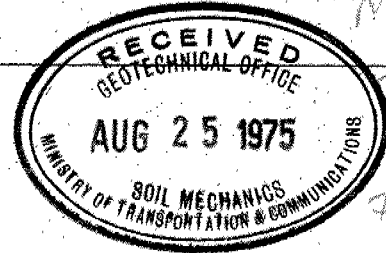
Attention: Thunder Bay.

Date: August 22, 1975.

Our File Ref. _____

In Reply to _____

Subject: Highway 17, Blind River
Feasibility Study.



Recently I attended a meeting with the Mayor of Blind River, the Minister of Natural Resources and the Deputy Minister of Transportation and Communications. At this meeting we were asked whether it was possible to adopt a navigational clearance in the order of 40' rather than 60' due to the fact that the only vessels using the harbour would be pleasure craft. In addition we were requested to investigate the feasibility of a low level causeway with a swing bridge. Hence would you please contact Transport Canada regarding the navigational clearances and also would you comment on the feasibility of the swing bridge proposal?

If either of the above suggestions is to be feasible then further work may need to be undertaken on developing profiles and costing. Hence your comments would be appreciated at the earliest possible date both by this office and Mr. Lowman who will most probably be doing the aforementioned work.

As I indicated in our recent phone conversation, we hope to report back to Mr. Gilbert in approximately four to six weeks on the above proposals.

Geoffrey Allen,
Project Planner.

GA/js

c.c. L. Shorr
D. Lowman
P. Dickey
C. Meyers
D. Aspinwall
C. Mirza ✓
J. Carter

G. Allen, P. Eng.
Project Planner
Route Projects Planning Office

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

August 21, 1975

W.P. 931-73-00

your memo of Aug. 14/75

**PRELIMINARY PROFILE AND
Cross-Sections, -Routes F and C-3
Blind River By-Pass Study
Dist. 18**

I have studied the preliminary concepts and offer the following comments:

1. The reference to "approximate bedrock" on the C-3 profile and low causeway X-section is incorrect and should be deleted.
2. If rock fill is available in abundance for the Line 'F' causeway, then I agree with the last paragraph on your construction notes. I would further suggest that for comparative cost evaluation, the lake bottom and rock fill be assumed to be competent to support the required structure on spread footings.
3. Due to probable scarcity of rock fill for the causeway, I suggest the adoption of the earth core design with one modification to conserve rock fill--viz: elimination of the perimeter rock fill towards the bay or harbour side. The earth portion of the causeway can be built progressively inward and upward starting with the initial perimeter rock fill facing the lakeside.

C. Mirza

c.c. W.L. Lees Attn: L.P. Shorr
D. Lowman
B. McKenna
R. Morgenroth

Files
Record Services



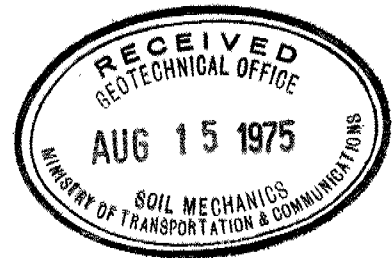
Ontario

Ministry of
Transportation and
Communications

Route Projects Planning Office
1201 Wilson Ave
Downsview, Ontario
M3M 1J8

August 14, 1975

C. Mirza
Head, Soil Mechanics Section
Geotechnical Office
East Building



Dear Mr. Mirza:

Re: Highway 17 Feasibility Study
Blind River

Enclosed please find information on the proposed
causeway treatments on line C3 and line F as below:

1. Profile of the water gap section of Line C3,
showing the grades across the causeway and proposed
structure. The profile also illustrates the cross-
sections through the structure, low causeway fill, and
high causeway fill. You will note 2 causeway treatments
are illustrated, one being the cross-section should
rock fill be used, and the other having rock fill at
the fill perimeters only, and an earth fill core.

2. A profile of Line F showing the causeway
structure across the mouth of the Blind River at Lake
of the Mountains, and a typical causeway cross-section.

3. Notes on construction to accompany the above

Should you have any comments or questions please
contact the writer at the above address.

G. Allen
Project Planner

NOTES

The preliminary design of all proposed causeways and structures was based on preliminary investigations and recommendations by the M.T.C. Hydrology Section and the M.T.C. Soil Mechanics Section, Downsview.

The causeway design elements as indicated on the Line C3 profile will be applied to all causeways contained in any of the route alternatives.

Rock fill material to be obtained from adjoining rock cut sections in the alignment.

Assumed that earth fill will be from Granular Borrow Pit sources north of Blind River (Hwy. 555 Area).

Rock fill will be used to a minimum elevation 2 feet above the established maximum wave height of Lake Huron (Line C3), or 2 feet above the High Water Level of Lake of the Mountains (Line F) (to be established).

Rock fill sections will be constructed to slopes of $1\frac{1}{2}$ to 1, with a minimum 25' top beneath high fills and a 15' top beneath lower fills, and placed around the perimeter of any causeway and across the median near structure abutments.

Structure clearances on Line F to be based on minimums set for recently constructed structure on Highway 557 crossing of the Blind River.

Line F Causeway and structure approaches would in all probability be constructed entirely of rock due to the larger rock excavation requirements east of the causeway. The embankment slopes would then be $1\frac{1}{4}$ to 1 above water, $1\frac{1}{2}$ to 1 below water.





Memorandum

To: L. Shorr

From: Route Projects Planning Office

Attention:

Date: August 11, 1975

Our File Ref. 263-163-1

In Reply to

Subject: Highway 17 Blind River
Feasibility Study



On Wednesday August 6th last I attended a meeting in the office of the Honourable Leo Bernier, Minister of Natural Resources at which Mayor Venturi of Blind River presented various proposals to the Minister concerning his community. In attendance also were Mr. Bernt Gilbertson M.P.P., Mr. H. Gilbert, Assistant Deputy Minister, Ministry of Transportation and Communications, and Mr. C. Meyers of this office.

Of particular concern to this Ministry were Mayor Venturi's suggestions as below:

1. A new alignment in the western section south of the Mississagi River; this would join with a possible southern by pass of Iron Bridge in the north and with C3 in the south (ie W3)

2. A new alignment for the western portion of C3 such that it would by pass the existing 'S' bend on highway 17 to the south and reconnect with existing highway 17 at the closest location (W3B)

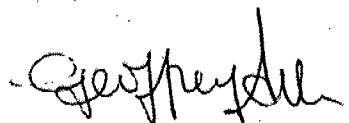
3. The vertical clearance for the structure on line C3 could be lowered to approximately 35 feet.

Mr. Gilbert promised that this Ministry would investigate the above proposals. My present feeling is that we gather all the relevant information on the above routes and present a preliminary analysis to the Minister for a decision on whether to include them in the evaluation. As a first step would you please ask Mr. Lowman if he could investigate the aforementioned routes and a possible second link (W3A) to connect W3 to C1. (This link was not mentioned by the Mayor but I think it should be considered.) In addition representatives of Ministry of Natural Resources and Ministry of the Environment have requested that we investigate a re routing of C1 northerly ie C1'A' as indicated in the enclosed map.

WP 931-73-00

Apart from cost estimates, any comments you or any of your staff have on the route alternatives that would be of value in the preliminary analysis would be appreciated.

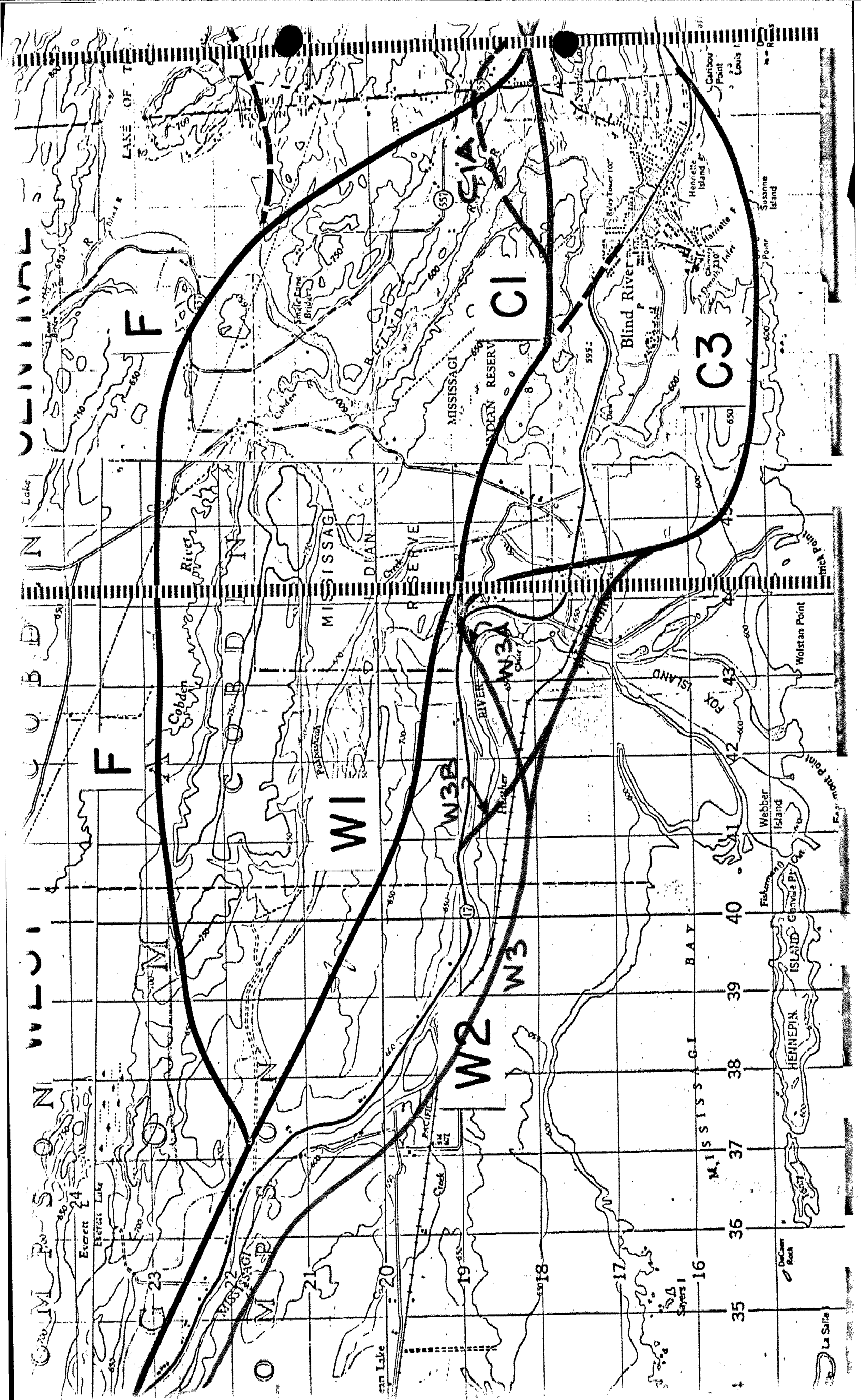
Please find enclosed a map indicated the aforementioned routes.



G. Allen
Project Planner

GA/kt

cc: D. Lowman
P. Dickey
C. Meyers
C. Mirza
D. Aspinwall



Mr. W.L. Lees
Manager, Planning and Design
Northwestern Region

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

Mr. L.P. Shorr

July 25, 1975

W.P. 931-73-00

**FEASIBILITY STUDY - PROPOSED
Blind River "Causeway"
District 18**

Following a meeting in Thunder Bay on May 21, 1975, this Section agreed to a subsurface investigation of lake bottom conditions in the North Channel along proposed Route C-3 ("Causeway" route).

Due to adverse weather conditions and the need to control costs, the exploration program was not as expansive as might have been desired. Therefore, considerable reliance has been placed on geological evidence and reports in order to assess the most probable subsurface conditions along the "Causeway" route within the North Channel.

A report detailing the site and subsurface conditions along proposed Route C-3 (as well as Routes C-1 and F to some extent) will be prepared in due course. The following information should suffice in the meantime to permit rough cost evaluations.

The proposed "Causeway" Route C-3 will encounter water depths of up to 20 ft. and will be underlain by compact to very dense non-cohesive foundation materials such as sands, silts and, at depth, gravels, etc. This foundation material is considered competent to adequately support the causeway fill which will be roughly 25 to 30 ft. in height except at the approaches to a proposed high level bridge where the fill will be up to about 70 ft. in height.

The causeway fill should be constructed of non-cohesive material such as sands and gravels or rock to 2 ft. above the design high water mark. Above this elevation any suitable fill material may be used provided (a) it is protected from erosion and (b) it is properly benched or sloped to ensure inherent internal stability (see Estimating guideline below)

...

ESTIMATING GUIDELINE - CAUSEWAY FILL

<u>Material</u>	<u>Environment</u>	<u>Min. Slope</u>	<u>Compaction Effort</u>	<u>Remarks</u>
Rock Fill	under water	1-1/2:1	Nil	Recommended
Rock Fill	above water	1-1/4:1	High	Not suitable for piled foundations
Earth Fill (non-cohesive)	under water	2:1	Nil	Allow 15-25% for "wastage" during placement
Earth Fill (non-cohesive)	above water	1-3/4:1	Med.	Provide benching to control erosion
Earth Fill (cohesive)	under water	varies	Nil	Not recommended
Earth Fill (cohesive)	above water	2:1 or better	Med.	Ensure inherent stability

In addition to the above, the following points should also be considered:

1. Long continuous slopes should be avoided by provision of benches at suitable heights. Width and location of benches will depend on materials used. For estimating purposes assume a 10 ft. min. bench at any horizontal plane of contact between two different types of causeway fill material. Also assume a 10 ft. min. bench width for any slope greater than 25 ft. in height constructed with cohesive material.
2. Fills should be constructed 6 months to a year in advance of the high level bridge to allow major settlements to occur.
3. Structure abutments could be placed on spread footings provided the entire approach embankment is constructed with rock fill. If insufficient rock is available, the causeway embankment should be zoned from rock to earth fill (non-cohesive below the water) in the vicinity of the structure to permit pile support of the abutments (piles are driven with difficulty, if at all, through rock fills).
4. For structural planning and estimating purposes, assume settlements to be 1.5% of the height of the fill due to self weight after construction to full height. (Settlement of the foundation subsoil will occur during construction. Residual settlements after construction should be negligible.)

5. Driven piles should penetrate at least 40 feet below the lake bottom. Practical refusal is likely to be encountered 40 to 60 feet below lake bottom for end-bearing piles.
6. Sufficient armour by way of suitable rip-rap should be provided to protect against wave action and ice movement damages.
7. Where rock fill is not used in the causeway embankment below the high water mark, the armour protection should extend below the low water mark to the depth of influence of the waves so as to safeguard against sub marine erosion and loss of material.

The above information is to be used for estimating purposes only.
It is not intended for design.

C. Mirza

c.c. G. Allen (3)
D. Lowman (2)
B. McKenna
H. Munford

M. Harris
Record Services

"Please distribute extra 2 copies as you see fit, e.g. Dist."

BLIND RIVER TOWN CLERK 356-2251

JERRY MURRAY, MINISTRY OF NATURAL RESOURCES, BLIND RIVER
356-2234

MINISTRY OF NATURAL RESOURCES, MONTSVILLE

MR. NORM MCGUIRE } 789-4471
MR. PAGET }

KARL CHRISTENSEN, ST. LAWRENCE SEAWAY,
ST. CATHERINES 684-6571

ROCK FILL - COMPOSITION & SHAPE - SLATE, SLABBY
60' FILL ON BEDROCK. SETTLEMENT OF ABOUT 1' OVER
5 TO 7 YEARS - IN HALIFAX.

Failures originating within the embankment
itself may be caused by poor materials,
unsatisfactory construction methods, or ineffective
quality control procedures.

For embankment construction, rock is defined
as a material containing more than 25 to 35
percent by weight larger than $\frac{3}{4}$ in size.

The lift thickness requirements for compacting
soil generally are relaxed to accommodate
the larger particle sizes in rocky fills.
Vibratory compaction of 4 ft. lifts with
3 ft. maximum particles.



Memorandum

To: M. Devata
Supervising Engineer

From: Soil Mechanics Section
Geotechnical Office
West Building, Downsview

Attention:

Date: May 23, 1975

Our File Ref.

In Reply to

Subject:

W.P. 931-73-00
Feasibility Study
Hwy. 17, Blind River Area
Dist. 18

This memorandum outlines the involvement of this Section in the above study.

Background

Mr. Don Lowman, Regional Photogrammetrist, Northwestern Region, has provided us with a 1" to 1000' plan showing Lines C-], C-3 and F. Profiles for each line have also been prepared.

At a meeting in Thunder Bay on May 21st, 1975, the following decisions were reached with respect to these lines:

1) Line C-]

Subsoil conditions within Blind River between Stations 120[±] and 148[±] are not known. However, it was agreed that even if subsoil conditions were extremely poor, the extra cost involved in coping with such conditions would be but a small percentage of the total cost of Line C-]. Hence, subsurface investigation between Sta. 120 and Sta. 148 is to be limited to a geological study only.

2) Line F

This line, crosses open water between Stations 130[±] and 146[±] just north of an east-west trending Hydro power line. The line has been designed to skirt around Indian Reserve #8 (Mississaugi Reserve). At this water crossing location, Blind River exits Lake Duborn (Lake of the Mountains).

Mr. H. Munford will attempt to contact Ontario Hydro for information relative to soil conditions at tower locations in the vicinity of the water crossing. This information should be sufficient for feasibility costing purposes.

...

At both C-1 and F water crossings, Mr. Karl Wright, Regional Engineering Surveys, will carry out soundings by probing. This information should prove sufficient for the moment for purposes of this study.

3) Line C-3

A major portion of the cost of this line depends on subsoil conditions within the North Channel. The Water Crossing lies between Stations 258⁺ and 320⁺. Poor soil conditions would almost double costs on this alignment. It is therefore justifiable to undertake an exploratory investigation at this time.

The investigation will consist of an over-water drilling exploration at three locations, viz: Sta. 263⁺, 296⁺ and 314⁺. More boreholes may need to be drilled if deep clay strata are encountered.

It is proposed to obtain third order coordinates for these drilled locations for use later on in a geophysical survey of the Line C-3 corridor. More boreholes may be necessary in order to obtain sufficient control data for the geophysical survey. The decision to drill more boreholes will depend on:

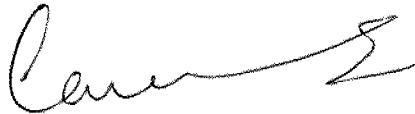
- (1) Progress to date
- (2) Weather forecast
- (3) Subsurface conditions

Plan of Action

1. Mr. H. Shah to be project engineer, assisted by Mr. T. Kazmierowski, student geologist - engineer.
2. Drilling: - Mr. L. Pasquale of Canadian Longyear was in our office on May 22/75. He proposes to build a raft out of six 200 gallon drums. He was given colour aerial photographs to assist him in determining a suitable location for launching of the raft.
3. Mr. Shah to meet drillers at North Shore Motel.
4. Mr. Shah has received three Motorola walkie-talkie sets and three charger units. These are very costly items obtained on loan from Mr. Harry George of Special Services, 7th Floor, West Tower. They are to be returned immediately upon completion of field work.
5. Surveying has been arranged with Mr. Karl Wright. He will arrive in Blind River on Monday May 26, 1975 and discuss our requirements with his party chief, Mr. Murray Slator. Mr. Slator is staying at the North Shore Motel in Blind River.

...

6. Mr. Don Lowman will arrange for the supply of a photomosaic sepia at scale of 1" to 2000' for preparation of our report.
7. Details of report to be discussed later between Messrs. Kazmierowski, Shah, Devata and Mirza.
8. Decision for geophysical exploration to be confirmed, at appropriate time, with Pavement Structure Design Section. Possible Consultant assignment.
9. Mr. Kazmierowski to undertake study of geological information on area and to identify faults for future reference. Field and airphoto studies to be used in mapping geological information pertinent to this feasibility study.
10. Messrs. Shah and Kazmierowski to rendezvous with Canadian Longyear crew on afternoon of May 26th in Blind River, North Shore Motel.



C. Mirza
Section Head

c.c. Messrs. G. Allen
T. Kazmierowski
D. Lowman
H. Shah
K. Wright

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P.O. Box 998,
Blind River, Ontario,
May 13, 1975.

John Rhodes, Minister,
Transportation and Communications,
Queen's Park,
Toronto, Ontario.

Honourable Sir:

The membership of the Blind River Chamber of Commerce unanimously petition the construction of Alternate Route C3 as shown on the Highway 17, Stage 2 feasibility study map, and for the following reasons:

PROXIMITY - Statistics indicate that the nearer a by-pass, the greater the chance of a community's economic survival.

THEREFORE, Route C3 being located along the shores of Lake Huron would

- (1) - skirt the town's southern limits, thus eliminating the distant factor considered an economical deterrent.
- (2) - cause no disruption of established businesses, nor disturb few, if any, residential properties.
- (3) - result in almost negligible costs of property settlement, mainly Crown-owned.
- (3) - eliminate any future possibility of a highway once again bisecting the town with even greater traffic hazards than presently witnessed on highway 17's two lanes. The feasible direction of the town growth being to the northward.

CAUSEWAY, - Granting the cost of construction may appear prohibitive at first glance, its benefits would prove multiple to this no-industry, hard-pressed community.

BECAUSE, the causeway part of Route C3 would

- (1) - act as an efficient breakwater, thus eliminating the recurrent possibility of flooding at St. Joseph's Hospital located on the lake shore.
(In the recent past an emergency wall (sandbags) had to be erected to prevent inundation of the hospital boiler room in the first instance, and the further fear of patient evacuation had the wall not held the waters in check.)
- (2) - replace the existing breakwater which despite federal government expenditures of thousands of dollars to raise and extends remains inadequate for the purpose intended.
(It has been suggested the possibility of federal funds being made available for this project, likewise, provincial savings in property settlements should, at least in part, compensate for causeway costs.)

continued.....

TOURISM - Lacking any form of industry, it would appear that the economic survival of Blind River grows more and more dependent upon tourism.

THEREFORE, Route C3 would

- (1)- via the causeway, provide a most suitable and safe harbouring of lake pleasure craft which presently pass through due to lack of facilities with a resultant and great loss of tourist dollars to the community.
(This year alone an enquiry was made to the Chamber concerning docking facilities for some 150 craft of an American Yachting Club, and it had to be turned down because of lack of accomodation.)
- (2)- the instituting of an exit ramp on the mainland at the western limits of the causeway would allow tourists, not only to visit the town, but gain entrance to beautiful Hamill Park located near the mouth of the Mississauga River at Lake Huron, an ideal tourist spot for fishing and relaxing.
- (3)- Route C3 in its entirety would provide one of the finest scenic drives along the North Shore.

We, the Chamber of Commerce membership, urge that you give serious consideration to the points enumerated in this petition, as we feel the construction of Alternate Route C3 is the one that would be the most beneficial to Blind River, both presently and in the future.

Respectfully submitted,



Ray Sholberg,
Chamber of Commerce President,

c.c. Bernt Gilbertson, MPP
A.A. Wishart
Maurice Foster, MP
MTC Study Team
Blind River Council

Mr. D. Lowman
Regional Photogrammetrist
Northwestern Region
Thunder Bay

Soil Mechanics Section
Geotechnical Office
West Building, Downsview

May 2, 1975

W.P. 931-73-00

W.P. 931-73-00
FEASIBILITY STUDY
Line C-3, Highway 17, Blind River, District 18

This will confirm our telephone conversation regarding the above.

You will prepare for us a plan showing Line C-3 in its entirety. For this Line you will prepare a groundline profile on which you will identify swamp and rock outcrops as interpreted from aerial photographs or already known from work done by Regional Materials & Testing. On this profile, you will superimpose the proposed profile grade. You will also identify, on a plan, potential areas along other routes which may require our attention during the course of this feasibility study.

For our part, I am proposing the following tentative plan of action:

1. Issue survey request to Regional Engineering Surveys for triangulation of boreholes in the bay from shore positions at time of drilling operation.
2. Arrange for supply of suitable drilling machinery and ancillary equipment for offshore drilling at Blind River for a potential commencement date of May 26, 1975.
3. Mr. M. Devata and I will meet you in Thunder Bay on May 21. Mr. G. Allen will accompany us to Thunder Bay for the meeting.

At the May 21 meeting, we will review the plans and profile that you will have prepared by that time, and chalk out a more detailed work schedule for our involvement in this study.

I am proposing to accompany Mr. B. McKenna on a trip to Cochrane via Hornepayne the following day (subject to confirmation by Mr. McKenna).

/

Mr. Devata will then take over the management of our involvement in this project and will be responsible for all future project decisions from this Section for this particular study.

C. MIRZA
Section Head.

c.c. G. Allen
D. Aspinwall
B. T. Darch
M. Devata
B. McKenna
R. Morgenroth
K. Roberts
L. P. Shorr

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Memorandum

To: B. J. McKenna
Regional Structural
Planning Engineer
Northwestern Region

From: Route Projects Planning Office
3rd Floor, West Tower

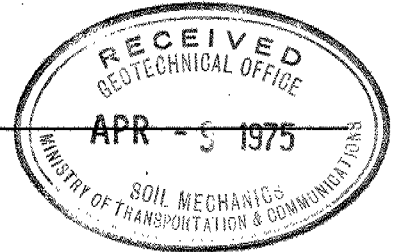
Attention: W.P. 931-73-0

Date: April 7, 1975

Our File Ref. 263-163-1

In Reply to

Subject: Highway 17 Feasibility Study
Blind River



In your memo to Mr. L. P. Shorr of March 14, 1975 you indicated that clearances required for a structure associated with a causeway across the Blind River Harbour would be in the order of 70 feet (vertical) by 40 feet (horizontal). Apparently these dimensions were only approximate and given informally by phone by Mr. Stokes of Canada Transport. As these dimensions are critical to our investigation it is essential that we obtain the actual clearances that would be applied in this location. Hence, would you please make an official request for navigational clearance? Once this information is obtained we can calculate suitable profiles and from these determine accurate costs, aesthetic quality and the various other environmental problems associated with this alignment.

The aforesaid information would be appreciated at the earliest possible date to prevent any delay in the study.

G. Allen
Project Planner

cc. B.T. Darch
L. Shorr
P. Dickey
D. Aspinwall
C. Mirza
J. Carter

GA/md

SUMMARY OF MEETING re BLIND RIVER (W.P. #931-73-00)

Causeway Alternative

PLACE: Foundations Office, Downsview.

TIME: Friday, March 7/75. 3:00 p.m. - 4:50 p.m.

ATTENDANCE:

Mr. C. Mirza)	Foundations Office
)	(Soils Mechanics
Mr. G. Cautillo)	Section)
M.N. Gergely)	Hydrology Office

PURPOSE:

- (a) - Report of progress made to date (M.G.)
- (b) - Evaluation of progress (C.M.)
- (c) - Planned future investigation, and time required (M.G.)
- (d) - Further suggestions and method of approach that M.G. should take (C.M. & G.C.)

(a) - Submitted diary, and suggested that contact with Mr. D.A. McDonald, P. Eng., (Blind River) has not yet been established (awaiting his return call), and that information from him would be potentially very valuable.

- Further time would be required to make any worthwhile report or commitment.

(c) - Open to suggestions, plus follow up previously established leads. Need minimum 2 - 3 days, and would also like to visit Blind River, personally, accompanied by any other interested parties.

(d) - Look further at Hydrology map to get at source of Blind River, and get total Δ Elev. and number of lakes that sediments may be deposited in before reaching Blind River harbour. Evaluate other rivers that may contribute sediments to delta.

- Find out who owns the dam upstream, and **when** go to Blind River, see if they have any silting problems.
- Talk to a Geologist re Blind River (history, ice age, clay origin if expect any such deposits). C.M. expects assumed "Hard Bottom" therefore most of clay would be dumped by then, in upstream lakes, before it reaches Blind River harbour.
- Talk to Geoff. Allen re copy of mosaic for whole route; therefore even if causeway o.k., may have problems in W. end of route re swamp areas, Natural Wildlife Preservation. (Discuss any restrictions to proposed route from environmental point of view, talk with P. Dickie, re Ministry of Natural Resources). (any problem with highway going through swamp). Talk also to Geoff. Allen re feasibility of rest of route, assuming causeway not the factor.
- From Mrs. Porter, get B & W mosaics of area that covers whole route.
- Get profile of grade, and have all of the route photo-interpreted (incl. swamp).
(Introduced to Clerk, Ida Steinberg of Pavement Structure Design Section, who will try to get me any soils information they have of the Blind River area, and/or Hwy. #17 (Soils Profile).

WHAT WE NEED (Information) (According to C.M.)

- (i) Photomosaic of whole C-3 route.
- (ii) Rough profile (incl. swamp), depth of water (lake bottom).
- (iii) Accurate profile (from survey notes).

REPORT SHOULD CONTAIN: (Route C-3)

eg. 7% causeway ——— hard ——— 1 bridge
 9% rock cut ——— soft ———
 2% swamp

- Talk to P. Dickie re Sewage Plant location and implications.
- Any control or effect that swamps have on it?
- Dredging records - any clue to silt and/or sand.
- Note that need another railroad crossing at W. end of route, before re-join exist. Hey. #17.
- Talk to railroad (C.P.R. or C.N.R.) Toronto Office to find Eng. Headquarters for that area. (see if they have any soils information we could use, eg. clay, varved clay, etc.)
- Supposed due date for Mr. C. Mirza is March 19 (extendable if needed).
- Note: We are at "PROBLEM IDENTIFICATION STAGE" ie. want to ask the pertinent questions, and then solve those questions.

Michael N. Gergely

Michael N. Gergely

(Armchair Investigator).

cc: Mr. C. Mirza
 Mr. J.D. Harris
 Mr. J.W. Carter

Mr. J. Carter
Project Hydrology Engineer
Hydrology Section
West Bldg.

Route Projects Planning Office
West Tower - 3rd Floor

March 10, 1975

263-163-1 - WP 931-73-00

Highway 17 Feasibility Study Blind River



As per our recent conversations regarding the causeway proposed (Route C.3) in the above study, it would be appreciated if your office could provide us with the following information:

- i) Minimum to maximum causeway heights and the associated operating conditions. Possibly a recommendation on suitable height based on other causeway experiences.
- ii) Causeway construction details: Side slopes and type of stone required, any associated problems.
- iii) Operating hazards: Details of ice and wind problems on road i.e. how big a problem will these factors be? How often and how long will these conditions apply?

Some of the above questions were answered in your preliminary report, hopefully a more detailed study will confirm this data and give us valuable information for use in evaluating the alternative routes.

gd
G. Allen
Project Planner

GA: lo

cc: B. Darch
P. Dickey
L. Shorr
D. Aspinwall
C. Mirza ✓
B. McKenna

Mr. B. J. McKenna
Regional Structural Planning Eng.
Regional Office
Thunder Bay

G. Allen
Route Projects Planning Office

March 10, 1975

263-1-1
W.P. 931-73-00



Highway 17 Feasibility Study Blind River

At the last Internal Team meeting concerned with the above study, one of the routes presented for comment was a southern bypass of Blind River involving a causeway across the lake (line C3). At that time, we did not anticipate carrying this route further into the study and certain assumptions were made regarding lakebed material, minimum height of causeway and required navigational clearances in order to arrive at preliminary construction costs.

However since that time we have received strong direction from the Blind River Town Council and the Blind River Chamber of Commerce to undertake a detailed investigation of such a causeway, which they see as providing a sheltered harbour for possible tourist boats and cruisers. Hence we are now conducting further study into the causeway proposal as follows:

- i) Lakebed material: The Geotechnical Office (C. Mirza) is presently preparing a report based on available information. Should this report indicate a field investigation is necessary then drill testing and possibly a geophysical survey will be carried out in May once the ice has disappeared in the bay.
- ii) Minimum Height of Causeway: The Hydrology Office (J. Carter) is looking into appropriate heights of causeway and construction details to suit local conditions.
- iii) Navigational Clearances: Apparently Messrs. B. Darch and L. Shorr have already spoken to you regarding the validity of the clearances assumed in the preliminary estimates. If you would undertake to find out the appropriate clearances for the Blind River Harbour and any associated information, it would be very much appreciated.

If you require any further information regarding the Study, please contact either Mr. Len Shorr in your office or myself at the above address.

g.l.
G. Allen
Project Planner

GA: lo

cc: B.T. Darch, P. Dickey, L. Shorr
D. Aspinwall, C. Mirza, J. Carter

Mr. G. Allen
Project Planning Engineer
Route Project Planning Office
West Tower

Soil Mechanics Section
Geotechnical Office
West Building

February 28, 1975

W.P. 931-73-00

**SUBSURFACE CONDITIONS, PROPOSED CAUSEWAY
Line C-3, Vicinity Blind River
Sault Ste. Marie District**

In reference to our recent discussions regarding the above, we have discovered that the ice conditions along the proposed route C-3, south of Blind River in the north channel of Lake Huron, are not favourable in so far as the minimum support strength requirements are concerned for conventional drill testing of subsurface conditions. The following information on the ice conditions was received yesterday from Mr. M. Slator of Engineering Surveys, Northwestern Region: 4 ins. of slush, over 4 ins. of soft ice, over 10 ins. of hard blue ice. Our Office had made tentative arrangements to have a drilling machine available at the site. However, we were informed by the drillers that they are not willing to risk their equipment on 10 ins. of hard ice.

In light of the above, I have arranged with Mr. Jim Carter of the Hydrology Section to give us the services of Mr. Mike Gergely who will conduct an armchair investigation of this area. This investigation will consist of contacting people who might be able to contribute to our present knowledge of the soil conditions in the area, to read-up on the geology of the area, and to search out such other literature and information as may throw pertinent light on the subject.

Hopefully the net result of these efforts will lead to some conclusion regarding the validity of the cost estimate reached by Mr. Don Loman based on a hard bottom condition along the proposed route. We hope to have this information available to you by approximately March 19, 1975.

I suggest that in light of the climatic conditions, we hold in abeyance the decision to pursue with on-site physical investigations until the ice has completely disappeared. By about mid-April, we should be jointly in apposition to assess:

1. Whether in fact the physical investigation is necessary
2. Whether a geophysical investigation, which would cost about \$5,000, would be worthwhile exercise to carry out in light of decisions that may be coming forth between now and the middle of May.

C. MIRZA, Section Head.

c.c. J. Carter

File
Record Services

MEMORANDUM

TO: Mr. G. Allen,
Project Planner,
Feasibility Planning Office,
East Bldg.

FROM: Hydrology Section

ATTENTION:

DATE: February 17, 1975.

OUR FILE REF.

IN REPLY TO

SUBJECT: Re: Proposed Causeway and Bridge at Blind River
WP BW 2602
Hwy. 17, Dist. 18, Sault Ste. Marie

In reply to your request for preliminary data for the above project we have the following information obtained from existing publications and a site visit, on February 12th.

Lake Levels of

Ref: Dept. of the Environment, Lake Levels,
Vol. 1, Inland, 1973, (Thessalon gauge)

Mean water level = 578.6 GSC
Maximum daily mean = 581.8 GSC
Maximum instantaneous level = 582.3 GSC

Water Depth.

Ref: Dept. of the Environment,
Canadian Hydrograph Service
Nautical Chart #2252

Maximum water depth, 10± ft.
(referred to mean lake level)

crest
STILL WATER
- V
trough
10'±

Wave Height

A local resident suggested a maximum height of 10' ft.; this is a difficult value for an observer to describe, let alone measure. Strong winds create considerable splash or spray, especially along shore as the waves break and run up the beach. The elevation of maximum wave height is estimated to be of the order of 500± GSC Bed Material.

Per local information, the material is firm sand; bedrock or very large boulders may be visible, e.g. Susanne Island. Approximate average depth to bedrock is 20 ± ft.

Embankment Protection

The recent breakwater construction reportedly used "3 ton" stone as armour against wave and ice action. This would be required on the lake side only, from at least elevation 575. GSC (minimum daily mean) to elevation 500 GSC.

Freeboard
Max WAVE 5' 543
Max DAILY Mean 538
Mean 532
10' 578.6
568

Embankment Height

Disregarding the required grade to suit a high level bridge, the total height of embankment from lake bed to grade, including ~~54~~ ⁵⁴ ft. freeboard ~~and~~ above elevation 500. is 25[±] ft.

Lake Ice

A thickness of 2 ft. was reported; on shore, ice cakes probably pile up to a depth of 5 or 6 ft. Shallow water offshore generally breaks the ice fields into relatively small cakes.

Bridge

Tentative navigation clearances of 50 ft. vertical and 200 ft. horizontal have been mentioned; I understand that these clearances are negotiable.

The navigation clearances would likely be large enough that there would be ample hydraulic capacity to handle Blind River runoff.

Other Hazards

Obviously, during spring and fall, there will be periods when the lake is open, but air temperatures are below freezing. At such times, moisture and spray could freeze on the roadway.

Wind velocities over the causeway and bridge would tend to be higher and possibly gustier than on shore.

References

Mr. Ernest Carlson, Carlson Fisheries, Blind River.
Capt. Harvey Bell, Lakeside Ave., Blind River.

* Federal Department of Public Works
Mr. E. Ashton, 123 March St., Sault Ste. Marie.
Mr. R. Seawright, 25 St. Clair Ave. E., Toronto.
Mr. M. Kitchen " " " "

Federal Ministry of Transport,
Mr. J. Kennedy, Parry Sound.

I hope that this information is sufficient for your preliminary assessment of the proposed causeway alignment. If this scheme is chosen, detailed studies would obviously be required to provide a final hydraulic design.



J.W. Carter,
Project Hydrology Engineer.

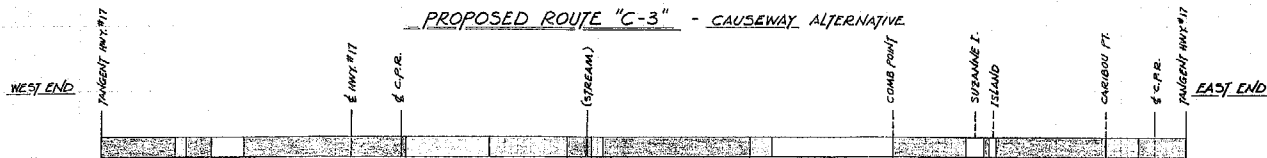
JWC/rh
cc: B. McKenna

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W.P. # 931-73-00

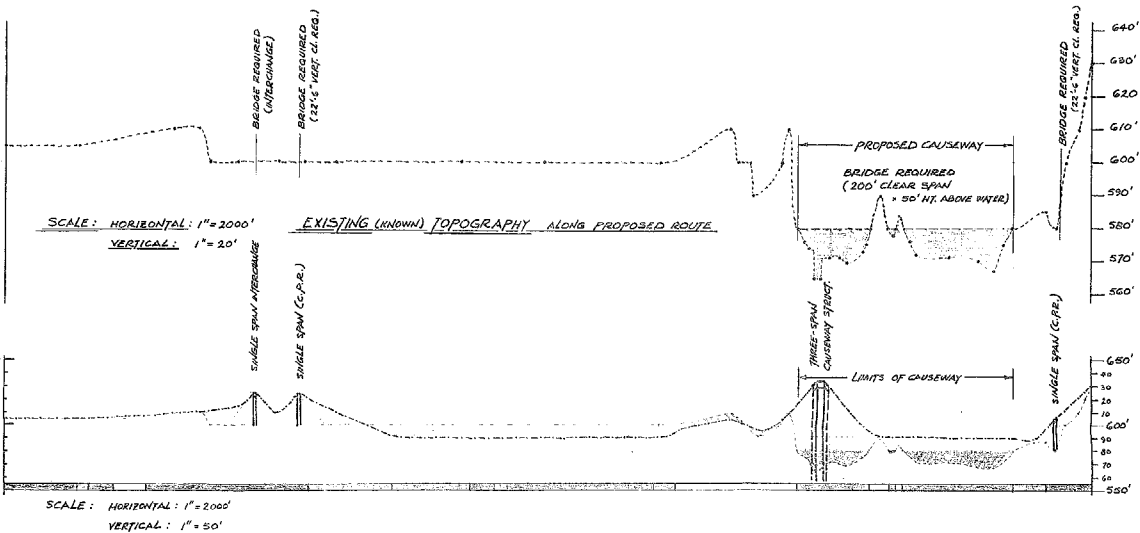
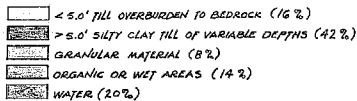
"MAP # A-7-c"

PROPOSED ROUTE "C-3" - CAUSEWAY ALTERNATIVE



SOIL CONDITIONS ALONG PROPOSED ROUTE "C-3"

HORIZONTAL SCALE : $1'' = 2000'$ ±



GEOMETRIC DESIGN CRITERIA: (FOR FREEWAY) - 80 M.P.H. DESIGN SPEED X-SECTION: 2-12' LANES & 10' SHOULDER EITHER WAY, SEPARATED BY 20' MEDIAN

- HORIZ. - MAX. 2" 30' CURVE
- VERT. - MIN. 1200' VERTICAL VISIBILITY CURVE; MAX. GRADE 3% (3' in 100' = 30' in 1000' = 60' in 2000') ie, @ THESE SCALES
- MIN. HEIGHT OF CAUSEWAY ABOVE WATER = 10'

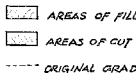
FOR CAUSEWAY STRUCTURE - MIN. CLEAR SPAN = 200', MIN. CLEARANCE FROM WATER = 50'

FOR RAILWAY STRUCTURE - MIN. VERT. CLEARANCE = 22'-6", MIN. HORIZ. CLEARANCE =

FOR INTERCHANGE STRUCTURE - MIN. VERT. CLEARANCE =

----- PLOT OF POSSIBLE PROFILE GRADE, ASSUMING MAX. DEPTH OF BEAM OR GIRDER = 5.0' FOR CAUSEWAY STRUCTURE
3.0' FOR OTHER STRUCTURES

ALSO, DEPTH FROM P.G. TO BASE OF SUBGRADE =



ORIGINAL GRADE

