

**FOUNDATION INVESTIGATION AND DESIGN REPORT**  
**NAISCOOT ACCESS ROAD BRIDGE**  
**HIGHWAY 69, TOWNSHIP OF HARRISON**  
**W.P. 5199-06-01, SITE: 44-043**  
**G.W.P. No. 5076-06-00**

**Geocres Number: 41H-113**

**Report to**

**MMM Group Limited**

Thurber Engineering Ltd.  
2010 Winston Park Drive, Suite 103  
Oakville, Ontario  
L6H 5R7  
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April 17, 2012  
File: 19-5161-21

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Road Bridge FIDR-FINAL.doc

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## **FOUNDATION INVESTIGATION AND DESIGN REPORT**

### **NAISCOOT ACCESS ROAD BRIDGE**

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## **PART 1: FACTUAL INFORMATION**

### **1 INTRODUCTION**

This report presents a review and summary of the factual information available at the site of the Naiscoot Access Road Bridge in the Township of Harrison, Ontario. The new bridge will be constructed on or close to the existing alignment of the former Highway 69 bridge over Naiscoot Lake.

The purpose of the review was to examine the reported subsurface conditions at the site and the foundation design and performance of the existing bridge to determine the parameters that would influence design and construction of the new bridge. A model of the subsurface conditions was interpreted from the data from the construction of the existing bridge. The information used in this interpretation was obtained from a 1960 report by GEOCON Ltd, Reference S7064 and titled "Report to Department of Highways Ontario on Construction Procedure – Approach Embankments, Naiscoot River Bridge, Township of Harrison, Ontario"

Thurber prepared this report as a sub-consultant to MMM Group, under the Ministry of Transportation Ontario (MTO) Agreement Number 5006-E-0030.

### **2 SITE DESCRIPTION**

The site of the proposed structure is located approximately 3.9 km north of the intersection of Highway 529 and Highway 69. The proposed structure lies on the alignment of the existing Highway 69 bridge over Naiscoot Lake. At this location Naiscoot Lake is approximately 115 m wide but the road crossing has been narrowed to approximately 37 m by placement of approach embankments extending into the lake. The water level in the lake was measured at Elevation 182.07 in August 2010.

The existing bridge has a span of approximately 37 m and is approached by embankments extending into the lake from the north and from the south.

The lake shore slopes are generally well treed, with grass and shrubs along the edges and in the open areas. In places, bedrock is exposed at surface at the lake shore. A photo of the existing Highway 69 bridge is included in Appendix C.

The lands immediately surrounding the site are generally undeveloped except for a marina to the west of the site and scattered cottages on the lake.

The site lies within the physiographic region known as the Georgian Bay Fringe, which covers Parry Sound and Muskoka. The region is characterized by very shallow overburden and bare rock knobs and ridges. Bedrock is exposed in many areas and intermittent swamps were filled in when glacial lake Algonquin inundated the area. The overburden materials consist of sand, silt and clay. Recent organic deposits of peat and muck occur in abundance in the bedrock hollows and valleys.

The area is underlain by strongly foliated and highly to moderately deformed rocks of Precambrian age of the following types:

- Gneisses of metasedimentary origin.
- Migmatitic rocks and gneisses.
- Felsic igneous rocks (tonalite, granodiorite, monzonite, granite, syenite, derived gneisses).
- Tectonite unit (tectonites, various gneisses).

### **3 SITE INVESTIGATION AND FIELD TESTING**

No site investigation and testing was carried out at the Naiscoot Access Road bridge site under the current assignment.

The stratigraphy at the site has been interpreted from information available in the MTO GEOCRES files relating to the construction of the existing bridge.

A copy of the GEOCRES information is included in Appendix A and the existing bridge foundation drawings in Appendix B.

### **4 LABORATORY TESTING**

No laboratory testing was carried out for this site under the current assignment.

## **5 DESCRIPTION OF SUBSURFACE CONDITIONS**

### **5.1 Site Stratigraphy**

Based on the information contained in the GEOCRE files, the original stratigraphy identified at the bridge abutment is summarized as follows:

#### **5.1.1 Lake Water**

The lake is shown as being approximately 5 m deep at the time of investigation in the year 1960.

#### **5.1.2 Very Soft Clay**

The soil encountered in the lake bed was described as “very soft very silty clay”. The clay stratum is shown as being approximately 5 m thick. The clay was overlain locally at the north abutment by “extremely loose org. silt”.

#### **5.1.3 Sand and Gravel**

The very soft silty clay was underlain by loose to compact, fine to coarse sand and fine gravel. This layer is shown as being approximately 4 m thick.

#### **5.1.4 Bedrock**

The soils described above were found to be underlain by “igneous sound hard bedrock”.

### **5.2 Groundwater Conditions**

For the purposes of the design and construction of the new bridge, the groundwater level can be assumed to be essentially the same as the level of the water in Naiscoot Lake.

## **6 ORIGINAL CONSTRUCTION**

The GEOCON report describes the original construction of the approach embankments and the subsequent modifications to the structure design. The salient facts are summarized below.

The initial design for the site consisted of a three-span structure supported on caissons bearing on the bedrock. Rock fill embankments were planned for the north and south approaches.

During construction of the south approach in May 1959, a series of subsidences occurred in the rock fill that were attributed to failure of the very soft silty clay on which the rock fill was being placed. It was also observed that the toe of the failed rock fill had come to rest to the north of the centreline of the south abutment. Following discussions between the DHO staff and GEOCON it was decided, in 1960, to use blasting to displace the very soft clay below the embankment fill in

order to achieve a stable embankment geometry. At this stage it was also decided to substitute granular fill for the rock fill in the initial design.

After the first three stages of blasting had been completed, it was observed that rock fill covered the area of the intended caissons at the south abutment. After further discussions, a decision was made to return to rock fill embankments and modify the bridge design to a single span.

As far as can be determined from the available report and drawings, the embankments were completed using rock fill, with blasting used to displace the clay. A single-span bridge supported on spread footings bearing on the rock fill was then constructed. There was a recommendation to using a temporary modular bridge at first until settlement of the rock fill would be complete. This was implemented in 1960 and the new bridge was subsequently constructed in 1963.

## 7 MISCELLANEOUS

Mr. Alastair E. Gorman, P.Eng prepared the Foundation Investigation Report.

Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects, reviewed the report.

Thurber Engineering Ltd.



Alastair E. Gorman, P.Eng.,  
Senior Foundations Engineer



Report Reviewed by:  
P.K. Chatterji, P.Eng.,  
Review Principal, Designated MTO Contact

## **FOUNDATION INVESTIGATION AND DESIGN REPORT**

### **NAISCOOT ACCESS ROAD BRIDGE**

### **HIGHWAY 69, TOWNSHIP OF HARRISON**

**W.P. 5199-06-01, SITE: 44-043**

**G.W.P. No. 5076-06-00**

**Geocres Number: 41H-113**

## **PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

### **8 GENERAL**

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations for the foundations and approach embankments for the new structure proposed for this site.

The GA Drawing provided by MMM Group Limited shows that Naiscoot Access Road crosses Naiscoot Lake on a single span structure that will lie essentially on the same alignment as the existing structure. The new structure will have a similar span to the existing and the top of pavement will range from 188.8 at the south abutment to 189.3 at the north abutment. Consideration is being given to the option of a single lane new structure which will impose reduced total loads on the abutments.

The discussion and recommendations presented in this report are based on the information supplied by MMM Group Limited and on the factual data derived from previous studies.

### **9 STRUCTURE FOUNDATIONS**

Drawings D-454-3-2 and D-454-3-3, the foundation drawings for the existing bridge are included in Appendix A.

These drawings show that the existing bridge abutments are supported on spread footings bearing on the embankment fill. It is assumed that these footings bear on the rock fill placed to construct the embankments in 1960. Based on the original drawings, the founding elevations of the existing abutments are:

- North – 185.88
- South – 185.46



The drawings show that the footings are 1.88 m wide.

The reported ground conditions at the abutments consist of rock fill.

A summary of bridge inspection reports provided by Mr. Mike McCormick, P. Eng. of MTO Structural Section, NE Region, indicates that the south abutment settled 2 inches during construction. The settlement/movement of the abutments started right away during construction and within about 5 years of construction, the original construction gap of 1.5 inches between the girder and the ballast wall started to close at the north abutment. Within another 8 years, the gap was completely closed at both abutments and the deck jammed tight against the abutments. The structure was jacked in 1964 and 1965 to relieve tilting of bearings.

The report also indicates that settlement of the approaches has occurred since the early 1980's and the girders butting against ballast walls, tilting of bearings and settlement of approaches have continued right up to the last inspection in 2010. It is not confirmed that the settlements at the abutments have stopped.

## **9.1 Suitable Foundation Types**

### **9.1.1 Deep Foundations**

The presence of rock fill underlying the site presents difficulties with respect to advancing deep foundations. Accordingly, deep foundations are not recommended for the new structure.

### **9.1.2 Shallow Foundations**

The new structure abutments may be founded on spread footings bearing on the existing rock fill provided the following measures are taken to mitigate settlement of the abutments of the existing bridge. These measures are:

- The new bridge loads on the abutments must not be more than the existing bridge loads. Consideration should be given to designing a single lane new structure which will significantly reduce the load per abutment compared to a two lane new structure.
- The new bridge footings must be positioned at the same locations as the existing footings. The width of the new footing should not be less than the width of the existing footing i.e. 1.88 m. The width of the new footing, therefore, must be greater of 1.88 m or the width obtained based on the following maximum geotechnical resistances:
  - Factored ULS = 600 KPa
  - SLS = 400 KPa.

These resistances are for vertical loads and must be modified in accordance with CHBDC Clause 6.7.4. where inclined loads are present.

- It is recommended that the new footings be founded at or above the elevation of the existing footings i.e. 185.88 at the north and 185.46 at the south. If the new founding elevation is higher than the existing, the difference must be made up using mass concrete or engineered fill consisting of Granular B Type II.

Preparation of the founding surface must be as described in this report.

## **9.2 Frost Depth**

The design depth of frost penetration at this site is 1.8 m.

However, frost penetration is not an issue for footings founded on rock fill above the water level.

## **9.3 Lateral Resistance**

For computing lateral sliding resistance, it is recommended that an unfactored coefficient of sliding friction of 0.6 be used. This recognizes the fact that the footing may bear on a leveling bed of granular material.

## **9.4 Abutment Design Considerations**

From a geotechnical perspective, the conditions at this site are considered to be suitable for the design of conventional or semi-integral abutments.

The site is not considered to be suitable for the design of integral abutments unless special measures are adopted to install piles through the rock fill and to provide pile flexibility.

Design of the abutment must take account of the CHBDC requirements for scour and erosion protection.

## **9.5 Recommended Foundation**

From a geotechnical and foundation cost perspective, the recommended foundation consists of spread footings bearing on the existing rock fill.

# **10 EXCAVATION AND BACKFILL**

## **10.1 General**

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA) and in accordance with OPSS 902. For the purposes of the OHSA, the fill in the existing approach embankments at this site is classified as Type 3 soil.

### **10.2 Excavation at the Abutments**

Excavation at the abutments will encounter the granular pavement structure and underlying rock fill. Temporary excavation slopes in these materials must not be steeper than 1H:1V.

Two concerns that can be identified with respect to excavating into the rock fill at the abutments are:

- Disturbance of the rock fill if excavation is carried out below the existing founding elevation
- Loss of fresh concrete into voids in the rock fill

Accordingly, it is recommended that the new foundations not be founded at a lower elevation than the existing foundations.

### **10.3 Abutments**

Backfill to the abutment must be granular material placed to the extents shown in OPSD 3101.150.

Where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular B Type II.

The backfill to the abutment walls must be in accordance with OPSS 902, November 2010. All granular material should meet the requirements of SP 110S13 Amendment to OPSS 1010, April 2004.

Compaction equipment to be used adjacent to the abutment walls must be restricted in accordance with OPSS 501.

## **11 GROUNDWATER AND SURFACE WATER CONTROL**

Based on the recommended founding elevations, the composition of the fill and the elevation of Naiscoot Lake, no dewatering requirements are anticipated for this site.

However, all the requirements of OPSS 902, with respect to dewatering, shall continue to apply.

## **12 BRIDGE APPROACHES AND EMBANKMENTS**

The existing approach fills are estimated to be approximately 13 m high close to the abutments and are reported to have been constructed of rock fill. Visual inspection indicates that these embankments are stable although they have undergone settlement.

No grade raise is required in association with the construction of the new bridge. Accordingly, the embankments are expected to continue to be stable and no work is anticipated beyond reinstatement of areas disturbed by the bridge construction.

### 13 ROADWAY PROTECTION

Naiscoot Access Road will be closed for construction of the new bridge.

Accordingly, there will not be a need for roadway protection.

### 14 EARTH PRESSURE

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K(\gamma h + q)$$

where:  $p_h$  = horizontal pressure on the wall at depth  $h$  (kPa)

$K$  = earth pressure coefficient (see Table 13.1)

$\gamma$  = unit weight of retained soil (see Table 13.1)

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 13.1.

**Table 13.1 – Earth Pressure Coefficients (K)**

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ, \gamma = 19.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*	0.20	0.26*
At Rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive	3.7	-	3.3	-	5.0	-

\* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall.

The factors in Table 13.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 1.7 m for Granular B Type I or at a depth of 2.0 m for Granular A or Granular B Type II.

## **15 SEISMIC CONSIDERATIONS**

### **15.1 Seismic Design Parameters**

The bridge is located in an area where the overburden is underlain by Pre-Cambrian rocks of very low activity.

The site is treated as lying in Seismic Zone 1. The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 1
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.08

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

### **15.2 Liquefaction Potential**

The embankment material and the underlying soil are not considered to be at risk of liquefaction.

### **15.3 Retaining Wall Dynamic Earth Pressures**

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that incorporate the effects of earthquake loading.

For the design of retaining walls, the coefficients of horizontal earth pressure in Table 14.1 may be used:

**Table 14.1 – Earth Pressure Coefficient for Earthquake Loading**

Earth Pressure Coefficient (K) for Earthquake Loading						
Wall Condition	Granular A or Granular B Type II $\phi = 35^\circ; \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ; \gamma = 19.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active ( $K_{AE}$ )*	0.28	0.42	0.32	0.51	0.21	0.28
Passive ( $K_{PE}$ )	3.6	-	3.2	-	5.0	-
At Rest ( $K_{OE}$ )**	0.47	-	0.52	-	0.38	-

\* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

\*\* After Woods

## 16 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

### 1. Variation in Site Conditions

The ground conditions and foundation design recommendations contained in this report have been based on information from the construction of the existing bridge. It is possible that variations from the assumed conditions will be encountered. The Contract Documents must instruct the Contractor to advise the Contract Administrator if he is of the opinion that there is a variation in conditions. If the Contract Administrator cannot resolve the issue on site, it should be referred to the design team.

## 17 CLOSURE

Engineering analysis and preparation of the Foundation Design Report were carried out by Mr. Alastair E. Gorman, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.



Alastair E. Gorman, P.Eng.,  
Senior Foundations Engineer



P. K. Chatterji, P.Eng.,  
Review Principal

**Appendix A**  
**GEOCON Report**







BH 2A DOES NOT COINCIDE WITH THE SOUNDING

BLASTING DATES

<u>I</u>	MARCH 28
<u>II</u>	APRIL 12
<u>III</u>	APRIL 22
<u>IV</u>	APRIL 26
<u>V</u>	MAY 31
<u>VI</u>	MAY 31
<u>VII</u>	APRIL 20

BORE HOLE DATES

1	APRIL	2	} AFTER BLAST I
2	"		
3A	"		
3	APRIL	5	
4	"		
5	APRIL	8	
6	"		
7	APRIL	9	
8	"		
9			
10	APRIL	13	} AFTER BLAST I, II & III.
11	"		
12	MAY	4	
13	"		
14	"		
15	MAY	4	
16	"		
17	"		

SOUNDING DATE

MARCH 29 - AFTER BLAST I

May 4. 170+70 9' west

B.H. 16

0-15 water

15-20 clay

20-23 rock

44 soil

B.H. 17

0-70 water

170+102

10' west

71-74 clay

75-76 rock

B.H. 18

No change above bore

Summary

168+80 to 169+90

started May 28 completed June 29

DEFECTS IN NEGATIVE DUE TO  
CONDITION OF ORIGINAL DOCUMENT

Mr. A. M. Toye,  
Bridge Engineer.  
Materials & Research Section.

March 7, 1960.

FOUNDATION REPORT -- by  
Geocor, Limited.

Attention: Mr. S. McCombie.

Re: -- MAISCOOT RIVER BRIDGE --  
Hwy. No. 69 - Contract No. 59-05  
District No. 17

Enclosed herewith, is the report on the  
Maiscoot River Subsoil Investigation, submitted by Geocor,  
Limited.

The remedial construction programme necessitated  
by the rock slide which occurred, has been discussed in detail,  
with Mr. E. A. Tregaskes and the Bridge Design Engineer,  
Mr. B. Davis.

This report is forwarded for your records.

LGS/M&EF  
Attach.

*for Mr. McCombie*  
L. G. Soderman,  
PRINCIPAL SOILS & FOUNDATIONS ENGINEER

cc: Messrs. A. M. Toye (2)  
H. A. Tregaskes  
H. D. McMillan  
C. K. Hunter  
E. A. Cash  
L. Eadie  
E. B. Saint  
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Dut 28-12

# GEOCON LTD

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W.P. 904-57

Rexdale, Ontario,  
October 5th, 1960.

Department of Highways, Ontario,  
Materials and Research Section,  
Downsview, Ontario.

Attention: Mr. A. Rutka, P. Eng.,  
Acting Materials and Research Engineer.

Re: Construction Procedure - Approach Embankments,  
Naiscoot River Bridge,  
Township of Harrison, Ontario.

Dear Sirs:

This letter accompanies our detailed report on the construction procedures adopted for the approach embankments at the Naiscoot River Bridge, Township of Harrison, Ontario.

It is felt that the control blasting procedure, which was evolved in cooperation with your staff, is unique. Consequently detailed records of the efficiency of the blasting and filling operation were kept and are presented in this report together with other pertinent data.

Yours very truly,

GEOCON LTD

*V. Milligan*

V. Milligan, P. Eng.,  
Assistant Chief Engineer.

VM/dw  
S7064

S7064  
REPORT  
TO  
DEPARTMENT OF HIGHWAYS, ONTARIO  
ON  
CONSTRUCTION PROCEDURE - APPROACH EMBANKMENTS  
NAISCOOT RIVER BRIDGE  
TOWNSHIP OF HARRISON                      ONTARIO

Distribution:

- 10 copies - Department of Highways, Ontario,  
Downsview, Ontario.
- 2 copies - Geocon Ltd,  
Rexdale, Ontario.

**GEOCON**

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## INTRODUCTION

Geocon Ltd has been retained by the Department of Highways, Ontario, by verbal agreement, to provide supervisory service during the construction of the approach embankments for the Naiscoot River bridge in the Township of Harrison, District of Parry Sound, Ontario.

The purpose of the supervision was to control the remoulding and displacement of soft salty clay beneath the existing portion of the south approach embankment and to control the removal and/or remoulding of clay in front of successive advances of the approach embankments.

## SITE

The site is located in the Township of Harrison, District of Parry Sound and about 2 miles north of Pointe au Baril where the re-alignment of Highway 69 crosses the Naiscoot River.

## HISTORY

The original bridge design, as shown on D.H.O. drawing D4011-1, consisted of a simply supported 3-span steel beam superstructure, founded on caissons driven to bedrock. The total length of the structure was to be 140 feet. The south and north approach embankments were to be constructed of rock fill about 40 feet wide at grade level and about 135 and 85 feet long respectively. It is understood that the dimensions of the river channel were based on hydrologic considerations and on requirements set forth by the Navigable Waters Act.

During the construction of the south approach embankment in May, 1959, several subsidences occurred, and it was noticed that the toe

## HISTORY (continued)

2.

of the embankment had slid into the river for a distance of about 70 feet. The operations were subsequently stopped. Geocon Ltd carried out a site investigation in February, 1960 and the results are contained in our report S 7045, dated February 22nd, 1960. It was found that the toe of the south approach embankment had slid into a layer of very soft silty clay, about 20 feet in thickness. It was further found that about 5 to 10 feet of this material were entrapped between the subsided portion of the embankment and the underlying sand and gravel stratum.

When it was known that the rock fill slide had covered the area of the proposed south abutment, it was decided to move the proposed bridge towards the north for a distance of about 25 feet. It was further decided to displace the clay entrapped beneath the embankment by blasting and to construct the remainder of the embankments of granular fill, using stage construction and pre-removal or remoulding of the clay.

A drawing showing the revised positions of the piers and abutments and a schematic outline of suggested stage construction, together with the recommended blasting patterns were submitted in our report S 7045, dated February 22nd, 1960 and are reproduced in this report on Drawing S 7064-1 and in Appendix I, respectively. The blasting procedures adopted are given in detail in Appendix II.

## PROCEDURE

The work was commenced on March 15th, 1960 and completed, except for final grading and widening of the embankment, on May 5th, 1960.

PROCEDURE (continued)

3.

The charges in Stage II were placed on April 11th and 12th and blasted on April 12th. Cross-sections after blasting are shown on Drawing S 7064-2.

The charges in Stage VII were placed on April 19th and April 20th and blasted on April 20th. The charges in Stage III were placed on April 21st and blasted on April 22nd.

The charges in Stage IV were placed on April 25th and blasted on April 26th. Sections over the check borings put down after blasting are shown on Drawing S 7064-2.

One additional line of charges at chainage 170+05 was fired on April 29th, and at chainage 170+02 on May 4th to make the toes of the north and south approach embankments meet.

Geocon Ltd left the site on May 5th. At that time final grading and widening of the embankment was still in progress.

A detailed procedure of the blasting operations is given in Appendix II to this report.

RESULTS

The various construction Stages will be discussed separately in order of construction progress.

Stage I

The results of soundings taken after blasting in this Stage are shown on Drawing S 7064-2 and indicated the presence of rock fill and scattered boulders north of the row of caissons for the south abutment planned at chainage 169+62. They further indicated the presence of clay

RESULTS (continued)

4.

Stage I (continued)

overlying rock fill north of chainage 169+20. This chainage corresponds to that where the clay stratum underlying rock fill before blasting increased in thickness.

At a subsequent meeting with the Department of Highways, it was decided to check, by putting down test borings, the extent of rock fill north of chainage 169+50 and to carefully check the progress of the advancing south approach embankment by soundings, ensuring that no rock fill would advance north of chainage 169+50.

Stages V and VI

Based on the experience obtained in Stage I, it was decided to place a granular berm at the toe of the north approach embankment, prior to blasting to prevent the embankment from sliding over the location of the proposed north abutment at chainage 171+02.

About 700 cubic yards of granular fill was used for the construction of the toe-berm extending to about chainage 170+90.

Stages V and VI were fired together. As a result of the blast the granular berm together with part of the rock fill embankment subsided. The results of soundings taken subsequently are shown on Drawing S 7064-2 and indicated the presence of rock fill at the location of the proposed north abutment and extending south from it to about chainage 170+80.

A second meeting with the Department of Highways was then held and it was decided to depart from the initial bridge design. It was proposed to maintain the planned blasting program and filling operations

## RESULTS (continued)

5.

### Stages V and VI (continued)

using rock fill instead of granular fill. The embankment was to be constructed at temporary grade elevation 600 to chainages 169+68 and 170+42 at the south and north sides respectively, thereby maintaining a navigable channel about 70 feet wide. The above chainages were based on the approach embankments meeting each other at the toes at chainage 170+05 under a natural slope of 1 vertical to 1-1/4 horizontal. It was further decided to blast and fill symmetrically in relation to chainage 170+05. The embankment was to be advanced under a wedge-shaped front in order to ensure the maximum displacement of clay. With the embankments completed, a simple span would be put across the opening, supported on spread footings within the rock fill. To allow for probable settlement within the rock fill, a temporary crossing of the Bailey bridge type would be used before placing the final structure.

### Stages II, VII, III and IV

As construction proceeded, the blasts in Stages II, VII, III and IV were set off individually and in that order. Soundings were taken regularly at the toes of the advancing embankments and the displacement of clay was found generally to be satisfactory. Due partly to the frozen condition of the rock fill stock piles which made blasting of the stock piles necessary, and partly to lack of sufficient equipment to handle the rock fill, the embankments could not be advanced at full width at elevation 600. During construction the width of the approach embankment was generally 40 to 60 feet. It was found that a mud wave was formed at the

## RESULTS (continued)

6.

### Stages II, VII, III and IV (continued)

toe of the embankment, about 10 feet long and 4 to 5 feet high over the full width of the embankment. It was further found that the natural slope of the rock fill by end dumping was about 1 to 1. Several borings were put down through the rock fill to check the degree of displacement of the clay. The results were generally satisfactory and wherever clay was found entrapped between the rock fill and the underlying sand and gravel stratum, dynamite charges were put down the hole and fired. On re-drilling such holes, the clay invariably had been displaced and replaced by rock.

### Additional Blasting

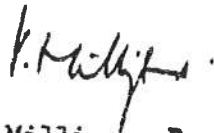
After each blast the rock fill embankment end slopes generally subsided as shown on Drawing S 7064-3. The positions of the embankment before and after the blast in Stage IV are shown. The toes were found to be still 15 to 20 feet apart after blasting. It was then decided to blast another row of charges at chainage 170+05. Previous to this blast the embankments were advanced 2 feet beyond the desired positions and overloaded at the edge of the end slope. After the blast the toes had met in several places. One final blast together with overloading of the end slopes was then set off at chainage 170+02 with the result that the toes met completely over the width of the embankment. About 4 to 5 feet of remoulded clay covered the rock fill where the toes met.

PERSONNEL

7.

The field work was carried out by Mr. A. Tilk and Mr. A. Prior.  
The report was written by Messrs. Tilk and Prior and reviewed by Mr.  
V. Milligan.

VM/dw  
S7064

  
V. Milligan, P. Eng.

GEOCON

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**APPENDIX I**

**RECOMMENDED BLASTING PATTERNS**



Recommended Procedures  
for  
Remolding River Bed Clay  
and Granular Approach Fill

A. South Approach:

- (1) (a) Between chainages 168+79 and 169+50 (i.e., area where rock fill overlies clay) boreholes shall be drilled at 10' centres to a depth sufficient to contact the sand stratum underlying the clay. The limits of the grid of borings to the left and right of centreline between the above chainages shall be determined by the intersection of a 1:1 side slope from profile grade with the upper horizon of the clay layer.
- (b) In each of the holes drilled, a 40 lb. charge of C.I.L. ditching dynamite (i.e., 50% Forcite - 1-1/4" x 8" lg. sticks) shall be placed at the base of the clay layer where the thickness of the stratum is less than 8 feet or above the base of clay layer equal to 1/3 the clay layer thickness where it is in excess of 8 feet.
- (c) Every fourth charge in one row of loaded holes shall be positively fired; propagation firing may be used for intermediate charges. The entire pattern may be fired as one shot.

A. South Approach: (continued)

- (2) Immediately after the above blasting has been carried out, probings should be made to determine whether or not rock fill has moved in a longitudinal direction beyond Station 169+50. If rock is encountered beyond this chainage, this rock will have to be excavated for the entire width of area blasted before proceeding with step (3).
- (3) Rock fill can be advanced to chainage 169+12 at elevation 600.0'. This chainage point corresponds to centreline chainage at the intersection point of the "as dumped" slope of rock with horizontal line at elevation 600.0'.
- (4) Wash and load holes at 10' centres on two lines at chainages 169+50 and 169+60. Blast these two rows as one shot. Limits of grid of holes to the right and left of centreline should be as defined in 1(a); charge per hole to be as defined in 1(b).
- (5) Advance approved granular fill (not rock fill) to chainage 169+25 at Elev. 600.0'. Definition of chainage as given in (3).
- (6) Wash and load holes at 10' spacing in two rows 10 feet apart. Rows located at chainages 169+75, and 169+85. Fire both rows as one blast.
- (7) Advance approved granular fill to chainage 169+50 at Elev. 600.0'.

A. South Approach: (continued)

- (8) Wash and load holes at 10' spacing in two rows. Rows located at chainages 170+00, and 170+10. Fire both rows as one blast.
- (9) Advance approved granular fill to Station 169+75, at Elev. 600.0'.

B. North Approach:

- (1) (a) Underlying the North approach fill, the thickness of the clay is less than at the South approach location. The number of pounds of powder per hole can be varied where the clay is less than 8 feet thick. A powder factor of one should be maintained - (i.e., 1 lb. of dynamite per cu.yd. of clay to be remolded). The depth at which the charge should be placed is defined in 1(b).
- (b) Wash and load holes spaced at 10' centres in two rows 10 feet apart. Row chainages at 170+85, and 170+95. Limits of rows right and left of road centreline to be determined using intersection of 1:1 side slopes, with upper horizon of clay stratum as for South approach fill. Fire both rows as one blast.
- (2) Advance approved granular fill to chainage 171+11 at elevation 600.0'.

B. North Approach: (continued)

- (3) Wash and load holes at 10' centres in two rows 10 feet apart. Row chainages at 170+70, and 170+60. Fire both rows as one blast.
- (4) Advance approved granular fill to chainage 170+95, with top of fill at elevation 600.0'.
- (5) Wash and load holes at 10' centres in two rows 10 feet apart. Row chainages at 170+45, and 170+35. Fire both rows as one blast.
- (6) Advance approved granular fill to chainage 170+70, with top of fill at elevation 600.0'.
- (7) Piling for caissons can now be driven. A protective layer of at least four feet of rock fill should be placed on the side and end slopes of the approach fill granular core. The maximum dimension of individual rock sizes should be 2'.

C. Pump Specifications:

The washing of holes in which the charges of dynamite are to be placed, requires the provision of a pump with the following specifications:-

Capacity .....	1,000 gallons/min.
Discharge Pressure .....	150 - 200 p.s.i.
Discharge Orifice .....	2" Diameter.

C. Pump Specifications: (continued)

Also required is the provision of a total footage of 50' of 2" O.D. standard pipe supplied in 5' lengths, each length threaded at both ends; couplings, 1 tee pipe section, 1 valve and 1 end cap to be provided.

D. The above blasting and filling procedures are illustrated on Drawing S7064-1.

Prepared by: L. G. Soderman,  
Department of Highways,

V. Milligan,  
Geocon Ltd.

S7064

**GEOCON**

## **APPENDIX II**

### **BLASTING PROCEDURES USED**

## APPENDIX II

### BLASTING PROCEDURES USED

The blast holes in Stage I were put down through the rock fill using standard machine drillrigs. In view of the number of holes to be drilled in Stage I and the time limit on explosives under water, three drillrigs were employed in this stage each working 24 hours per day in two 12-hour shifts. The holes in the remaining stages were jetted, using one drillrig and one day shift only.

The explosive used was C.I.L. 50 per cent Ditching Dynamite with a velocity of 17,500 feet per second. The maximum allowable lag between placing and detonating was 12 days and preferably not in excess of 10 days. The blast holes were loaded either through AX or BX casing. To jet the charges in place, a Jaeger pump was employed with a rated capacity of 500 gallons per minute at a discharge pressure of 150 pounds per square inch. Each blast hole was loaded with about 40 to 50 pounds of dynamite or about ninety 1-1/4 by 8 inch sticks.

Initially, the holes were loaded 5 sticks at a time, then gradually increased to 10 to 15 sticks at a time. It was found by experiment that a jetting pressure of 130 pounds per square inch was required for BX casing and 135 pounds per square inch for AX casing to properly deposit the charge in the clay stratum. In this way an average of 12 to 15 holes were loaded in a 12-hour shift.

Detailed layouts of the charges are given on Drawing S 7064-4. The holes to be fired positively were primed with 4 sticks attached to

plastic wrapped Primacord. The Primacord was then attached to stakes driven into the ice or poles driven into the clay. When all holes were loaded 1 or 2 electric detonator caps together with 1 extra primer stick were connected to the Primacord and all caps were hooked up into a circuit of connecting wire. A sketch showing this arrangement is given on Drawing S 7064-4. The circuit was tested by passing a small electrical current through it. The primed charges were then detonated by closing the circuit through a 1200 Series Blasting Machine with a rated voltage of 480. Non-primed charges were detonated by propagation.

Part of Stage II was fired simultaneously with Stage I as shown on Drawing S 7064-4. The purpose of firing the extra line was to effect a counterforce limiting the trend of the rock fill embankment to slide. A similar consideration was incorporated in the lay-out of primed charges in Stages V and VI.

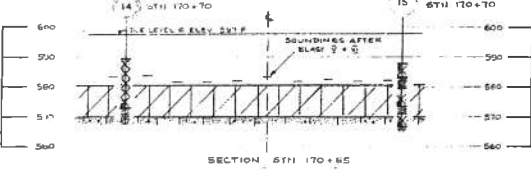
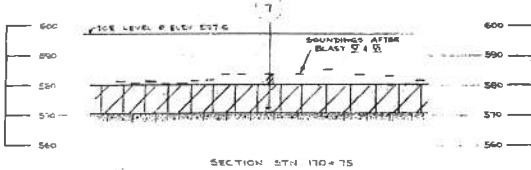
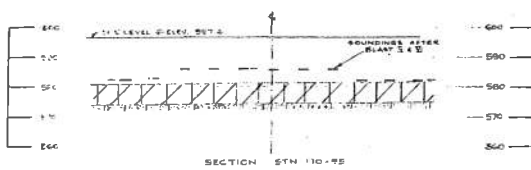
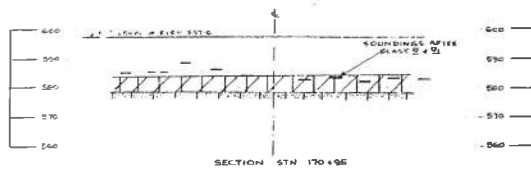
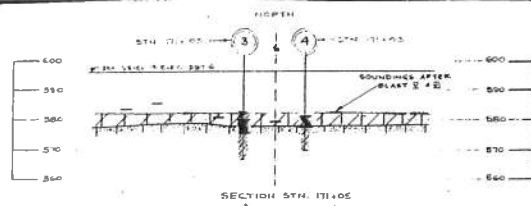
Two additional blasts were set off along chainage 170+05 and 170+02 to let the toes of the approach embankments meet. The second of these blasts was effected by using "bombs" consisting of whole cases of ditching powder put down by their own weight in the already remoulded clay and set off by using electric caps and connecting wire only.

The materials used for blasting are summarized in the following table.

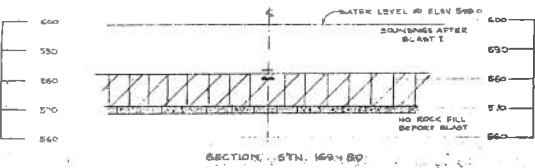
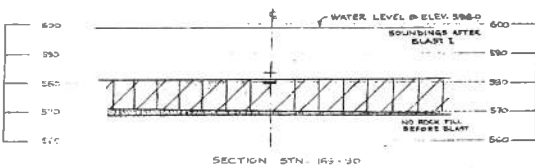
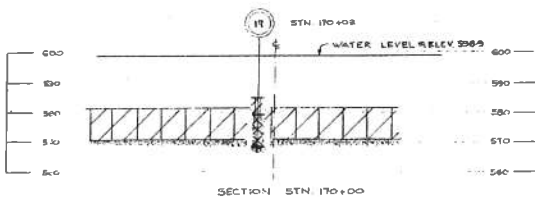
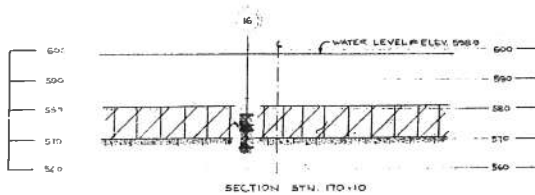


<u>Stage</u>	<u>50% Ditching Dynamite Pounds</u>	<u>Plastic Wrapped Primacord feet</u>	<u>Detonator Caps No. 8</u>	<u>Connecting Wire feet</u>	<u>75% Forcite pounds</u>
I	2900	1200	40	850	--
II	800	140	4	900	--
III	1125	380	10	850	--
IV	1300	392	11	400	--
V, VI	1765	490	14	800	--
VII	1120	275	8	850	50
170+05	550	70	6	900	200
170+02	700	--	7	650	--
	<hr/>	<hr/>	<hr/>	<hr/>	<hr/>
Total	10260	2947	100	6200	250

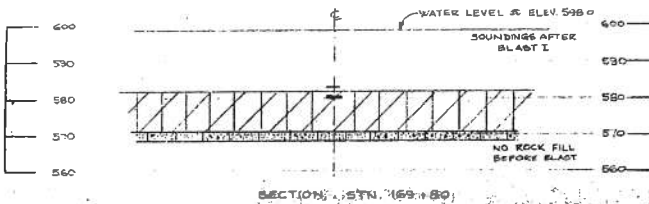
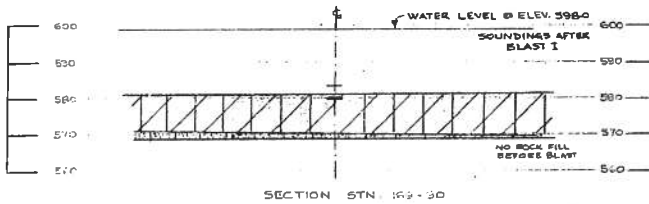
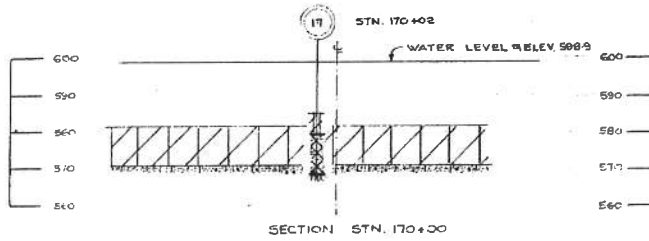
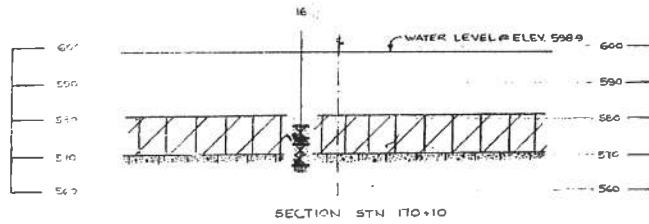
NOTE: Part of connecting wire salvaged after each blast and re-used.



SOUNDINGS & BORINGS INCLUSIVE  
FOR AREA STN. 170+10 - 170+65



SOUNDINGS & BORINGS INFORMATION  
FOR AREA STN 170+10 - 170+65



LEGEND

- BOREHOLE IN ELEVATION S706.4
- BOREHOLE IN ELEVATION S704.5
- ROCK SURFACE, SOUNDINGS AFTER BLASTING
- CLAY SURFACE, SOUNDINGS AFTER BLASTING

STRATIGRAPHY

- ROCK FILL
- VERY SOFT GREY - BROWN SILTY CLAY
- LOOSE TO COMPACT GREY SILTY SAND TO SANDY SILT
- COARSE TO FINE GRAINED SAND AND GRAVEL

DRAWING FROM  
STN. 169+80 TO 171+05

DEPARTMENT OF HIGHWAYS, ONTARIO

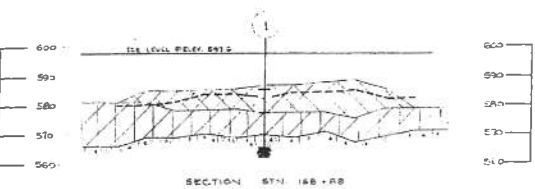
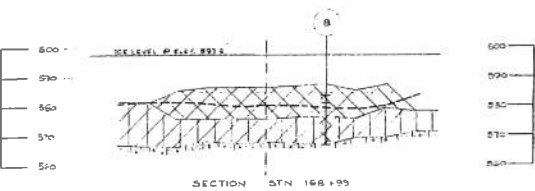
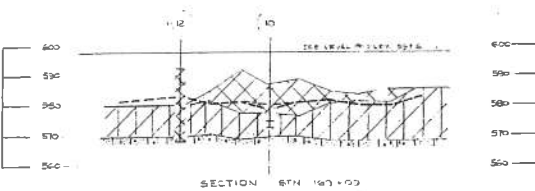
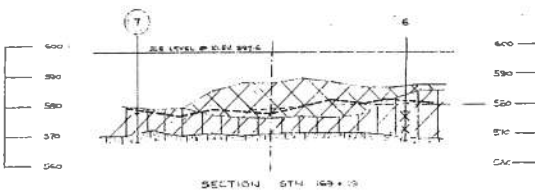
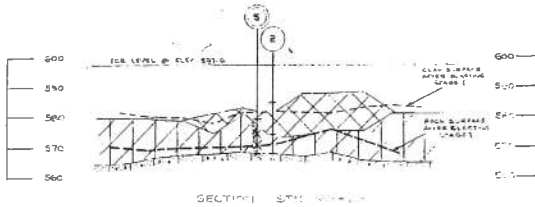
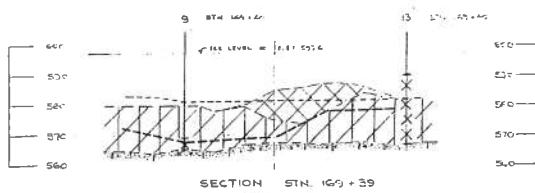
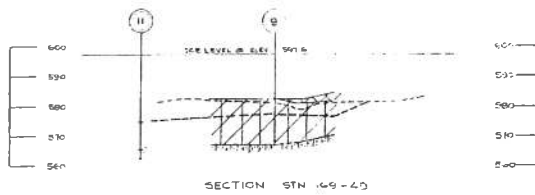
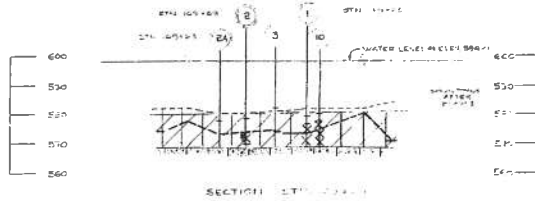
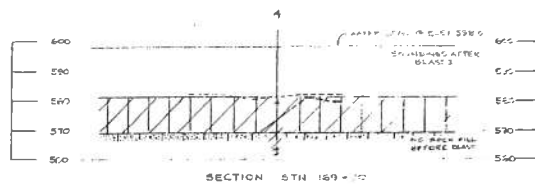
NAISCOOT RIVER BRIDGE

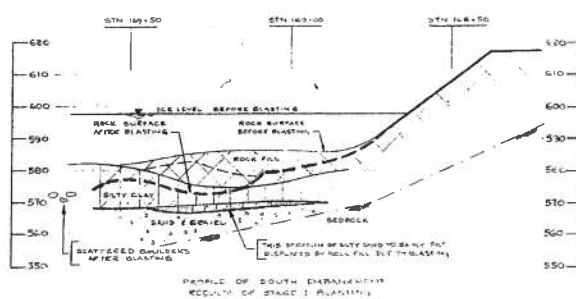
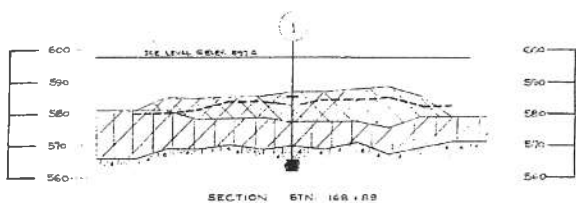
SOIL STRATIGRAPHY BEFORE BLASTING  
SOUNDINGS AND CHECK BORINGS AFTER BLASTING

GEOCON LTD

DATE: SEP 15, 1960 SCALE: 1" = 10'

ART. 171 No. S706-2A





PROFILE OF SOUTH DAKOTA  
RESULTS OF STAGE 1 PLAYING

### LEGEND

ROCK COIL IN ELEVATION 5706.1

BORE HOLE IN ELEVATION 57045

RECEIVED: 1967 OCT 16

CLAY SURFACE, SUNDRIES 31.1.1955

## STRATIGRAPHY

T Gray - Brown Silty Clay

COMPACT GREEN SILTY SAND TO  
SANDY SILT

## REFERENCES



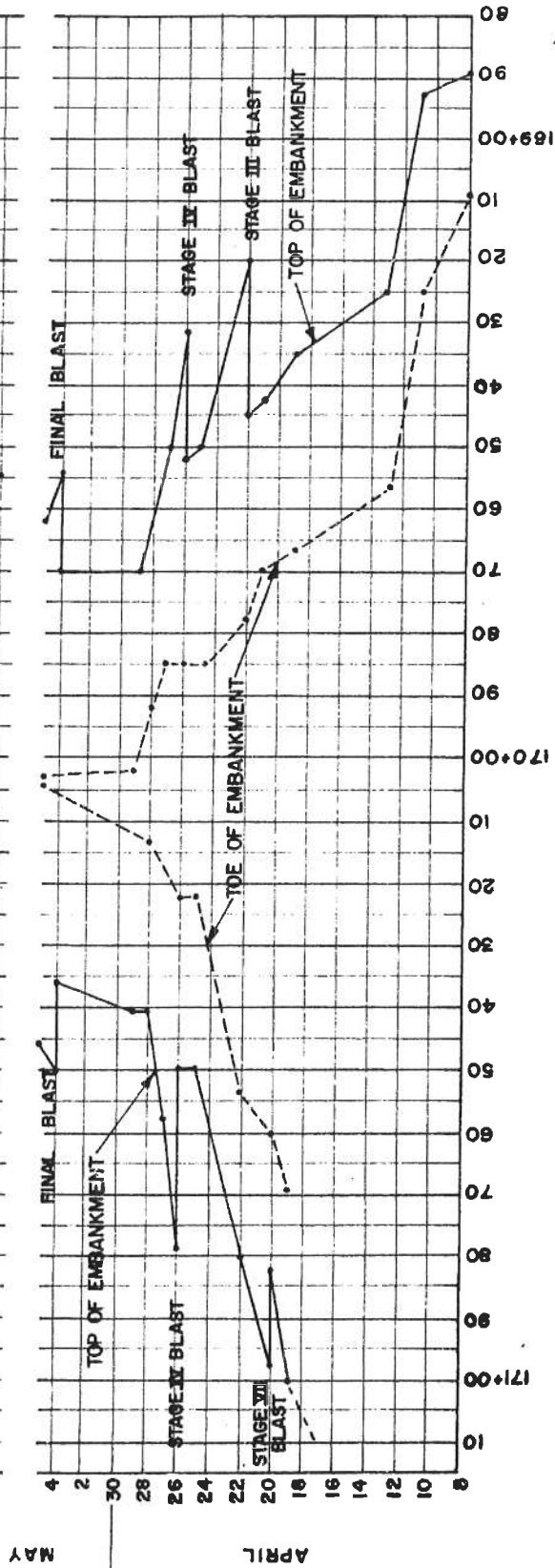
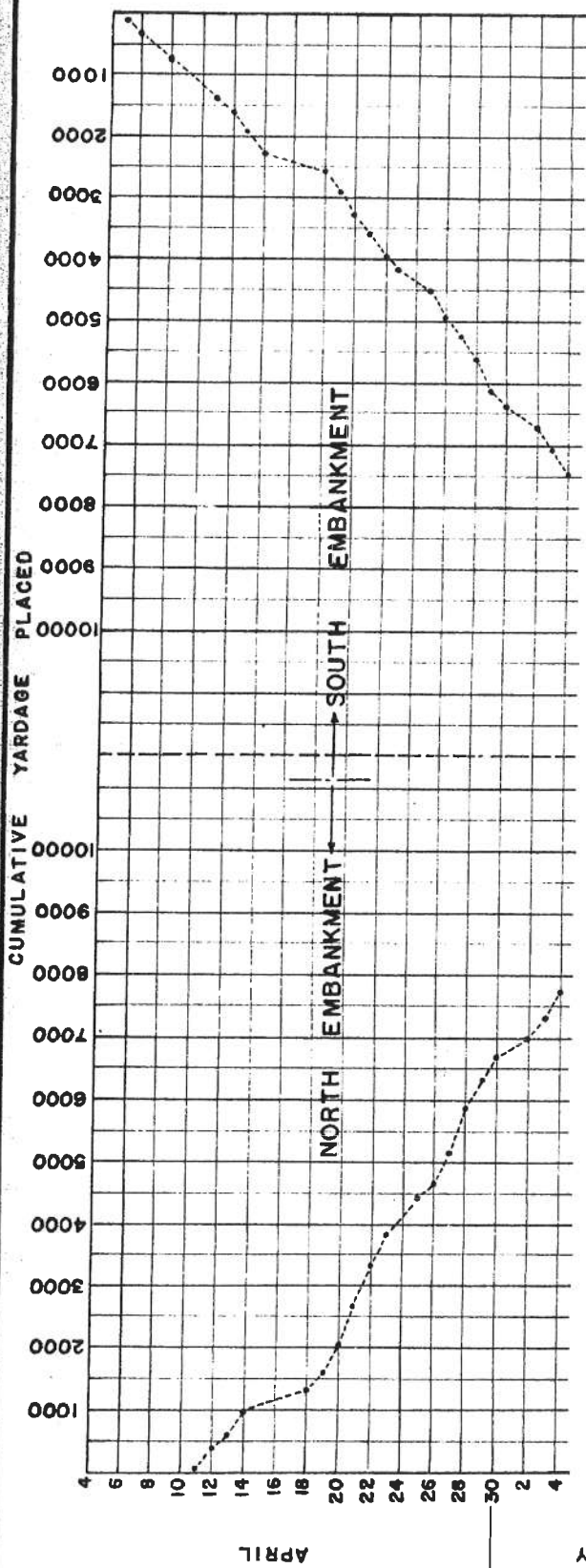
169-72

2000

OUT

BEFORE BLA, 51  
SOEING: 4-772 214

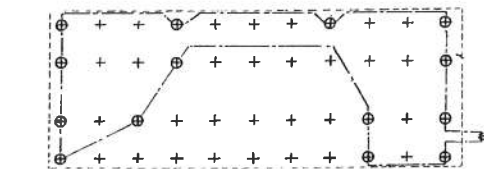
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DEPARTMENT OF HIGHWAYS, ONTARIO		GEOCON LTD	
TORONTO		ONTARIO	
NAISCOOT RIVER BRIDGE		DATE SEPT. 16/60 SCALE -	
TOWNSHIP OF HARRISON		MADE BY	
PROGRESS OF EMBANKMENT VS TIME		CHKD. BY	
		APPD. BY	
		No. S 7064 - 3	



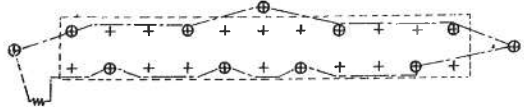
BLASTING AREA  
STAGE I & II



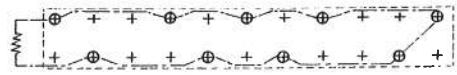
BLASTING AREA  
STAGE III



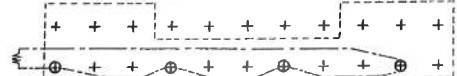
BLASTING AREA  
STAGE IV



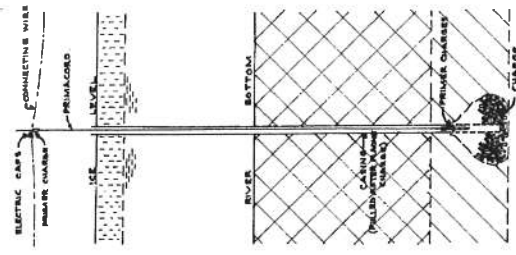
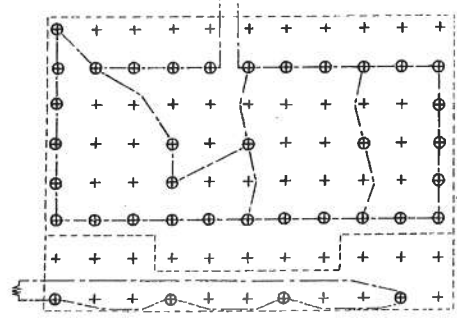
BLASTING AREA  
STAGE V



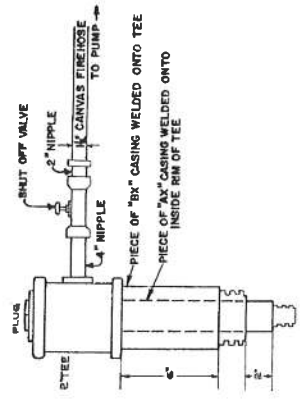
BLASTING AREA  
STAGE VI



BLASTING AREA  
STAGE VII



TYPICAL DIAGRAM  
OF POSITIVELY FIRED CHARGE



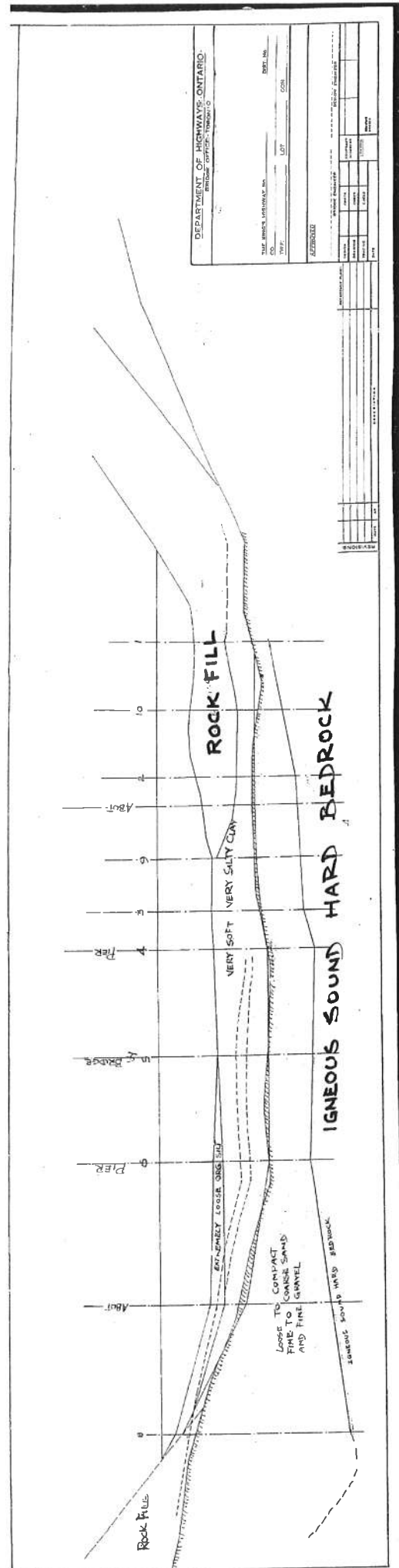
JETTING TEE FOR 8" X 8" CASING

LEGEND

- ⊕ CHARGE PRIMED AND POSITIVELY FIRED
- + CHARGE, FIRED BY PROPAGATION
- TO BLASTING MACHINE

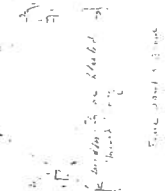
PLAN SHOWING LAYOUT OF CHARGES  
SCALE 1" = 20'-0"

DEPARTMENT OF HIGHWAYS, ONTARIO		<b>GEOCON LTD</b>	
TORONTO		DATE SEPT. 19/60 SCALE AS SHOWN	
NAISCOOT RIVER BRIDGE		DRAWN BY: H. J. No. S7064-4	
TOWNSHIP OF HARBURG		CHECKED BY: H. J.	
DETAILS OF BLASTING PROGRAMME			



DEPARTMENT OF HIGHWAYS-ONTARIO  
 DIVISION OF HIGHWAYS  
 PROJECT NO. \_\_\_\_\_  
 SHEET NO. \_\_\_\_\_  
 DATE \_\_\_\_\_  
 DRAWN BY \_\_\_\_\_  
 CHECKED BY \_\_\_\_\_  
 APPROVED BY \_\_\_\_\_  
 TITLE \_\_\_\_\_  
 LOCATION \_\_\_\_\_  
 SCALE \_\_\_\_\_  
 SHEET NO. \_\_\_\_\_  
 PROJECT NO. \_\_\_\_\_  
 DATE \_\_\_\_\_  
 DRAWN BY \_\_\_\_\_  
 CHECKED BY \_\_\_\_\_  
 APPROVED BY \_\_\_\_\_





- 1) Powder Factor is a function of powder and velocity
- 2) When the velocity is increased the powder charge placed at the very large
- 3) when the distance increases & charge placed at distance when same velocity charge placed in the large distance
- 4) charge is placed in small & distance decreases in increasing powder
- 5) Powder factor is less at low velocity

# STRATIGRAPHY

- EXISTING ROCK FILL  
VERY LOOSE DARK BROWN ORGANIC SILT  
VERY SOFT GREY-BROWN SILTY CLAY  
LOOKS TO COMPACT GREY SILTY CLAY TO SANDY SILT  
LOOSE TO DENSE FINE TO COARSE SAND AND GRAVEL  
HARD TIGHT GRANITIC BEDROCK

SECTION ~~A-A~~  
SCALE 1" = 20' - 0"

DEPARTMENT OF HIGHWAYS, ONTARIO TORONTO	PROPOSED BRIDGE HAINES CUP OF LAKES AND REVISSED HIGHWAY 29 TOWNSHIP OF MUMFORD, ONTARIO PROPOSED BLASTING & FILLING PROCEDURES	GEOCON LTD DATE: FEB. 12/80 SCALE: AS SHOWN DRAWN BY: [Signature] No. 57045-2
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QUANTITIES

PRICES

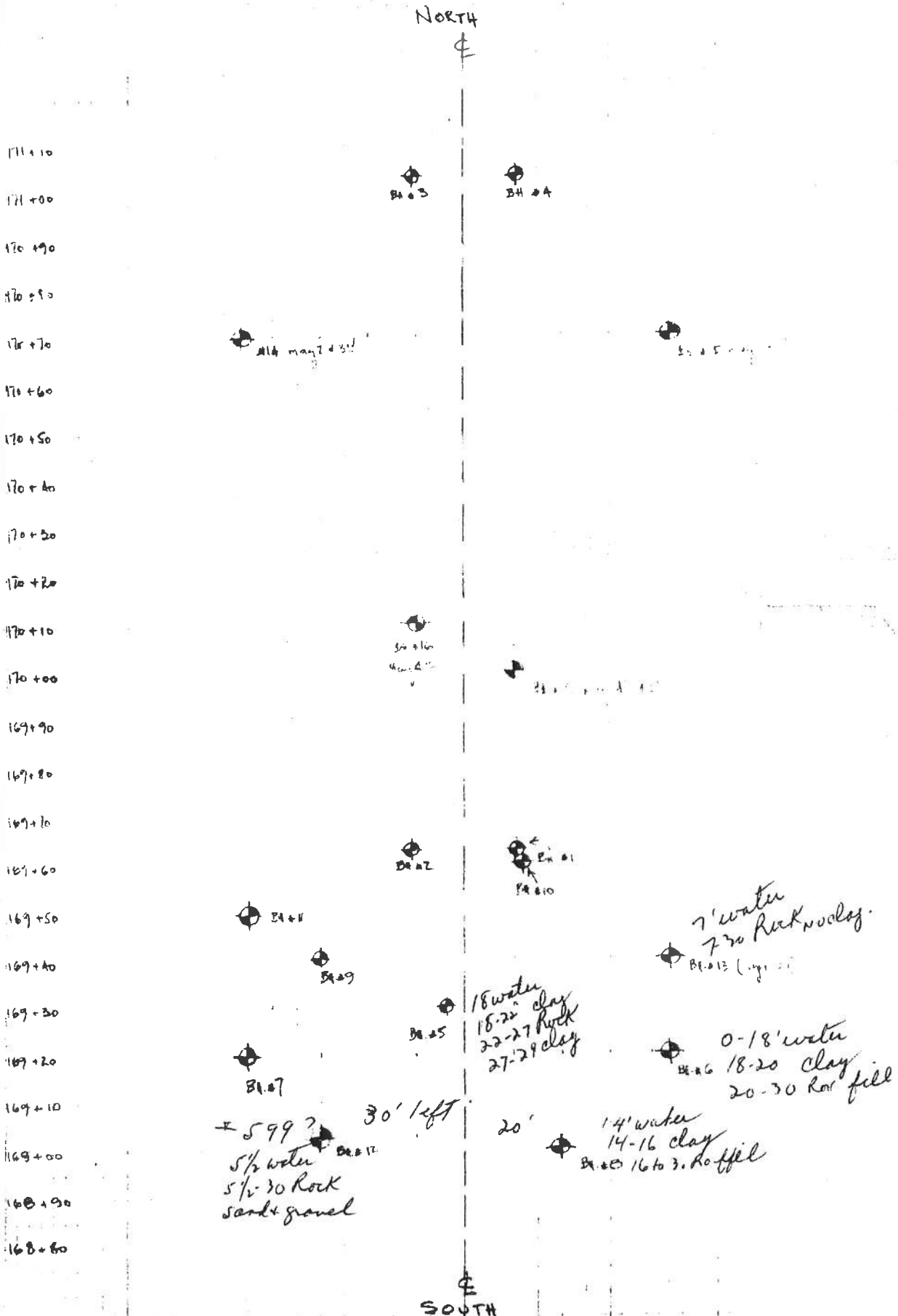
EXTENSIONS

647

APR 1960

S-7064 NAISCOOT BRIDGE SITE BLA...

SKETCH OF TEST PROBINGS (E...)

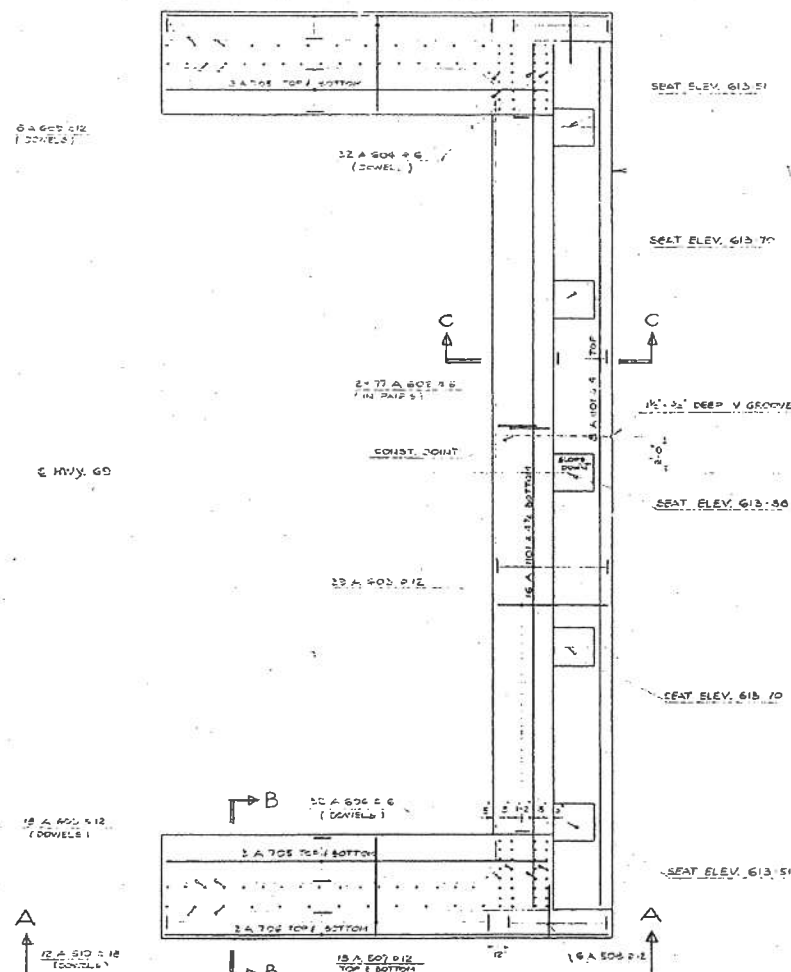


## **Appendix B**

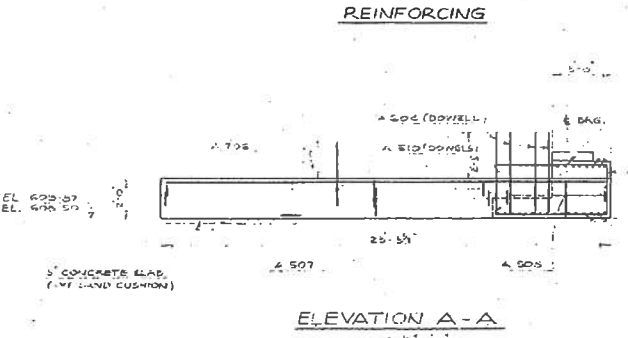
### **Existing Structure Drawings**

SHEET No.	TOTAL SHEETS
2	5

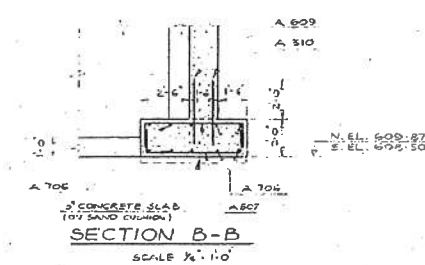
12 A 512 12 (DOVIELLS)  
2 A 705 TOP BOTTOM  
12 A 507 12 TOP BOTTOM  
12 A 508 12



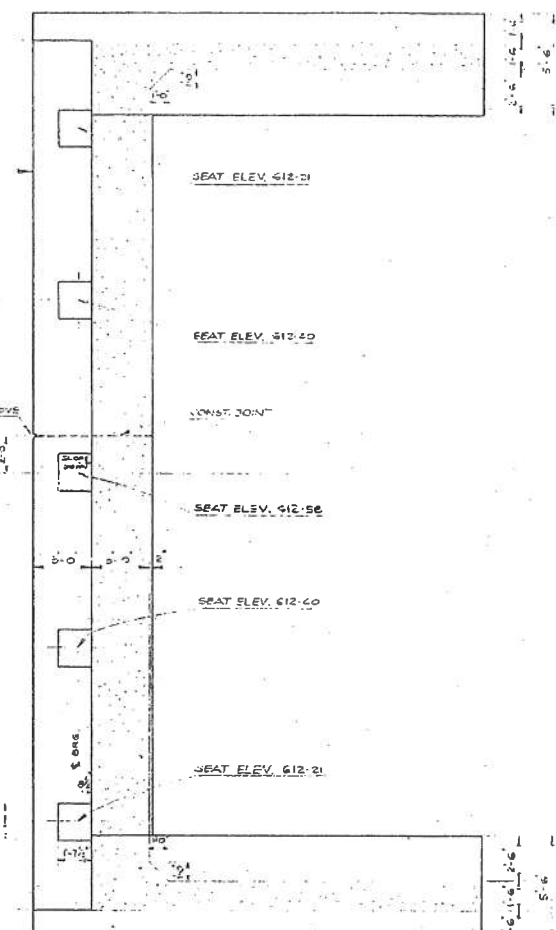
**LAYOUT OF FOOTINGS**  
SCALE 1/4" = 1'-0"



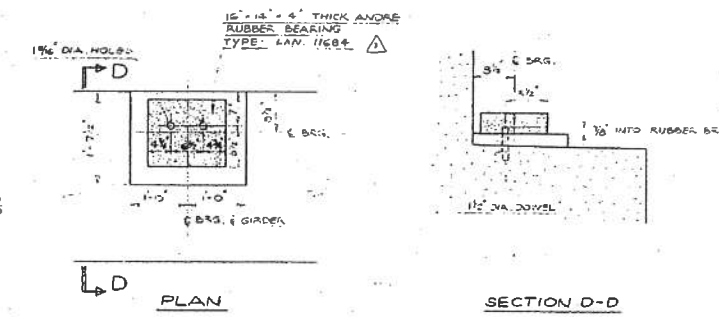
**ELEVATION A-A**  
SCALE 1/4" = 1'-0"



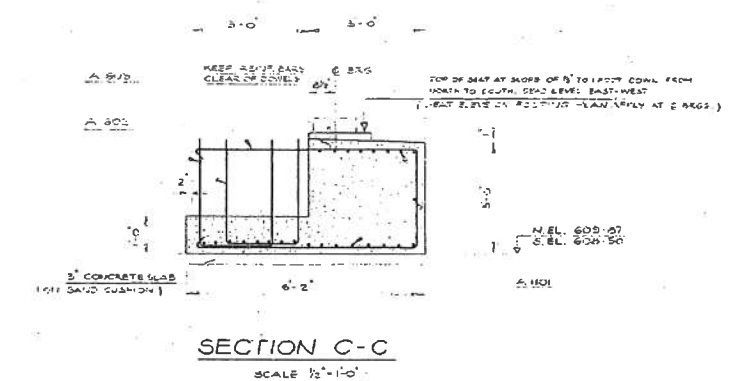
**SECTION B-B**  
SCALE 1/4" = 1'-0"



**DIMENSIONS**



**BRG. SEAT DETAIL**  
SCALE 1/4" = 1'-0"



**SECTION C-C**  
SCALE 1/2" = 1'-0"

4-6-62	B.S.R.	BEARING SEAT ELEVATIONS RAISED, SLOPE ON SEAT PROVIDED
11-22-66	J.G.G.	BEARING TYPE REVISED L.A.N. 11686
1-9-68	J.G.G.	RUBBER BEARING TYPE CHANGED IN SEAT
DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO BRIDGE DIVISION			
<b>NAISCOT RIVER BRIDGE</b>			
KING'S HIGHWAY No. 69 T.C.M.		DIST. No. 17	
CO. PARRY SOUND		LOT 26 CON. 2	
FOOTINGS, DIMENSIONS & REINFORCING			
APPROVED	W.P. No. 504-57		
DESIGN	CHECK	CONTRACT	
DRAWING	W.L.	LOADING	
DATE	JAN. 62	DRAWING	

TWP# 454-43-2-2



## **Appendix C**

### **Site Photograph**

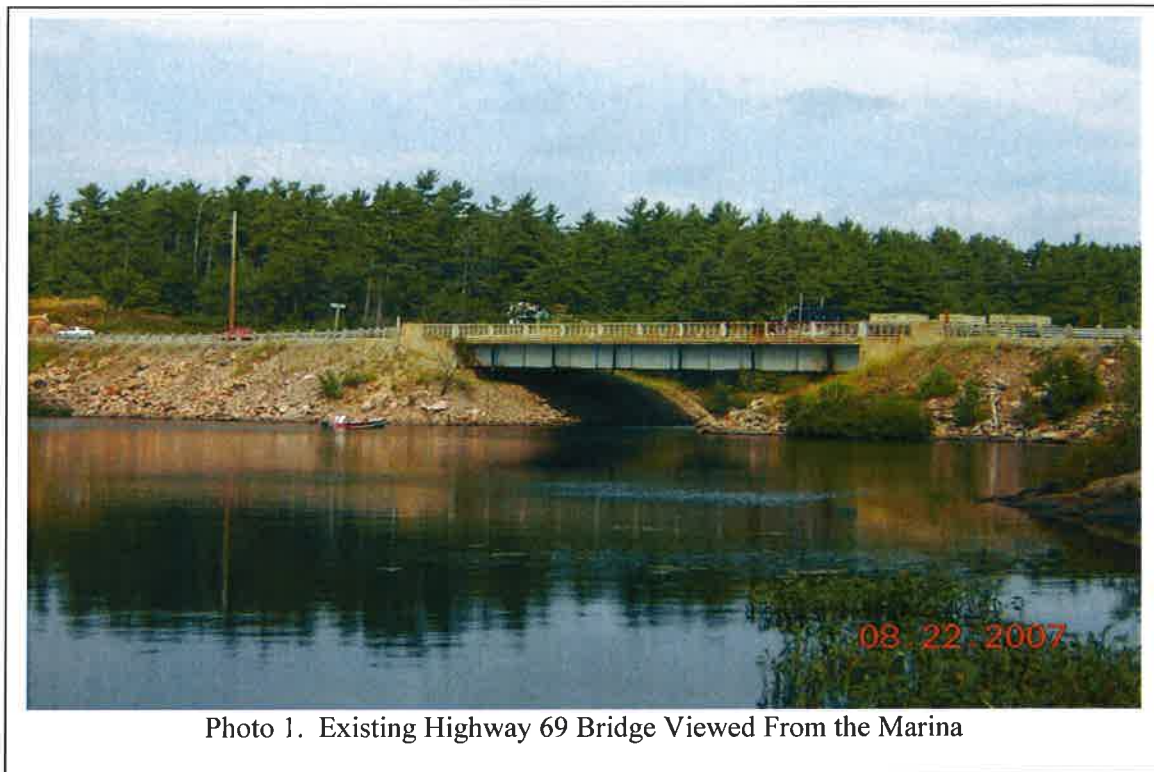


Photo 1. Existing Highway 69 Bridge Viewed From the Marina

## Appendix D

Technical References and Suggested Text for Selected NSSP



**List of Special Provisions and OPSS Documents Referenced in this Report**

- OPSS 902
- OPSD 3101.150
- SP 110F13 Amendment to OPSS 1010, April 2004

**2. List of Canadian Highway Building Design Code References in this Report**

- Clause 6.7.4
- Clause 6.9.3