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KINGSTON MILLS SWING BRIDGE

RIDEAU CANAL

GEOTECHNICAL INVESTIGATIONS

1999

QUONTACON ASSOCIATES

KINGSTON MILLS SWING BRIDGE

RIDEAU CANAL

KINGSTON, ONTARIO

GEOTECHNICAL INVESTIGATION REPORT

1999

PROJECT: 51173-30001725

QUONTACON ASSOCIATES
January 2000

TABLE OF CONTENTS

SECTION 1

INTRODUCTION

SECTION 1 - INTRODUCTION

1.1 Authorization and Scope of Work

In November 1999, Quontacon Associates was authorized by Parks Canada, Government of Canada, Cornwall, Ontario to undertake Survey and Geotechnical Investigation at Kingston Mills Swing Bridge Foundations, Rideau Canal.

The following were the terms of reference for this work :

Scope of Work

Arrange a site meeting with PWGSC and Rideau Canal Administration to discuss the specific requirements of this work. To conduct geotechnical and site investigations, field surveys and lab tests as well as other work that may be required in order to produce the final geotechnical report and provide information for design.

Field Survey

.1 Provide a detailed structural and topographic field survey of the bridge abutments and pivot pier. Take measurements of the abutments; all rail and rail components; anchors in the pivot pier.

.2 It is not required to survey and measure the actual bridge and bridge components.

.3 Take spot elevations on the canal walls; bridge abutments and grade in the vicinity of the bridge (both sides of the canal).

.4 Provide a site plan indicating all spot elevations taken; all measurements of the bridge abutments; track and track components. Provide drawings, plan and sections.

.5 All drawings for the bridge and field survey are to be done on AutoCad version 12 or better. Provide four hard paper copies of the drawings and also a 3 1/2 inch diskette of the drawings at completion.

Geotechnical Investigations

.1 Conduct vertical coring investigation and lab testing as follows :

- Drill 6 vertical holes, to obtain 100 mm diameter cores through the abutments and pivot pier foundation to a depth of 1 metre into bedrock. Drill 1 vertical hole through the bridge deck into the concrete bridge counterweight.
- 4 vertical holes will be placed in the bridge abutments, one at each corner of the bridge. Place each hole 15 cms in from outside face of the abutment.
- 2 vertical holes will be placed in the concrete pivot pier foundation adjacent to the circular track beneath the bridge. On the north side a small section of the steel grating will have to be removed for drilling access. Do not cause any damage to the grating in its removal and reinstallation after drilling.
- 1 vertical hole will be placed into the concrete counterweight at the west end of the swing bridge. This will be placed through the asphalt roadway.

.2 Provide a site plan of the work area and indicate the location of each bore hole on the plan. (The exact location of each bore hole will be determined on site by the Consultant). The number, depth and location of the bore holes may be altered by Public Works, depending upon the initial coring results.

.3 For bore holes drilled into concrete and bedrock (including the counterweight), determine

.1 the quality of the bedrock/ concrete interface;
.2 the RQD factor and record it in the boring log ;
.3 air entrainment in the concrete;
.4 the compressive strength of the concrete;
.5 the effect and presence of any alkali-aggregate reactivity. Comment on the severity of the reaction in regards to accelerating the deterioration of the abutments;
.6 the quality of the concrete of the abutments and counterweight;
.7 Log the presence of all voids, discontinuities, seams and joints encountered during coring in the concrete abutments and pivot pier.

.4 Establish geodetic elevations for all bore holes by field leveling. Reference the bore hole elevations to a fixed reference point with field measured dimensions and indicate these measurements on the site plan.

General Requirements

.1 All data and work must be in the metric system.
.2 Notify the Rideau Canal office in Smiths Falls in advance of starting geotechnical work on site.
.3 All drilling to be supervised by an experienced Soils Technician, under the general supervision of a Geotechnical Engineer registered with the PEO.
.4 Restore all damage caused during this contract to match the original conditions, including re-sodding of grassed areas. all tire ruts in grass are to be re-sodded.
.5 Bore holes in masonry and concrete must be filled with a non-shrink cement grout. Holes in asphalt must be compacted and leveled with asphalt.
.6 Holes in soil and overburden must be backfilled, compacted and leveled to match the surrounding grade.
.7 Remove all debris from the site on a daily basis.
.8 Allow no oil, gasoline, grout, debris or other contaminants to enter the water. No burying or burning on site is allowed.
.9 When drilling through the bridge deck into the concrete counterweight, do not block traffic crossing the bridge. Use barriers, warning signs and a traffic control person to direct on-coming vehicular traffic. Place the hole to one side of the bridge and allow traffic to pass on the other.

Schedule

.1 Start work immediately upon notice of award of contract.
.2 All work should be completed and final copies of report sent to Public Works no later than 6 weeks after the site work was started.
.3 All work must be fully completed and invoiced by December 31,1999.
.4 Make available to Public Works the drilling logs and results of both field and laboratory analysis upon request. Consultant may be asked to fax copies of the field results to PWGSC as they become available, as engineering design may be proceeding concurrent with the investigations.

Reports

Provide four copies of the final report containing the following :

.1 a site plan indicating the exact location of each bore hole and all dimensions and elevations recorded in the field. Indicate the location and elevation of the fixed site reference point;

.2 bore hole logs; results of lab testing;

.3 compressive strength results of the samples taken and air entrainment content;

.4 comment on the severity of the alkali-aggregate reactivity within the concrete and its long term effect upon the deterioration of abutments;

.5 discuss coring results, voids, seams and any discontinuities found during the drilling. Record locations where drill water is lost in cavities, voids, or where washouts occur during the drilling and comment on the possible cause of the loss;

.6 the report must be prepared by a Geotechnical Engineer registered in the province of Ontario with the PEO;

.7 final copies of the report are to be sent to RPS CH/EC PWGSC in Cornwall. Forward copies to:

Public Works and Government Services Canada
Real Property Services
111 Water Street East, 3rd Floor
Cornwall, Ontario
K6H 6S3

Attention: James Richardson, P. Eng.

1.2 Location

The area for the investigation is located at the Swing Bridge, Kingston Mills Lock on the Rideau Canal. Access is from highway #15, just north of the #401.

1.3 Background

The bridge abutments and pivot pier foundations are constructed of reinforced concrete and have deteriorated severely. The bridge abutments

are badly cracked and efflorescence is visible, caused by the use of the de-icing salts on the bridge deck during winter.

In January 1977, a site investigation behind the east abutment was carried out by Site Investigation Services Limited, Peterborough , Ontario for J. D. Lee Engineering Limited, Kingston, Ontario. Investigations indicated that the east lock wall abuts against the face of rock cut. Bedrock cores consisted of pinkish to reddish grey syenite. Top 3 metres of rock are partially weathered along horizontal and vertical joints. At greater depths, the rock is sound and massive.

The original foundation of the bridge was constructed in 1955 and the rehabilitation of the concrete of the foundation was carried out in 1977.

SECTION 2

SITE INSPECTION AND PROCEDURE

SECION 2 - SITE INSPECTION AND PROCEDURE

2.1 Introduction

The systematic visual inspection of the site and appurtenant structures is an essential part of any investigation. This inspection forms the basis for all geotechnical investigations, survey requirements and review. The site inspection was carried out by the following personnel :

James Richardson, P. Eng.	Public Works and Government Services
Eric Sunstrum, P. Eng.	Public Works and Government Services
Joe Bennett, P. Eng.	Inspec-Sol Inc., Kingston
George Bracken, OLS	George Bracken Ltd., Ontario Land Surveyors
M. H. Rehman, P. Eng.	Quantacon Associate, Kanata,Ontario
George Hallett	Quantacon Associates, Kanata, Ontario

The inspection took place on November 18, 1999.

2.2 Swing Bridge Foundations

In consultation with the Public Works personnel, the location of core holes in the abutments, pivot pier and the counterweight were decided. The location of holes was marked by Mr. Bennett. There would be one hole in each abutment, two inclined holes in the pivot pier , close to the track and two holes in the counterweight. The holes in the abutments will extend one metre into bedrock. The holes in the pivot pier will be through reinforced concrete and stopped if rockfill is encountered instead of bedrock. One hole through the counterweight will be one metre deep and the second one only 300 mm.

2.3 Field and Lab Work

Topographic survey and leveling work was carried during the week of November 21 by George Bracken Limited, Ontario Land Surveyors, Smiths Falls, Ontario.

The geotechnical work (concrete coring, field and lab testing) was sub-contracted to Inspec-Sol Inc., Kingston, Ontario. The work was carried out under direct supervision of Mr. R. McLachlan, P. Eng. The field work was carried out November 29, 30 and December 1,1999.

Mr. M. H. Rehman, P. Eng. provided the general direction and supervision and determined the exact location of core holes. Analysis and review of the observations, geotechnical data, and results of lab tests was carried out in December 1999 and January 2000.

The factual report by Inspec-Sol Inc. forms Appendix 'A'.

SECTION 3

GEOTECHNICAL SITE INVESTIGATIONS

SECTION 3 – GEOTECHNICAL SITE INVESTIGATIONS

3.1 Survey and Topographic Plan

Surveying and leveling of the site was carried out by George Bracken Limited, Ontario Land Surveyors, Smiths Falls, Ontario. The work was carried out on November 21, 1999. The topographic plan and sections of the east and the west abutments, the pivot pier were prepared as required in the terms of reference, covering all the areas and providing the details and information outlined. The location of core holes is shown on the plan attached to this report. The elevations of abutments and pivot pier have been verified, indicating no change from the previous drawings (1977).
A copy of the drawings has been transferred to a 3 ½ inch diskette (CAD Version 14) which is attached.

3.2 Geotechnical Investigation Program

3.2.1 General

A geotechnical investigation was carried out at the abutments, the pivot pier and the counterweight of the Swing Bridge, Kingston Mills Lock, Rideau Canal from November 29 to December 1, 1999 The main objective of the investigation was to determine :

1. the condition of concrete of the abutments, the pivot pier and the counterweight by taking 100 mm diameter cores;
2. compressive strength of concrete;
3. air-entrainment;
4. alkali aggregate reactivity if present and its severity;
5. presence of voids, seams, cracks, joints in concrete and rock foundation;
6. nature of concrete/bedrock interface;
7. bedrock type, properties and Rock Quality Designation (RQD).

The investigation work was carried out by Inspec-Sol Inc., Kingston, Ontario. The drilling work was sub-contracted to OGS Inc. A factual report of their work forms Appendix "A" of this report. It contains a detailed description of the concrete cores, core logs, laboratory test results and photographs. The location of the cores is shown on the drawing K6160-1.
A total of six (6) concrete cores, 100 mm diameter were obtained :
One (1) in the west abutment, south side.
One (1) in the east abutment, north side.
Two (2) in the pivot pier (one each south and north sides).
Two (2) in the concrete counterweight.

A portable electric drill with core barrel was utilized by OGS Inc. (Sub-contractor) for the drilling work.

3.2.2 Condition of concrete

In general, concrete in the abutments and pivot pier appeared to be in good condition, with micro-fissures both open and closed present in the cement paste and coarse aggregate and reaction rims visible around some limestone aggregate. RQD calculated ranged from 76% to 100% indicating good to excellent quality. The compressive strengths ranged from 24.5 MPa to 45.8 MPa.
The concrete of the counterweight did not exhibit any micro-fissures or reaction rims and the compressive strength was found to be 17.4 MPa.
All the core locations are shown on Plan K6160-1 and the cores are detailed in the core logs and the photographs (see Appendix "A").

Core # C1 was taken in the west abutment (south side). There is a presence of micro-fissures in the cement paste and limestone aggregate and reaction rims are visible. There does not appear to be any bond between syenite blocks (up to 170 mm size) with the cement paste; the syenite blocks may have been thrown into the concrete pour to minimize the quantity of concrete and were not cleaned of dust which might explain the lack of bond.

The abutment is founded on bedrock (syenite) and the interface between concrete and bedrock is tight. The core penetrated 700 mm into bedrock before terminating the hole.
The total length of core was 3.05 m.

Core # C2 was taken in the east abutment (north side). The top 300 mm of the core exhibits many open micro-fissures in the cement paste. There are micro-fissures in the cement paste and through limestone aggregate and reaction rims are visible in the upper 1,800 mm. Honeycombing is present in the bottom portion of the core. Syenite blocks 100 to 300 mm were found throughout the length of core without any bond with the cement paste. Due to the proximity of the lock wall, it was discovered that the abutment was founded on rockfill (backfill for the lock wall) and not on bedrock and for this reason the coring was discontinued.
The length of core was 3.75 m.

Cores # C3 and C4 were taken in the counterweight. There was about 50 mm of asphalt overlying concrete. No waterproofing membrane was found. There are no micro-fissures or reaction rims visible in the core.
The lengths of cores C3 and C4 were 300 and 400 mm respectively.

Core # C5 was taken in the pivot pier (north side). The coring was done on an incline, similar to that on the south side. The top 50-75 mm was totally disintegrated concrete. Open micro-fissures in the cement paste and coarse aggregate in the top 300 mm and closed micro-fissures to a depth of 1,300 mm were found. There were syenite cobbles also present. It was surprising to discover that the pivot pier is founded on a layer of medium to stiff clay with traces of gravel overlying bed rock. A split-spoon sampler was penetrated through clay to refusal in bedrock. Rock core could not be obtained due to presence of clay layer.
The total length of core was 1.98 m.

Core # C6 was taken in the pivot pier (south side) at an incline to the vertical (1in 5). The top 50-75 mm of concrete was found to be unsound (disintegrated), the remaining core appeared to be good except the presence of open and closed micro-fissures in the paste and coarse aggregate. Some reaction rims also visible. Like on the north side, the pivot pier was founded on a 300 mm layer of stiff to medium clay with traces of gravel over the bedrock. A split-spoon sampler was used to penetrate the clay until refusal in bedrock. It was not possible to obtain a rock core.
The total length of core was 1.61 m.

3.2.3 Test Results

.1 Five (5) compressive strength tests were done on samples from cores C1, C2, C4, C5 and C6. The compressive strength ranged from 24.5 to 45.8 MPa except for the sample C4 (counterweight) which had compressive strength of 17.4 MPa, being only fill- type concrete.
The concrete from cores C5 and C6 may be classified as 35 MPa concrete. The concrete from cores C1 and C2 may be classified as 30 MPa concrete. The compressive strength of the samples from C1, below the zone of open micro-fissures had a strength of 28.6 MPa, whereas the sample taken from core C2 in the zone of micro-fissures had a strength of 24.5 MPa. It appears that the micro-fissures due to alkali aggregate reactivity in core C1 have resulted in some loss of mechanical strength.
For details see Appendix "A".
.2 Two (2) samples from cores C5 and C6 were tested for tensile strength. The tensile strength was 1.85 and 1.53 MPa, which is within the acceptable limits.

.3 Two (2) samples from core C5 were tested for air-entrainment. Air void content was found to be less than 1%.
The spacing factor, L, was determined to be 1,237 um (core C6) and 1,311 um (core C5), which does not conform to CSA A23.1-94, Section 14.3.4 for concrete exposed to freeze-thaw cycles and de-icing salts. Under these conditions, the spacing factor should not exceed 230 um. The voids observed in the concrete cores are pores and entrapped air.

.4 Alkali Aggregate Reactivity

There is evidence of alkali aggregate reactivity exhibited by the presence of open micro-fissures and visible reaction rims. The loss of mechanical strength in the zone of micro-fissures in the sample from core C1 can be attributed to alkali aggregate reactivity.

A petrographic analysis on two (2) samples from cores C2 and C5 was carried out in accordance with ASTM C856-83 "Standard Practice for Examination of Hardened Concrete." The concrete was examined for mineralogy, texture, fabric, bond of coarse aggregate with matrix, and presence of alkali-silica reaction. The concrete is composed of coarse aggregate consisting of dolomite and micritic limestone. Some of the micritic limestone has undergone dolimitization and exhibits microtextures typical of an aggregate susceptible to alkali-silica reaction. Microfissures, some filled with silica gel were observed around the micritic limestone coarse aggregate grain boundaries and through the cement paste.

SECTION 4

DISCUSSION AND RECOMMENDATIONS

SECTION 4 – DISCUSSION AND RECOMMENDATIONS

4.1 Concrete quality

Core recovery was excellent with 100% return on all bore-holes. Quality was assessed by determining RQD (Rock Quality Designation). RQD calculated on concrete cores varies from 76 to 100%, indicating good to excellent quality. The compressive strength of concrete in the abutments and the pivot pier ranged between 30-40 MPa. The concrete in the counterweight was of fill-type with a strength of 15-20 MPa.
The tensile strength from cores C5 and C6 ranged from 1.5-1.8 MPa, which is within the acceptable limit (1.5 MPa).

4.2 Air-entrainment

The concrete is considered to be normally consolidated with good distribution of aggregate and the cement paste. However, the air-entrainment within concrete is less than 1% and the spacing factor, L, (1,200-1,300 um) was much greater than 230 um, the limit for concrete exposed to freeze-thaw cycles and de-icing salts. Local honeycombing and air voids were noted in C2 and C5.

4.3 Aggregates

The coarse aggregate consists of crushed stone composed of micritic limestone, crystalline limestone and argillaceous limestone, with a nominal size of 20 mm. The fine aggregate is 0-5 mm sub-angular to sub-rounded, of granitic origin. Numerous cobbles and blocks of syenite up to 170 mm in diameter were observed in all bore holes except C3 and C4. There is little or no bond between the syenite blocks and cement paste. The concrete paste and some of the coarse aggregate was cut by numerous white micro-fissures. These micro-fissures are typically open (1.5 to 3.0 mm) and are more numerous in the upper portion of the cores, whereas they decrease in number and become closed (tight) with depth. Reaction rims were also noted around limestone aggregate.

4.4 Alkali Aggregate Reactivity

The Kingston area limestone aggregates are known to have a siliceous component. This consists of very minute grains of quartz in the limestone matrix,

which can react with the cement paste and cause alkali aggregate reactivity. The presence of reaction rims, deposits of silica gel, with numerous micro-fissures visible in the cores (except C3 and C4) are typical of this reaction.
It can be concluded from the petrographic analysis of the hardened concrete that the alkali aggregate reactivity is actively taking place due mainly to the presence of coarse aggregate susceptible to the reaction and minute grains of quartz in the limestone matrix which reacts with the cement paste. The cracks in the exposed concrete due to freeze- thaw action (negligible air entrainment) and use of de-icing salts provide enough moisture for the reaction to progressively continue and flourish.

4.4 Counterweight

Low strength concrete (fill-type) of the counterweight appears to be in good condition. However, there is no bond between the asphalt and concrete and no waterproofing membrane was present.

4.5 Pivot Pier

The pivot pier, surprisingly, is founded on a layer (300 mm) of compact, dense, moist, brown clayey silt with traces of gravel overlying the bedrock. No settlement/ movement was noted in the elevations recorded (compared to 1977).

4.6 Recommendations

Despite the alkali aggregate reactivity taking place, it can be concluded from the test results that the compressive strength of concrete in the abutments and the pivot pier is adequate. Although there is some loss of mechanical strength in the areas exposed to de-icing salts, resulting in map cracking and micro-fissures, the tensile strength is still within the acceptable limits. The micro-fissures which are open in the upper portion of the cores become closed with depth.

There are two options for rehabilitation:

Option 1

Remove track from pivot pier, remove deteriorated concrete from the abutments and pivot pier, install reinforcing steel anchors and reface with quality, air entrained 35 MPa concrete, coat the exposed surfaces of concrete with an approved epoxy sealer, and reinstall the existing , cleaned track with new anchors. Estimated cost $ 160,000.

Option 2

Demolish and rebuild the two abutments and the pivot pier. In this option, a new hydraulic machinery for the movement of the Swing Bridge can be incorporated. Estimated cost $ 300,000.
(excludes the cost of new hydraulic machinery)

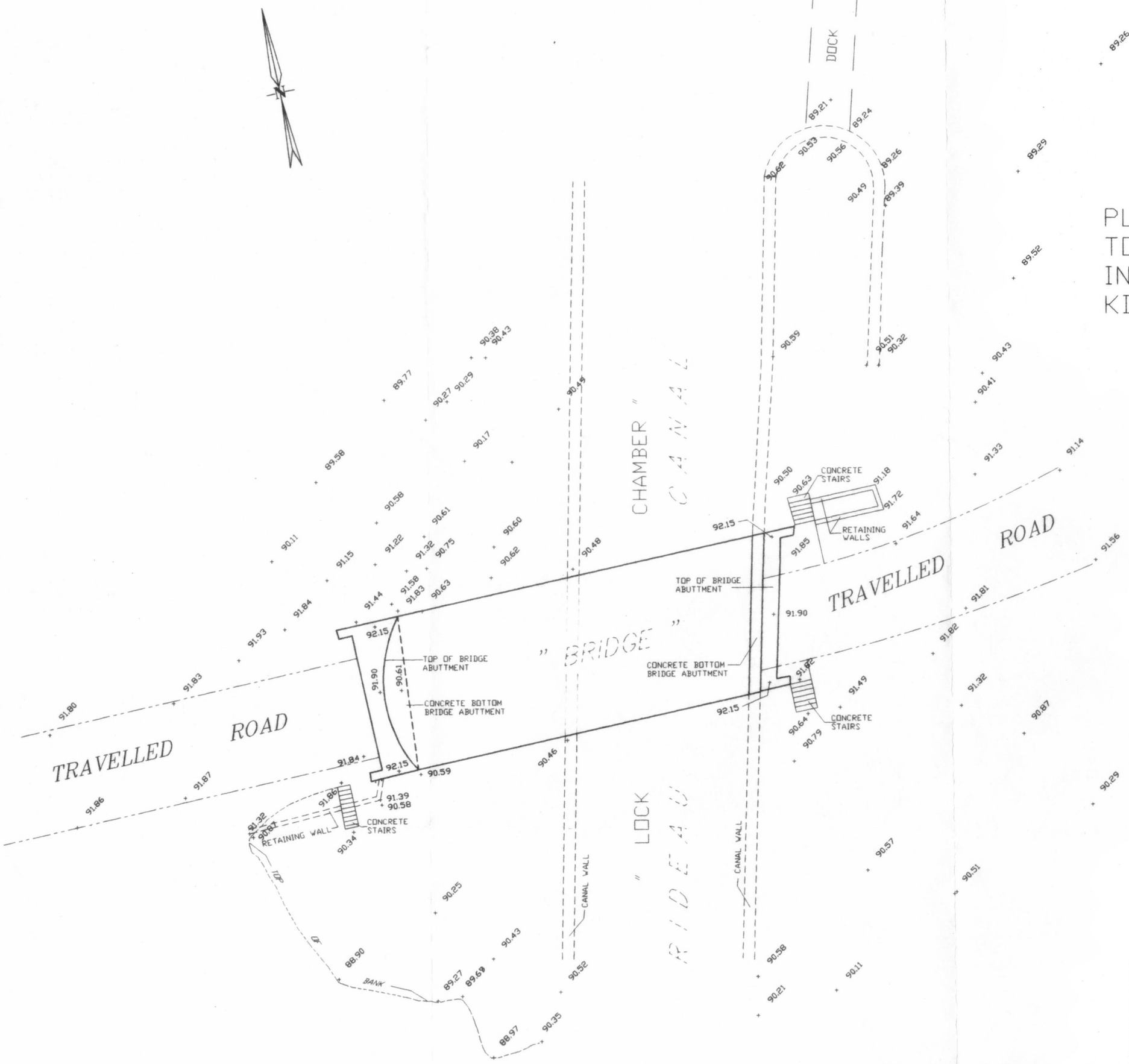
This report recommends Option 1. Based on past experience, the previous rehabilitation carried out in 1977 has served well for the past 22 years, despite concrete, not having air entrainment according to CSA Standards. Refacing with well-consolidated, quality concrete having air-entrainment according to CSA Standards and sealed with an approved epoxy coating should be considered. In the absence of water infiltration, the progression of alkali aggregate reactivity will be significantly arrested. Hence this report recommends Option 1. It restores the integrity of the Swing Bridge foundation at a minimal cost.

SECTION 5

PLANS

PLAN SHOWING TOPOGRAPHICAL INFORMATION ON THE KINGSTON MILLS LOCKS

SCALE 1:200 (METRIC)



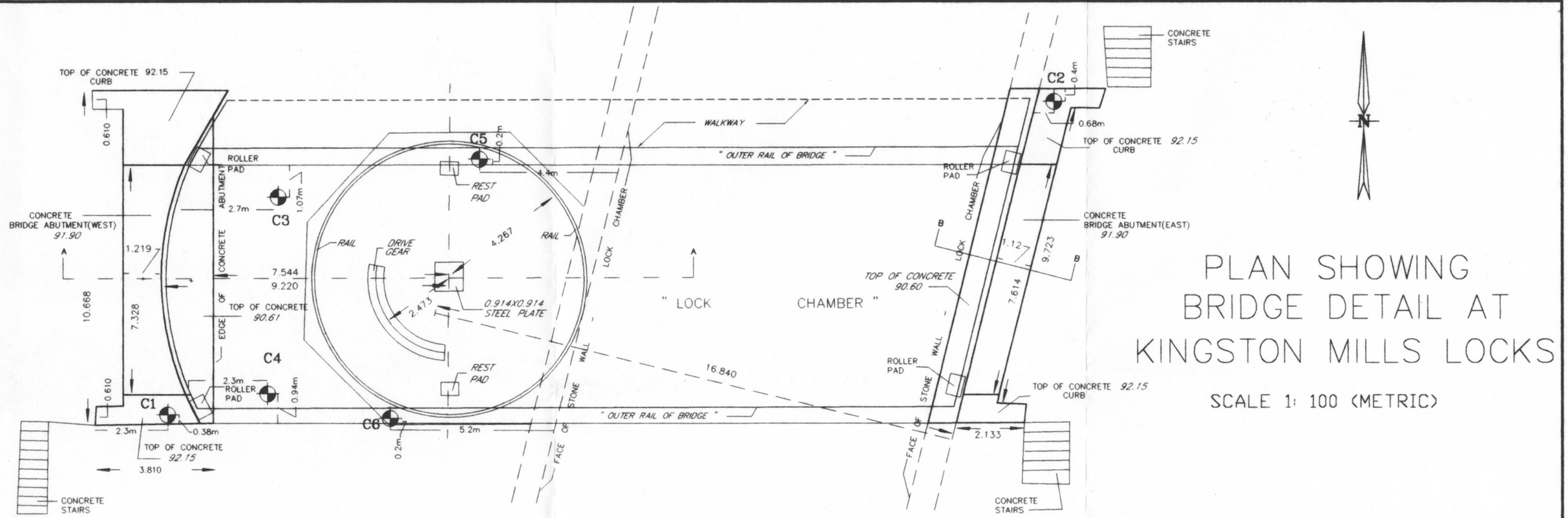
BENCH MARK

ELEVATIONS ARE GEODETIC AND WERE DERIVED FROM BENCH MARK No.700-G, A BRONZE TABLET 15.54 METRES EAST OF EASTWALL OF THE UPPER LOCK AND 20.42 METRES NORTHEAST OF EAST END OF THE SWING SWING BRIDGE OVER THE RIDEAU CANAL IN KINGSTON MILLS. TABLET IN THE WEST FACE OF LOW WALL OF ROCK, 0.405 ABOVE GROUND.

George Bracken Limited
ONTARIO LAND SURVEYORS
40 MAIN STREET, WEST
SMITHS FALLS, ONTARIO
K7A 4S9
PHONE: (613) 283-2233 FAX: (613) 283-6886

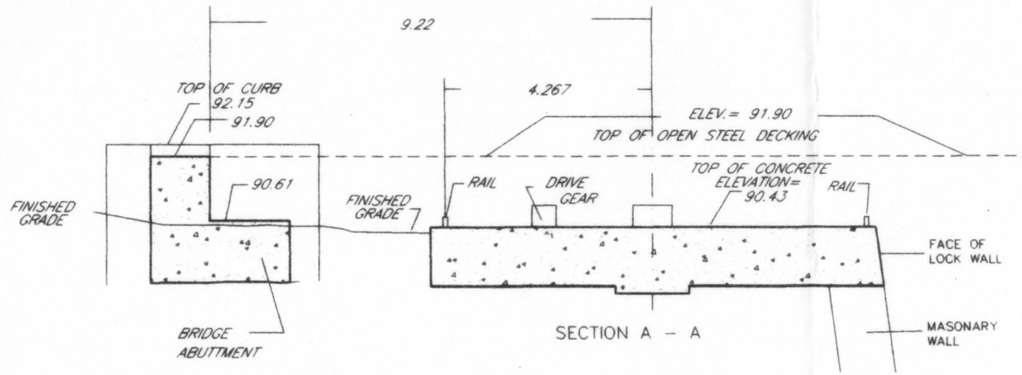
DRAWN BY: John R.Wanless C.S.T.	CHECKED BY: GNB
FILE: K-MILLS	JOB No.: B - 942
DATE: JANUARY 19th.,2000	

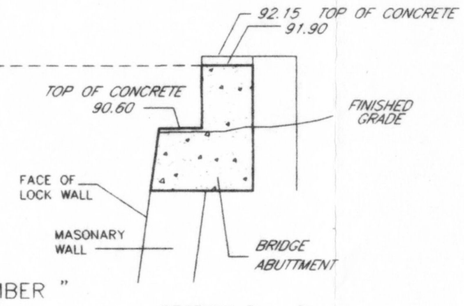
(to M.W.T.)



PLAN SHOWING BRIDGE DETAIL AT KINGSTON MILLS LOCKS

SCALE 1: 100 (METRIC)





NOTE:

C2 - DENOTES BOREHOLE
LOCATION OF BOREHOLES WERE TAKEN FROM PLAN
PREPARED BY INSPEC -SOL DATED DEC./1999
PLAN No. K6160-1

George Bracken Limited
ONTARIO LAND SURVEYORS
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FILE: K-MILLS	JOB No.: B-942(A)
DATE: JANUARY 19th., 2000	

SECTION 6

APPENDIX "A"

INSPEC-SOL INC. REPORT

INSPEC-SOL INC. 9-626 Cataraqui Woods Dr., Kingston, Ont. K7P 1T8 Tel.: (613) 389-9812 Fax: (613) 389-5287

QUONTACON ASSOCIATES
CONCRETE CORES
KINGSTON MILLS LOCKS – RIDEAU CANAL
KINGSTON, ONTARIO

DECEMBER, 1999

INSPEC-SOL INC. 9-626 Cataraqui Woods Dr., Kingston, Ont. K7P 1T8 Tel.: (613) 389-9812 Fax: (613) 389-5287

Reference No. 20226-K6160-B

December 20, 1999

Quontacon Associates
112 Moresby Drive
Kanata, Ontario
K2M 2J6

Attention: Mr. Mumtaz H. Rehman, P. Eng.

RE: Concrete Coring
Kingston Mills Locks - Rideau Canal,
Kingston, Ontario

Dear Sir:

We have completed the fieldwork for the above captioned project and present our findings in the following report.

The scope of our work included the retrieval of six cores of the concrete from the abutments, the central pivot pier foundation and the cantilever weight. The cores were then to be logged and described for condition, types of deficiencies, inclusions, and evidence of alkali-aggregate reactivity. Samples of the cores were also to be tested for compressive strength, tensile strength, air void content and alkali-aggregate reactivity.

Fieldwork

The fieldwork was completed on November 22 and 23, 1999. We completed a total of six cores whose locations and depths are described in the following Table 1. The attached plan K6160-1 shows the site and borehole locations. The detailed logs of the boreholes are attached as well as photographs of the core samples.

Table I
Concrete Core Information

Borehole Number	Location Description	Depth (ft)	Comments
1	South-West Abutment	10	concrete throughout core length
2	North-East Abutment	12.3	bottom of core penetrated cemented cobbles
3	Cantilever Weight, West Bound Lane	1	2 inches of asphalt over concrete, concrete fairly sound
4	Cantilever Weight, East Bound Lane	1.3	1.5 inches of asphalt over fairly sound concrete
5	Pivot Pier, North Side	7.5	top 2 to 3 inches of concrete comprised of un-cemented material
6	Pivot Pier, South Side	6.3	top 2 to 3 inches of concrete comprised of un-cemented material

LABORATORY RESULTS

The following table presents the results of the testing completed on selected samples from the concrete cores.

Table II
Laboratory Results

SAMPLE	TYPE OF TEST	RESULTS	COMMENTS
BH-1 @ 2'8"	Compressive Strength*	28.6 Mpa	acceptable break
BH-2 @ 1'2"	Compressive Strength	24.5 Mpa	acceptable break
BH-4 @ 2"	Compressive Strength	17.4 Mpa	acceptable break
BH-5 @ 2'5"	Compressive Strength	45.8 Mpa	acceptable break
BH-6 @ 2"	Compressive Strength	36.3 Mpa	acceptable break
BH-5 @ 3'1"	Tensile Strength**	1.85Mpa	
BH-6 @ 3'10"	Tensile Strength	1.53Mpa	
BH-5 @ 2'2"	Air Void Count***	<1%	
BH-5 @ 2'	Air Void Count	<1%	

*CSA A23.2-14C Obtaining and Testing Drilled cores for Comp. Str. Testing
** CSA A23.2-6B Method of Test to Determine Adhesion by Tensile Load
***ASTM C457-90

The air-void system was determined in two samples following ASTM C457-90 ''Microscopical determination of Parameters of the Air-Void System in Hardened Concrete''

The results from the two samples indicate that the air-entrainment within the concrete is less than 1%. The spacing factor, L, was determined to be 1237 um (BH-6) and 1311 um (BH-5) which does not conform to CSA A23.1-94, Section 14.3.4 for concrete exposed to freeze and thaw and deicing salts. Under these conditions, the spacing factor should not exceed 230 um. The voids observed in the concrete cores are pores and entrapped air.

The tensile strength tests were performed in accordance with CSA A23.2-6B ''Method of Test to Determine Adhesion by Tensile Load''. The tests yielded results of 1.85 (BH-5) and 1.53 MPa (BH-6), for an average of 1.69 MPa. According to the literature, a tensile strength of 1.5 MPa is acceptable.

With the assumption that the concrete elements that make up the bridge structure were poured at the same time the following is noted:

Based on the laboratory testing, the Uniaxial Compressive Strength (UCS) of the concrete from BH-5 and BH-6 may be possibly classified as a 35 MPa concrete. The concrete from BH-1 and 2 likewise may be possibly be classified as a 30 MPa concrete. The counterweight concrete may be a low strength fill type concrete or a 15 to 20 MPa class concrete.

The compressive strength of the sample taken from BH-1, below the zone of open microfissures yielded a UCS of 28.6 MPa whereas the sample taken from BH-2 in the zone of open microfissures yielded a UCS of 24.5 MPa. It is apparent that the microfissures due to AAR in BH-1 have resulted in some loss of mechanical strength.

The tensile strength tests yielded values of 1.85 MPa and 1.53 MPa which equates to approximately 4% of the compressive strength values. Normal tensile strength values of concrete fall in the 5-7% range. Therefore, it appears that there has also been some loss of tensile strength due to the microfissuring caused by AAR.

COMMENTS AND DISCUSSION

Quality

Core recovery was considered to be excellent with 100% return on all boreholes. The quality was partly assessed by applying the Rock Quality Designation(RQD) which is the cumulative length of pieces longer than 4 inches divided by the total recovered length. Values of RQD calculated on the concrete cores, ranged from 76% to 100% indicating good to excellent quality.

Air Entrainment and Consolidation

There is considered to be no Air Entrainment as noted in the cores tested but there are entrapped air voids through the full depth of the concrete. The concrete was considered to be normally consolidated with a good distribution of aggregate and paste. Air voids up ¼-in. in diameter were noted in the concrete. Local honey-combing was noted at the bottom of BH-2 and BH-5, with voids up to 2-in. long and ½-in. deep. This may be due to poor consolidation. In general, the bond between the aggregate and paste is good.

Reinforcing

There was relatively little amount of reinforcing steel encountered in the cores. The bars that were intersected were deformed type and the bond between the reinforcement was good in BH-1 and 5, but no bond at all in BH-5 and 6. Corrosion of embedded reinforcement was light with no important loss of section.

Aggregate

The coarse aggregrate consists of crushed stone composed of micritic limestone, crystalline limestone and argillaceous limestone, with a nominal size of ¾-in. The fine aggregate is 0-1/4-in. sub-angular to sub-rounded, of granitic origin. Numerous cobbles and blocks of syenite up to 7-in in diameter were noted at various intervals down the length of all boreholes, except BH-3 and 4. In almost all cases where the syenite was encountered, there was little or no bond between the syenite and cement paste.

The concrete paste and some of the limestone coarse aggregate was cut by numerous thin white microfissures. These microfissures were typically open (1/16 to 1/8-in. in aperture) and more numerous in the upper portions of the boreholes, whereas they decreased in number and were very tight with depth. Reaction rims were also noted around several of the limestone aggregate. Based on our knowledge of the limestone aggregate in the Kingston area, there are limestone intervals in the quarries which have a siliceous component. This consists of very minute grains of quartz in the limestone matrix which can react with the paste and cause AAR. The presence of reaction rims, deposits of silica gel, and the numerous microfissures visible in the cores (except BH-3 and 4) is typical of this reaction.

Ballast Concrete

Asphalt was encountered in BH-3 and BH-4 which were put down through the driving deck into the ballast concrete deck. There was no bond between the asphalt and concrete, and no waterproofing membrane was present. The concrete appears to be a relatively low strength fill concrete. The concrete from these cores was considered to be in good condition with no obvious evidence of Alkali Reactivity.

Subgrade

Possible fill consisting of a brown dense, moist, clayey silt till with traces of gravel was encountered at the bottom of BH-5 and 6. The material was considered to be compact but it was not in the scope of this investigation to determine the strength properties or recommended bearing pressures of subgrade materials.

Summary Comments

Based on our observations, the concrete structure has been variably affected by Alkali Aggregate Reactivity. The upper exposed sections of the structure, such as the parapet walls, abutments and sidewalls of the canal which are above the general water level have been affected more severely. The exposure of the upper sections to air and cyclic wetting cycles and freeze-thaw has caused the micro-fissures to open up and eventually delaminate, particularly in the absence of air entrainment. This also allows more penetration of moisture into the concrete which promotes the AAR reaction.

Concrete which remains below the general water level will suffer AAR, but the reaction and expansion proceeds at the same rate. The deeper concrete may not show as significant signs of AAR due to the lack of moisture and weathering penetration.

There was no sign of AAR in the cores retrieved from the deck.

We trust this report provides the information require at this time. Please do not hesitate to contact our office should any additional questions arise, or should you require further inspection and testing services.

Yours very truly,
INSPEC-SOL INC.

Myles Carter, M.Sc.
Director-Bldg. Science

Joseph B. Bennett, P. Eng.,
Manager-Kingston

By fax/mail (514) 694-5835
JBB

INSPEC-SOL INC. 9-626 Cataraqui Woods Dr., Kingston, Ont. K7P 1T8 Tel.: (613) 389-9812 Fax: (613) 389-5287

Reference No. 20226-K6160-B

January 17, 2000

Quontacon Associates
112 Moresby Drive
Kanata, Ontario
K2M 2J6

Attention: Mr. Mumtaz H. Rehman, P. Eng.

RE: Petrographic Examination of Concrete Cores
Kingston Mills Locks - Rideau Canal,
Kingston, Ontario

Dear Sir:

The following report is an addendum to our previous submission. This current report presents the results of the petrographic examination that we have completed on two samples retrieved from the above captioned project.

The objective of this examination was to examine the concrete samples for evidence of alkali-aggregate reactivity. This report follows our previous report dated December 20, 1999.

SAMPLE PREPARATION

The petrographic analysis was carried in accordance with ASTM C856-83 "Standard Practice for Examination of Hardened Concrete". Samples were taken from horizons of the concrete cores exhibiting visual signs of alkali-silica reactivity. Disk samples were cut from the concrete cores, parallel to the orientation of the microfissuring and thin sections were prepared for examination under the polarizing light microscope. Preparation of these thin sections required vacuum impregnation of epoxy in order to preserve microfractures and prevent loss of the softer cement paste.

The concrete was examined for mineralogy, texture, fabric, bond of coarse aggregate with matrix, and presence of alkali-silica reaction.

SAMPLE RESULTS

Sample C-2

Sample C-2 was taken from Boring C-2 at the depth interval of 1.5 ft. The following was noted:

Coarse Aggregate:

Composition:	Material	Percentage
	Dolomite:	45%
	Micritic limestone:	55%

Grain size: 3 – 25 mm Crushed Stone
Nominal: 20 mm
Shape: Angular to Sub-angular
Observations:

- Secondary dolomite visible in micritic limestone matrix (dolomitization). Could also consider this aggregate as an impure dolomitic limestone. Variable fossil content in micritic limestone.
- Presence of anhedral grains of quartz, opaques and clay in aggregate
- Microfissures, some with silica gel visible around grain boundaries and through micritic limestone coarse aggregate and paste.
  Carbonated reaction rims visible around dolomitic limestone

Fine Aggregate:

Composition: Material
Granitic with metamorphic component

Grain size: 0.01 – 3 mm
Shape: angular to sub-rounded

Composition:		
	Lithic fragments	5-7%
	Quartz	15-20%
	K-feldspar	45-50%
	Microcline	5-10%
	Plagioclase	3-5%
	Sericite	1-2%
	Mica/biotite	1%
	Chlorite	<1%
	Amphibole/pyroxene	2-3%
	Accessory minerals	1%

Observations: Optically strained quartz present as single grains and in lithic fragments.
Cement Paste:

- Irregularly shaped voids from entrapped air/water up to 2 mm in size
- Normal distribution of paste, good bond with aggregate
- No apparent entrained air

Summary

The concrete is comprised of coarse aggregate composed of dolomite and micritic limestone. Some of the micritic limestone has undergone dolimitization and exhibits microtextures typical of an aggregate susceptible to alkali-silica reaction. These may be considered as impure dolomitic limestones. Microfissures, some filled with silica gel were observed around the micritic limestone coarse aggregate grain boundaries and through the cement paste.

Sample C-5

Sample C-5 was taken from Boring C-5 at the depth interval of 1.0 ft.. This sample exhibits very similar petrographic features as sample C-2. The coarse and fine aggregate aggregate facies appear to be the same as C-2 with the sources of the aggregate likely originating from the same quarries. The following was noted:

Coarse Aggregate:

Composition:	Material	Percentage
	Dolomite:	25%
	Micritic limestone:	75%

Grain size: 1 – 20 mm Crushed Stone
Nominal: 20 mm
Shape: Angular to Sub-angular
Observations:

- Secondary dolomite visible in some grains of the micritic limestone matrix (dolomitization). Could also consider this aggregate as an impure dolomitic limestone. Variable fossil content in micritic limestone.
- Presence of anhedral grains of quartz, opaques and clay in aggregate
- Microfissures, some with silica gel visible around grain boundaries and through micritic limestone coarse aggregate and paste.
- Carbonated reaction rims visible around micritic limestone.

Fine Aggregate:

Composition: Material
Granitic with metamorphic component

Grain size: 0.01 – 12 mm
Shape: angular to sub-rounded

Composition:			
	Lithic fragments	5-10%	
	Quartz		15-20%
	K-feldspar		50-55%
	Microcline		5-7%
	Plagioclase		2-3%
	Sericite	1-2%	
	Mica/biotite		1%
	Chlorite	<1%	
	Amphibole/pyroxene		2-3%
	Accessory minerals		1%

Observations: Optically strained quartz present as single grains and in lithic fragments.
Cement Paste:

- Irregularly shaped voids from entrapped air/water up to 1 mm in size
- Normal distribution of paste, good bond with aggregate
- No apparent entrained air

Summary:

The concrete is comprised of coarse aggregate composed of dolomite and micritic limestone. Some of the micritic limestone has undergone dolimitization and exhibits microtextures typical of an aggregate susceptible to alkali-silica reaction. These may be considered as impure dolomitic limestones. Microfissures, some filled with silica gel were observed around the micritic limestone coarse aggregate grain boundaries and through the cement paste.

We trust this report provides the information require at this time. Please do not hesitate to contact our office should any additional questions arise, or should you require further inspection and testing services.

Yours very truly,

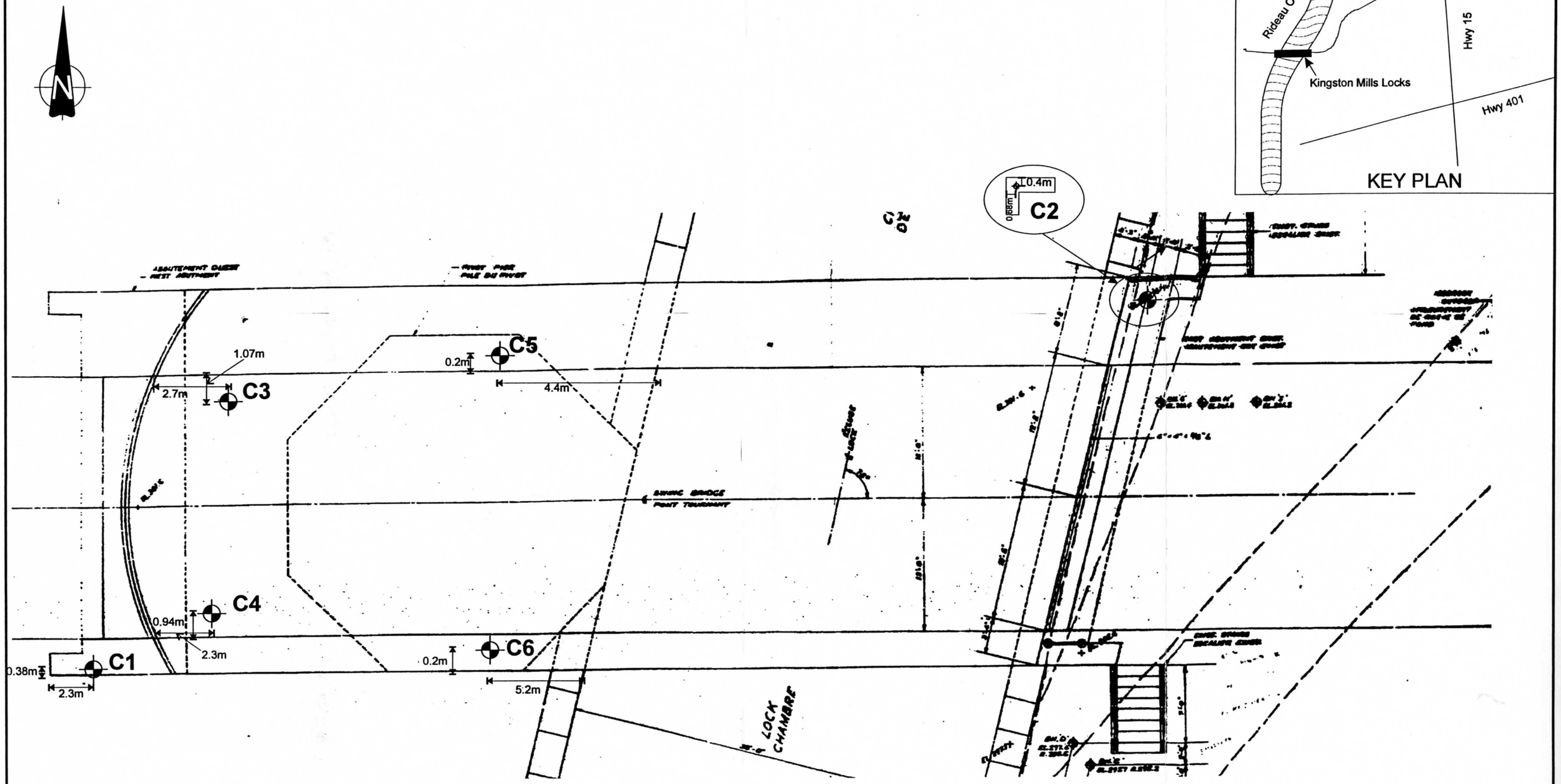
INSPEC-SOL INC.

Prepared by: Myles Carter

Reviewed by: Joseph B. Bennett, P. Eng.,
Manager-Kingston

By fax/mail (514) 694-5835

MC/lp



QUANTOCON ASSOCIATES
BRIDGE REHABILITATION, KINGSTON MILLS LOCKS
RIDEAU CANAL, KINGSTON, ONTARIO

CORING LOCATION PLAN

DRWN:	SCALE:	REF. No:
S.L..	NOT TO SCALE	20226-K6160E
CHKD:	DATE:	PLAN No
J.B.B.	DECEMBER 1999	K6160-1

INSPEC-SOL

K6160A

F0-010.11/08-95

REFERENCE No.: 20226-K-6160B ENCLOSURE No.: 1

INSPEC-SOL

BOREHOLE No.: C-1
ELEVATION: 302.4 FT

BOREHOLE REPORT
Page 1 of 1

CLIENT: QUANTACON ASSOCIATES
PROJECT: BRIDGE REHABILITATION
LOCATION: KINGSTON MILLS LOCKS, RIDEAU CANAL, KINGSTON, ONTARIO
DESCRIBED BY: M. CARTER CHECKED BY: M. CARTER
DATE (START): November 29, 1999 DATE (FINISH): November 29, 1999

LEGEND
- SS SPLIT SPOON
- ST SHELBY TUBE
- RC ROCK CORE
- WATER LEVEL

STRATIGRAPHY					SAMPLE				TEST RESULTS		
DEPTH Feet	DEPTH Metres	ELEVATION	STRATIGRAPHY	DESCRIPTION OF SOILS AND BEDROCK	STATE	TYPE AND NUMBER	RECOVERY	OTHER TESTS	BLOWS 6 in/15 cm or RQD	PENETRATION INDEX	COMMENTS
		302.4		GROUND SURFACE			%			N	
0	0			CONCRETE WITH AIR ENTRAINMENT, NORMALLY CONSOLIDATED, ACCEPTABLE QUALITY. LIMESTONE COARSE AGGREGATE: ¾" NOMINAL, ANGULAR. FINE AGGREGATE: 0-¼" GRANITIC, SUB ANGULAR TO SUB ROUNDED. MICRO FISSURES VISIBLE CUTTING through LIMESTONE AGGREGATE AND PASTE, MICRO FISSURES OPEN TO DEPTH OF 2'4". REACTION RIMS VISIBLE AROUND SOME LIMESTONE AGGREGATE.		RC-1	100		82%		COARSE AGGREGATE MICRITIC LIMESTONE: 90% CRISTALLINE LIMESTONE: 9 % SYENITE 1%. SYENITE COBBLES UP TO 7" IN SIZE. AIR VOIDS UP TO 3/8" IN SIZE. REINFORCING STEEL: ¾" Ø ROUND DEFORMED BAR AT 1'8" AND 4'7", NO CORROSION. COMPRESSIVE STRENGTH SPECIMEN
	1,0										
5				GOOD QUALITY CONCRETE, NORMALLY CONSOLIDATED. LIMESTONE COARSE AGGREGATE WITH LOCAL GRANITE COBBLES NO BOND BETWEEN GRANITE AND CEMENT PASTE. MICRO FISSURES IN PASTE AND AGGREGATE, BUT CLOSED.		RC-2	100		97%		5-8': CONCRETE COARSE AGGREGATE: MICRITIC LIMESTONE: 90% CRYSTALLINE LIMESTONE: 9 % SYENITE 1%. FINE AGGREGATE: 0-3/8" GRANITIC, SUB ANGULAR TO SUB ROUNDED. AIR VOIDS UP TO 3/8" IN SIZE.
	2,0										
8.0				SYENITE BEDROCK. MASSIVE, MEDIUM GRAINED WITH LINEATION OF MAFIC MINERALS. OXIDIZED JOINTS AT 8'-5", 9'-4", 10'-4", VERTICAL JOINT FROM 8'-9".		RC-3	100		62%		
10	3,0			END OF BOREHOLE NOTE: SAMPLE TAKEN FOR COMPRESSIVE STRENGTH TEST BETWEEN 2'8" AND 3'4".							
	4,0										
15											

REFERENCE No.: 20226-K-6160B

ENCLOSURE No.: 2

INSPEC-SOL

BOREHOLE No.: C-2

ELEVATION: 302.4 FT

BOREHOLE REPORT

CLIENT: QUANTACON ASSOCIATES

PROJECT: BRIDGE REHABILITATION

LOCATION: KINGSTON MILLS LOCKS, RIDEAU CANAL, KINGSTON, ONTARIO

DESCRIBED BY: M. CARTER CHECKED BY: M. CARTER

DATE (START): November 30, 1999 DATE (FINISH): December 1, 1999

LEGEND

- SS SPLIT SPOON
- ST SHELBY TUBE
- RC ROCK CORE
- WATER LEVEL

STRATIGRAPHY					SAMPLE				TEST RESULTS		
DEPTH		ELEVATION	STRATIGRAPHY	DESCRIPTION OF SOILS AND BEDROCK	STATE	TYPE AND NUMBER	RECOVERY	OTHER TESTS	BLOWS 6 in/15 cm or RQD	PENETRATION INDEX	COMMENTS
Feet	Metres	302.4		GROUND SURFACE			%			N	
0	0			CONCRETE WITH AIR ENTRAINMENT, NORMALLY CONSOLIDATED. UPPER 12" CUT BY NUMEROUS OPEN MICRO FISSURES, 6 MICRO FISSURES IN THE UPPER 2". OPEN MICRO FISSURES IN PASTE AND THROUGH LIMESTONE AGGREGATE TO DEPTH OF 6'. REACTION RIMS VISIBLE ON EXPOSED CONCRETE SURFACE TO DEPTH OF 6'. SOME SILICA GEL DEPOSITED IN OPEN PORES IN UPPER 1'-6".		RC-1	100		76%		COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 75% CRYSTALLINE LIMESTONE: 25 %. SYENITE BLOCKS 4" TO 12". NO BOND WITH CEMENT PASTE. COMPRESSIVE STRENGTH SPECIMEN FINE AGGREGATE: 0- 3/8", SUB ANGULAR TO SUB ROUNDED, GRANITIC.
	1.0										
5											
6.0											
	2.0			CONCRETE WITH AIR ENTRAINMENT, NORMALLY CONSOLIDATED TO 10'. VERY POROUS AND HONEYCOMBING FROM 10' TO 12'-4". CLOSED MICRO FISSURES THROUGH AGGREGATE AND IN PASTE UP TO DEPTH OF 9'.		RC-2	100		49%		COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 60% CRISTALLINE LIMESTONE: 40 % FINE AGGREGATE: 0-3/8" GRANITIC SUB ANGULAR TO SUB ROUNDED SYENITE BLOCKS 2" TO 7", POOR BOND TO PASTE AIR VOIDS UP TO ½" IN SIZE.
10	3.0										
12.3				END OF BOREHOLE NOTE: SAMPLE TAKEN FOR COMPRESSIVE STRENGTH TEST BETWEEN 1'2" AND 1'10".							
	4.0										
15											

REFERENCE No.: 20226-K-6160B

ENCLOSURE No.: 3

INSPEC-SOL

BOREHOLE No.: C-3

ELEVATION: 301.47FT

BOREHOLE REPORT

Page 1 of 1

CLIENT: QUANTACON ASSOCIATES

PROJECT: BRIDGE REHABILITATION

LOCATION: KINGSTON MILLS LOCKS, RIDEAU CANAL, KINGSTON, ONTARIO

DESCRIBED BY: M. CARTER CHECKED BY: M. CARTER

DATE (START): November 29, 1999 DATE (FINISH): November 29, 1999

LEGEND

- SS SPLIT SPOON
- ST SHELBY TUBE
- RC ROCK CORE
- WATER LEVEL

STRATIGRAPHY					SAMPLE				TEST RESULTS		
DEPTH		ELEVATION	STRATIGRAPHY	DESCRIPTION OF SOILS AND BEDROCK	STATE	TYPE AND NUMBER	RECOVERY	OTHER TESTS	BLOWS 6 in/15 cm or RQD	PENETRATION INDEX	COMMENTS
Feet	Metres	301.47		GROUND SURFACE			%			N	
0	0			2" ASPHALT OVER CONCRETE WITH AIR ENTRAINMENT, NORMALLY CONSOLIDATED.		RC-1	100		100%		COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 55% CRYSTALLINE LIMESTONE: 5 % ARGILLACEOUS LIMESTONE: 40 %
1.0				NO MICRO FISSURES OR REACTION RIMS VISIBLE.							FINE AGGREGATE: 0- ¾", SUB ANGULAR TO SUB ROUNDED, GRANITIC.
				DELAMINATION AT 4" FROM SURFACE.							
				END OF BOREHOLE							
	1,0										
5											
	2,0										
10											
	3,0										
	4,0										
15											

REFERENCE No.: 20226-K-6160B

ENCLOSURE No.: 4

INSPEC-SOL

BOREHOLE No.: C-4
ELEVATION: 301.47 FT

BOREHOLE REPORT

CLIENT: QUANTACON ASSOCIATES

PROJECT: BRIDGE REHABILITATION

LOCATION: KINGSTON MILLS LOCKS, RIDEAU CANAL, KINGSTON, ONTARIO

DESCRIBED BY: M. CARTER CHECKED BY: M. CARTER

DATE (START): November 29, 1999 DATE (FINISH): November 29, 1999

LEGEND

- ☒ SS SPLIT SPOON
- ▨ ST SHELBY TUBE
- ◫ RC ROCK CORE
- ▼ WATER LEVEL

STRATIGRAPHY					SAMPLE				TEST RESULTS		
DEPTH Feet	DEPTH Metres	ELEVATION	STRATIGRAPHY	DESCRIPTION OF SOILS AND BEDROCK	STATE	TYPE AND NUMBER	RECOVERY	OTHER TESTS	BLOWS 6 in/15 cm or RQD	PENETRATION INDEX	COMMENTS
		301.47		GROUND SURFACE			%			N	
0	0			ASPHALT 1½", THICK, NO MEMBRANE PRESENT.		RC-1	100		100%		COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 25% CRYSTALLINE LIMESTONE: 10 % ARGILLACEOUS LIMESTONE: 65 %
1.3				CONCRETE WITH AIR ENTRAINMENT, NORMALLY CONSOLIDATED.							
				NO MICRO FISSURES OR REACTION RIMS VISIBLE.							
				DELAMINATION AT 10" FROM SURFACE.							
	1,0			5/8" ROUND, DEFORMED REBAR AT 4" Ø FROM SURFACE. GOOD BOND WITH PASTE, NO CORROSION.							
				END OF BOREHOLE							
5				NOTE:							
				SAMPLE SUBMITTED FOR COMPRESSIVE STRENGTH TESTING.							
	2,0										
	3,0										
10											
	4,0										
15											

REFERENCE No.: 20226-K-6160B ENCLOSURE No.: 5

INSPEC-SOL

BOREHOLE No.: C-5
ELEVATION: 296.62 FT

BOREHOLE REPORT
Page 1 of 1

CLIENT: QUANTACON ASSOCIATES
PROJECT: BRIDGE REHABILITATION
LOCATION: KINGSTON MILLS LOCKS, RIDEAU CANAL, KINGSTON, ONTARIO
DESCRIBED BY: M. CARTER CHECKED BY: M. CARTER
DATE (START): December 1, 1999 DATE (FINISH): December 1, 1999

LEGEND
- SS SPLIT SPOON
- ST SHELBY TUBE
- RC ROCK CORE
- WATER LEVEL

STRATIGRAPHY					SAMPLE				TEST RESULTS		
DEPTH		ELEVATION	STRATIGRAPHY	DESCRIPTION OF SOILS AND BEDROCK	STATE	TYPE AND NUMBER	RECOVERY	OTHER TESTS	BLOWS 6 in/15 cm or RQD	PENETRATION INDEX	COMMENTS
Feet	Metres	296.62		GROUND SURFACE			%			N	
0	0			TOP 2 TO 3 INCHESTOTALLY DIS-INTEGRATED THEN CONCRETE WITH AIR ENTRAINMENT, NORMALLY CONSOLIDATED. OPEN MICRO FISSURES IN PASTE AND COARSE AGGREGATE TO 12", CLOSED MICRO FISSURES TO 4'-4". DELAMINATED AT 1" AND 4'-4". VERY POROUS WITH HONEYCOMBING FROM 4'-4" TO 5'-6". VOIDS UP TO 2" LONG AND ½" DEEP.		RC-1	100	AIR VOID SAMPLE COMP.STR. SAMPLE TENSILE STR. SAMPLE	82%		COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 20 % CRYSTALLINE LIMESTONE: 75 % ARGILLACEOUS LIMESTONE: 5 % SYENITE COBBLES (6" IN SIZE) AT 1'6".. FINE AGGREGATE: 0-¾", SUB ANGULAR TO SUB ROUNDED, GRANITIC. REINFORCING STEEL AT 4'-4": ¾"Ø ROUND DEFORMED BAR, NO BOND WITH PASTE, LIGHTLY CORRODED.
	1,0										
5											
5.5				POSSIBLE FILL: BROWN CLAYEY SILT TILL, WITH TRACES OF GRAVEL, MOIST		SS-1	100				
	2,0										
7.5				END OF BOREHOLE							
				NOTES: SAMPLE TAKEN FOR COMPRESSIVE STRENGTH TEST BETWEEN 2'5" AND 3'1". SAMPLE TAKEN FOR AIR-VOID ANALYSIS AT 2'2". SAMPLE TAKEN FOR TENSILE STRENGTH TESTING BETWEEN 3'1" AND 3'-10".						
10	3,0										
	4,0										
15											

REFERENCE No.: 20226-K-6160B ENCLOSURE No.: 6

INSPEC-SOL

BOREHOLE No.: C-6
ELEVATION: 296.7 FT

BOREHOLE REPORT
Page 1 of 1

CLIENT: QUANTACON ASSOCIATES
PROJECT: BRIDGE REHABILITATION
LOCATION: KINGSTON MILLS LOCKS, RIDEAU CANAL, KINGSTON, ONTARIO
DESCRIBED BY: M. CARTER CHECKED BY: M. CARTER
DATE (START): November 29, 1999 DATE (FINISH): November 30, 1999

LEGEND
- SS SPLIT SPOON
- ST SHELBY TUBE
- RC ROCK CORE
- WATER LEVEL

STRATIGRAPHY					SAMPLE				TEST RESULTS		
DEPTH Feet	DEPTH Metres	ELEVATION	STRATIGRAPHY	DESCRIPTION OF SOILS AND BEDROCK	STATE	TYPE AND NUMBER	RECOVERY	OTHER TESTS	BLOWS 6 in/15 cm or RQD	PENETRATION INDEX	COMMENTS
		296.7		GROUND SURFACE			%			N	
0	0			TOP 2 TO 3 INCHES DIS-INTEGRATD THEN CONCRETE WITH AIR ENTRAINMENT, NORMALLY CONSOLIDATED. OPEN MICRO FISSURES IN PASTE AND COARSE AGGREGATE TO DEPTH OF 10", THEN TIGHT MICRO FISSURES TO DEPTH OF 2'-6". DELAMINATED AT 4" AND 1'-7" FROM SURFACE. SOME REACTION RIMS AROUND COARSE AGGREGATE ON DELAMINATED SURFACE.				COMP.STR. SAMPLE			COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 20 % CRYSTALLINE LIMESTONE: 5 % ARGILLACEOUS LIMESTONE: 75 %. FINE AGGREGATE: 0-¾", SUB ANGULAR TO SUB ROUNDED, GRANITIC. REINFORCING STEEL AT 3'-5", ¾" Ø ROUND DEFORMED BAR, NO CORROSION, NO BOND WITH CEMENT PASTE.
	1,0								93%		
						RC-1	100		AIR VOID SAMPLE		
									TENSILE STR. SAMPLE		
4,8 5				POSSIBLE FILL: BROWN CLAYEY SILT TILL, WITH TRACES OF GRAVEL, HUMID		SS-1	100				
6.3	2,0			END OF BOREHOLE							
				NOTES: SAMPLE TAKEN FOR COMPRESSIVE STRENGTH TEST BETWEEN 2" AND 14". SAMPLE TAKEN FOR AIR-VOID ANALYSIS at 2'. SAMPLE TAKEN FOR TENSILE STRENGTH TESTING BETWEEN 3'-10" to 4'-7".						
10	3,0										
	4,0										
15											

Rideau Canal- KINGSTON MILLS LOCKS.

CONCRETE CONDITION INVESTIGATION

KINGSTON, ONTARIO

Photo No. 1-

CORE NUMBER C-1

Photo No. 2

CORE NUMBER C-2

Rideau Canal- KINGSTON MILLS LOCKS.

CONCRETE CONDITION INVESTIGATION

KINGSTON, ONTARIO

Photo No. 3

CONCRETE CORE C3 AND C4

Rideau Canal- KINGSTON MILLS LOCKS.

CONCRETE CONDITION INVESTIGATION

KINGSTON, ONTARIO

Photo No. 4

CORE NUMBER C-5

Photo No. 5

CORE NUMBER C-6

SECTION 7

APPENDIX "B"

J.D. LEE ENGINEERING LIMITED

REPORT JANUARY 1977

J. D. Lee Engineering Limited

1155 Division Street

Kingston, Ontario

KM# 7

SWING BRIDGE

AT

KINGSTON MILLS

KM#7

JOB #1608

JANUARY 1977

RECEIVED

JAN 19 19[illegible]

Ans'd.....

SITE INVESTIGATION SERVICES
LIMITED

677 CROWN DRIVE PETERBOROUGH, ONT. PHONE 743-6850

January 17, 1977

J. D. Lee Engineering Limited
1155 Division Street
Kingston, Ontario

Attention: Mr. Don McNeely, P. Eng.

Re: Kingston Mills Swing Bridge

Dear Sir,

We have completed an evaluation of soil and rock conditions behind the east abutment of the existing swing bridge. This letter describes the conditions encountered. Icy conditions prevented access to the west abutment area on December 23, 1976, when the east abutment holes were completed. Additional borings will be completed at the west abutment when we have a suitable track-mounted drill in the Kingston area.

EAST ABUTMENT CONDITIONS

Ten borings were attempted behind the east abutment of the swing bridge to assess soil and bedrock conditions. The locations of the borings are shown on Figure 1, and soil profiles are summarized on Table 1 and on Figures 2, 3 and 4.

In general, it appears that much of the east wall of the lock channel (on which the bridge abutment sits) is in rock cut with little or no gap existing between the wall and the cut face.

South of the swing bridge, the presumed bedrock level, as indicated by

refusal to augering, is within 3 feet of the top of the lock wall. The rock within 15 feet of the wall, is covered with topsoil and stony clayey silt fill up to 3 feet deep. Bedrock is exposed 15 to 20 feet from the wall line, at elevations above the top of the wall.

North of the swing bridge, at holes E and J, the bedrock surface is near elevation 288.5 feet. This is about 9.5 feet below the top of the concrete wall. The rock surface slopes up to a bedrock exposure, about 40 feet east of the wall. The rock between the outcrop and the concrete wall is covered with topsoil, fill and some native stony sandy clayey silt till. Fill is predominant and most of the fill consists of dark brown stony sandy clayey silt mixed with cobbles or rockfill.

At holes G, H and I, numerous attempts were made to penetrate a rockfill zone about 5 feet below road grade. Attempts were unsuccessful in spite of the fact that a very powerful CME 75 drill (9000 foot-pounds torque) was used for the augering. Attempts were discontinued when the augers started to break. Cone probes were driven below the auger refusal depth. We suspect that the cone probes met refusal above the level of the bedrock. Consequently, the cone probes indicate a minimum depth to rock. A more probable depth can be estimated by extrapolating between bedrock levels north and south of the roadway.

Bedrock cored at hole J consisted of pinkish to reddish grey syenite. Above 10.6 feet depth (elevation 287 feet) the rock is partially weathered along vertical and horizontal joints. At greater depths, the rock is sound and massive. A detailed description of the core is shown on Figure 5.

GENERAL DISCUSSION

Available data indicates that the east lock wall abuts against the face of a rock cut. However, the design of structures at Kingston Mills is far from conventional. Accordingly, we suggest that possible variations, such as placed

stone zones in overbreak areas behind the wall, be allowed for in any remedial work attempted.

I trust that the above data is adequate for your requirements. However, should you have any queries, please do not hesitate to contact me.

Yours very truly,

R. Marttila, P. Eng.

SITE INVESTIGATION SERVICES LIMITED

RM/bd

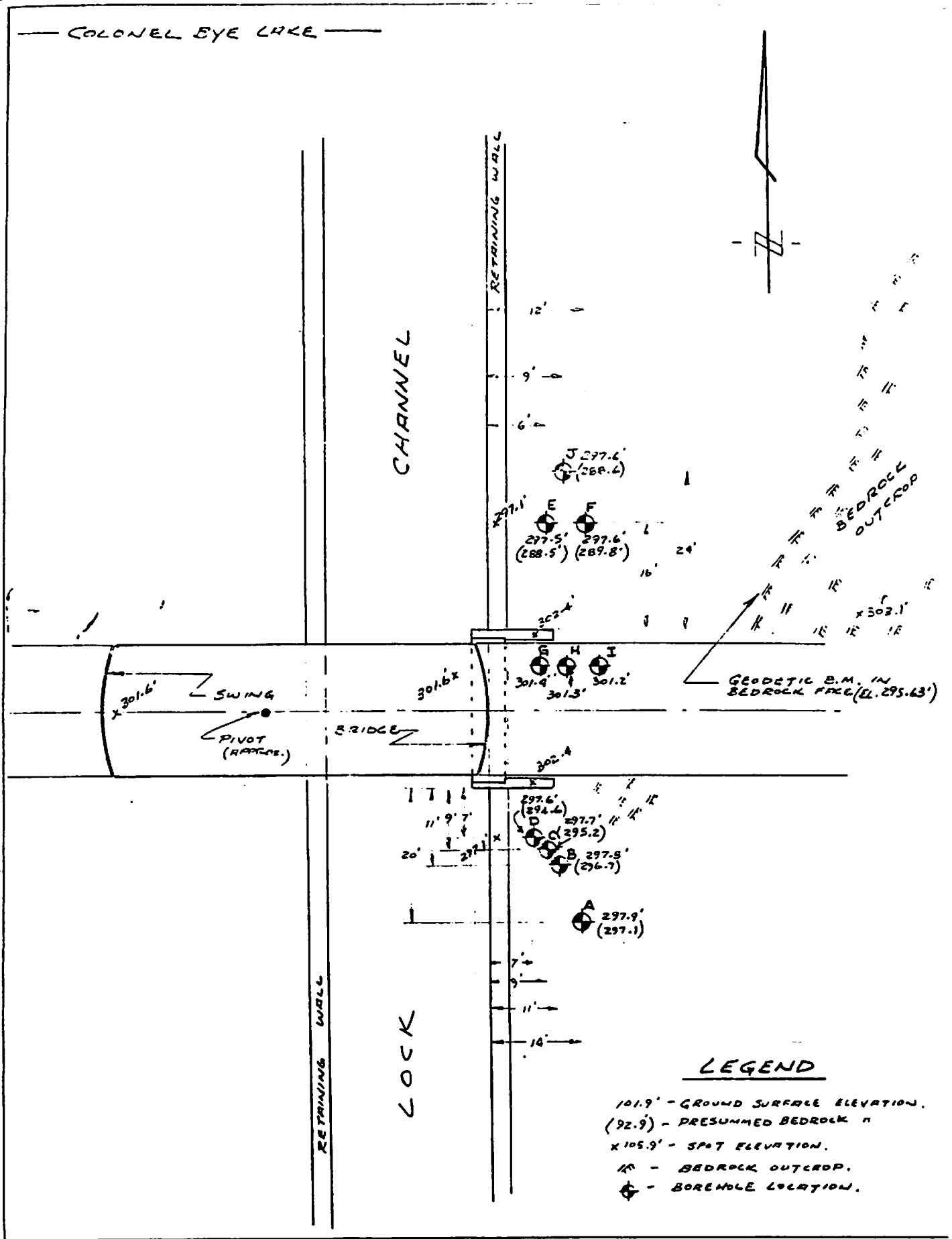
4 c.c.

TABLE 1

KINGSTON MILLS SWING BRIDGE

SUMMARY OF PROBE HOLES

HOLE NO.	ELEVATION	DEPTH	DESCRIPTION
A	101.5'	0 - 10"	Topsoil
		10"	Augering refusal (presumed bedrock)
B	101.1'	0 - 13"	Topsoil
		13"	Augering refusal (presumed bedrock)
C	99.6'	0 - 12"	Topsoil
		12" - 2.5'	Dark brown stony clayey silt
		2.5'	No further progress (presumed bedrock)
D	98.9'	0 - 12"	Topsoil
		12" - 3'	Dark brown to black pebbly clayey silt
		3'	Auger refusal (presumed bedrock)
E	92.8'	0 - 10"	Topsoil
		10" - 9'	Brown stony clayey sand silt (till) fill. Frequent cobbles and boulders below 6 feet. Loose and wet.
		9'	Auger refusal (presumed bedrock)
F	94.15'	0 - 9"	Topsoil
		9" - 6'	Dark brown stony clayey sandy silt (till) fill containing frequent cobbles and rockfill.
		6' - 7.8'	Dark brown stony sandy clayey silt till.
		7.8'	Auger refusal (presumed bedrock)
G	See Figure 2		
H	See Figure 3		
I	301.2'	0 - 4"	Asphalt
		4" - 15"	Brown gravelly sand
		15" - 29"	Brown sand mixed with cobbles.
		29" - 4.3'	Dark brownish grey cobbly sandy clayey silt fill
		4.3'	Auger refusal (rockfill - possibly bedrock
J	See Figure 4		



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KINGSTON MILLS SWING BRIDGE ABUTMENT STUDY — LOCATION PLAN —	SCALE: 1" = 20'	DATE: JAN. 1977
	DRAWN: DWN.	FIGURE: 1

BOREHOLE DATA and TEST SUMMARY

SITE INVESTIGATION SERVICES Ltd. | JOB No: 1608 | BOREHOLE No: G | FIGURE No: 2

Project - KINGSTON MILLS
Location - SWING BRIDGE ABUTMENT
Hole Location - SEE PLAN

Date - December 22, 1976
Elevation Datum -
Type of Drill - 3½" H.S.A. (D-8)

JOB No: 1608 **BOREHOLE No:** G

LEGEND
Gravel | Sand | Clay
(See Appendix "A" for Other Symbols)

PENETRATION RESISTANCE (Blows/Ft)
2" O.D. Split Spoon —o—o— 2" O.D. Cone —•—
10 20 30 40 50

SHEAR STRENGTH (Kips/Ft²)
Field Vane – X Unconfined Compression – □

SOIL DESCRIPTION	SOIL SYMBOL	ELEVATION IN FEET	DEPTH IN FEET	MOISTURE CONTENT and ATTERBERG LIMITS (%) (Plastic Limit / Moisture Content / Liquid Limit)	LAB. TESTS	SAMPLE TYPE AND NUMBER
		301.4				
ASPHALT (4") over GRAVELLY SAND						
SAND FILL - brown sand fill mixed with cobbles						
CLAYEY SANDY SILT - dark brownish grey stony sandy clayey silt fill			5			
ROCKFILL						
REFUSAL IN ROCKFILL →		293.7	10			
REFUSAL OF CONE PROBE →			15			
			20			

50 blows/9"

BOREHOLE DATA and TEST SUMMARY

SITE INVESTIGATION SERVICES Ltd. | JOB No: 1608 | BOREHOLE No: H | FIGURE No: 3

Project - KINGSTON MILLS
Location - SWING BRIDGE ABUTMENT
Hole Location - SEE PLAN

Date - December 23, 1976
Elevation Datum -
Type of Drill - 3¼" H.S.A. (D-8)

JOB No: 1608 BOREHOLE No: H

LEGEND
Gravel | Sand | Clay
(See Appendix "A" for Other Symbols)

SOIL DESCRIPTION	SOIL SYMBOL	ELEVATION IN FEET	DEPTH IN FEET
		301.3	
ASPHALT (4") over GRAVELLY SAND			
SAND - brown sand mixed with cobbles			
CLAYEY SILT FILL - dark grey sandy clayey silt fill mixed with rockfill			
ROCKFILL - red granite rockfill			5
AUGER REFUSAL		293.0	
CONE PROBE REFUSAL			10
			15
			20

MOISTURE CONTENT and ATTERBERG LIMITS (%)
Plastic Limit | Moisture Content | Liquid Limit

LAB. TESTS

SAMPLE TYPE AND NUMBER

PENETRATION RESISTANCE (Blows/Ft)
2" O.D. Split Spoon —o—o— 2" O.D. Cone —•—•—
10 20 30 40 50
50 blows/4"

SHEAR STRENGTH (Kips/Ft²)
Field Vane - X Unconfined Compression - □

BOREHOLE DATA and TEST SUMMARY

SITE INVESTIGATION SERVICES Ltd. | JOB No: 1608 | BOREHOLE No: J | FIGURE No: 4

Project · KINGSTON MILLS
Location · SWING BRIDGE ABUTMENT
Hole Location · SEE PLAN

Date · December 23, 1976
Elevation Datum · ---
Type of Drill · 3¼" H.S.A. (D-8)

JOB No: 1608 BOREHOLE No: J

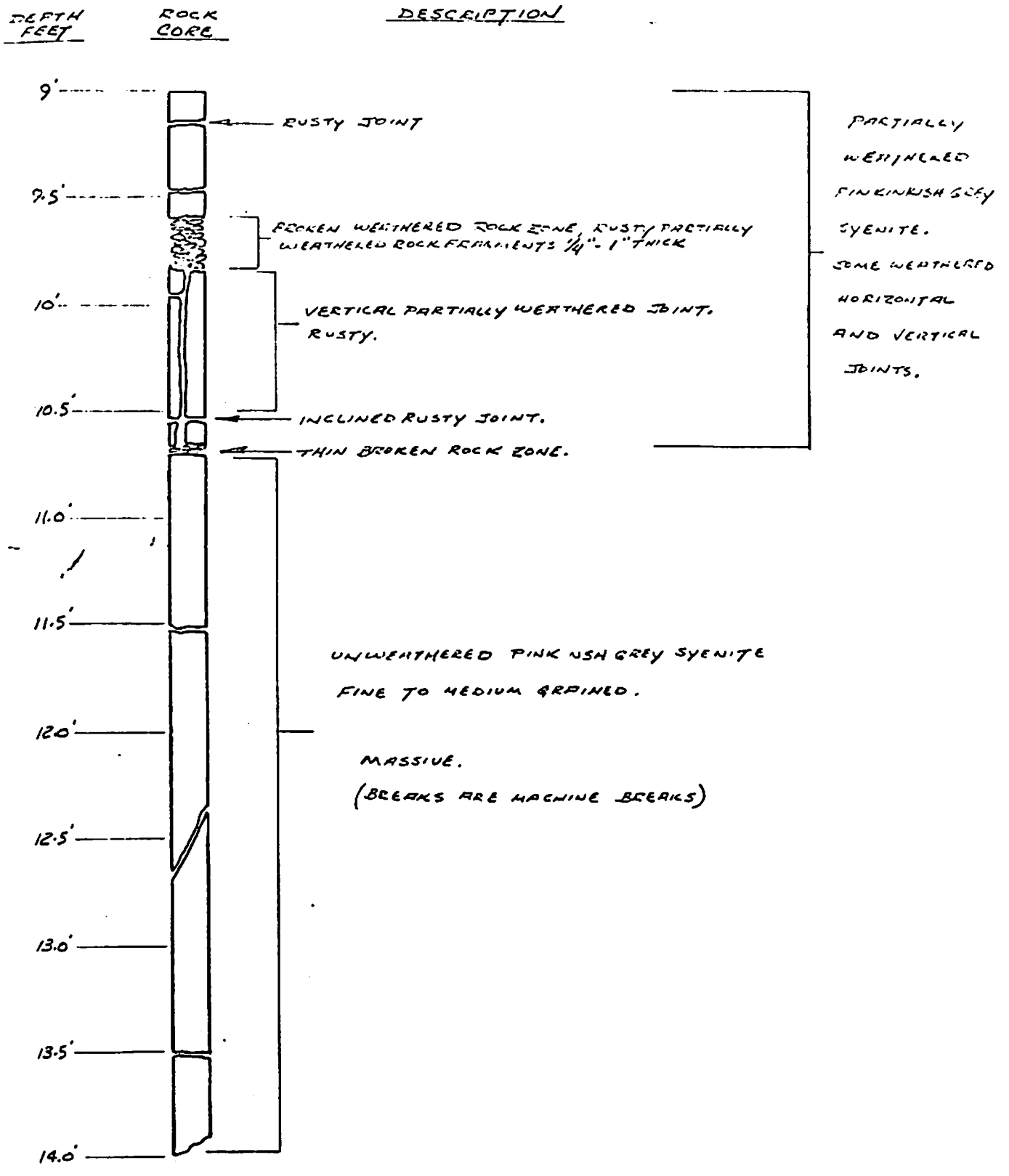
LEGEND
Gravel Sand Clay
(See Appendix "A" for Other Symbols)

PENETRATION RESISTANCE (Blows/Ft)
2" O.D. Split Spoon —o—o— 2" O.D. Conc —•—•—
10 20 30 40 50

SHEAR STRENGTH (Kips/Ft²)
Field Vane – X Unconfined Compression – □

SOIL DESCRIPTION	SOIL SYMBOL	ELEVATION IN FEET	DEPTH IN FEET	MOISTURE CONTENT and ATTERBERG LIMITS (%) Plastic Limit / Moisture Content / Liquid Limit	LAB. TESTS	SAMPLE TYPE AND NUMBER
		297.6				
TOPSOIL						
CLAYEY SILT FILL - dark brown cobbly sandy clayey silt fill			5			
CLAYEY SILT TILL - dark brownish grey stony sandy clayey silt till		288.6				
BEDROCK - pinkish grey fine to medium grained syenite - some weathered joints above 10.6 feet - sound and unweathered below 10.6 feet. Massive.			10			BX Core
			15			
END OF HOLE (Note: standpipe installed to 14' depth)			20			

98+% Recovery



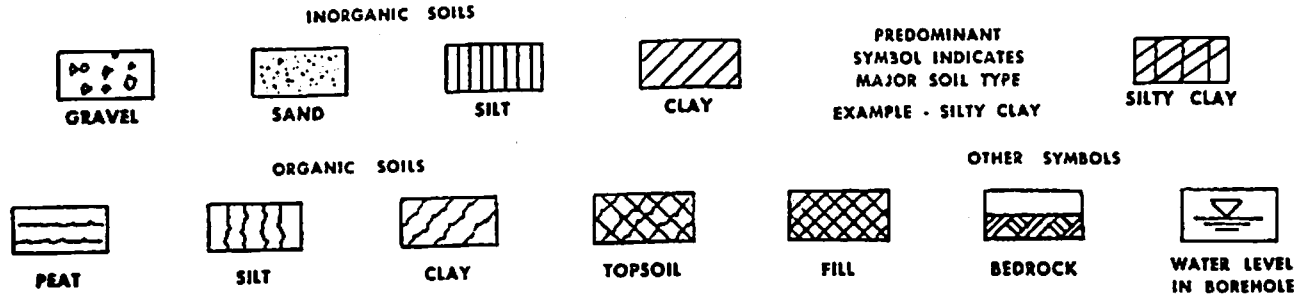
SITE INVESTIGATION SERVICES LIMITED

KINGSTON MILLS SWING BRIDGE EAST ABUTMENT AREA. LOG OF ROCK CORE FROM BOREHOLE 'J'	SCALE:	DATE: JAN. 1977
	DRAWN: DWN	FIGURE: 5

EXPLANATION OF SYMBOLS AND TEST DATA

SOIL DESCRIPTION

A description of visible characteristics of the soil as determined in the field and altered, if necessary, on the basis of laboratory classification tests.



SAMPLES

Condition:


RELATIVELY UNDISTURBED


DISTURBED


NOT RECOVERED

Type: D.S. - 1⅜" ID Drive Sample
A.S. - Auger Sample
U - Thin-walled Tube Sample
UP - Piston Sample

PENETRATION RESISTANCE:

(N) Indicates number of blows, of a 140-lb. hammer falling 30 inches, required to drive a 2" OD Drive Sampler a distance of 1 foot into the soil. This resistance is used to assess the relative density of cohesionless soils and the relative consistency of cohesive soils.

OTHER TESTS

M - Grain size analysis using seives or hydrometer or both - plotted graphically on a separate sheet.

V_1 - laboratory vane tests.

γ_d - dry unit weight.

C - consolidation test - results on separate sheet.

T - triaxial compression test - results on a separate sheet.

P - proctor compaction test.

K - laboratory permeability test.

SOILS PROFILES:

Where soil profiles are shown on drawings the soil profile applies only to the borehole location and may be different at intermediate locations on the site.

GROUND WATER:

Ground Water levels are generally measured in the open boreholes and apply to conditions at the time of [illegible] ground water fluctuations should be expected at most sites.

SECTION 8

APPENDIX "C"

TERMS OF REFERENCE

Terms of Reference
for
Surveys & Geotechnical Investigations
Kingston Mills Swing Bridge
Kingston Mills, Rideau Canal
Ontario

November 09, 1999 Revised Project # 51173-30001725

1.0 Location

The area for the investigation is located at the swing bridge, Kingston Mills Lock on the Rideau Canal. Access is from highway #15, just north of the #401.

2.0 Background

The bridge abutments and pivot pier foundation are constructed of reinforced concrete and have deteriorated severely. The bridge abutments are badly cracked and efflorescence is quite obvious, caused by the use of highway de-icing salts on the bridge deck during winter. Repairs to the concrete abutments and pivot pier will be carried out next year.

The field survey and geotechnical investigations should provide the necessary data, information and recommendations needed to carry out the design for repair to the abutments and pivot pier.

3.0 Scope of Work

Arrange a site meeting with PWGSC and Rideau Canal engineering to discuss the specific requirements of the work.

Generally, the Consultant is to conduct geotechnical and site investigations, field surveys and lab tests as well as all other work that may be required in order to produce the final geotechnical report and provide geotechnical information for design.

Field Surveys

.1 Provide a detailed structural and topographic field survey of the bridge abutments and pivot pier. Take measurements of the abutments; all rail; rail components; anchors in the pivot pier.

.2 It is not required to survey and measure the actual bridge and bridge components.

.3 Take spot elevations on the canal walls; bridge abutments and grade in the vicinity of the bridge (both sides of the canal).

.4 Provide a site plan indicating all spot elevations taken; all measurements of the bridge abutments; track and track components. Provide a drawing, plan and sections, with all measurements taken.

.5 All drawings for the bridge and field survey are to be done on AutoCad version 12 or better. Provide four hard paper copies of the drawings and also a 3 1/2" diskette with the drawing to PWC at completion.

Geotechnical Investigations

.1 Conduct vertical coring investigations and lab testing as follows:

Drill 4 vertical holes through the abutments and pivot pier foundation to a depth of *1 metre* into bedrock. Use 10cm (4") diameter cores through concrete. Drill one vertical hole through the bridge deck into the concrete bridge counterweight.

- 2 vertical holes will be placed in the bridge abutments, one on each end of the bridge. Place each hole 15cm (6") in from the outside face of the abutment.
- 2 vertical holes will be placed in the concrete pivot pier foundation adjacent to the circular steel track beneath the bridge. On the north side of the bridge a small section of steel grating will have to be removed for drilling access. Do not cause any damage to the grating in its removal or in its reinstallation after drilling.
- one vertical hole will be placed into the concrete counterweight at the west end of the swing bridge. This will be placed through the asphalt roadway.

.2 Provide a site plan of the work area and indicate the location of each borehole on the plan. (The exact location of each borehole will be determined on site by the consultant). The number, depth and location of the boreholes may be altered by Public Works, depending upon the initial coring results.

.3 For boreholes drilled into concrete and bedrock (including the counterweight):

.1 Determine the quality of the bedrock/concrete interface;
.2 Determine the RQD factor and record it in the coring logs;
.3 Determine air entrainment in the concrete;
.4 Determine the compressive strength of the concrete;
.5 Using concrete samples taken from each corehole determine the effect and presence of any alkali-aggregrate reactivity. Comment on the severity of the reaction in regards to accelerating the deterioration of the abutments;
.6 Determine the quality of the concrete of the abutments and counterweight;
.7 Log the presence of all voids, discontinuities, seams and joints encountered during coring in the concrete abutment and pivot pier.

.4 Establish geodetic elevations for all boreholes by field levelling. Reference the borehole elevations to a fixed reference point on site. Tie each borehole location to the fixed reference point with field measured dimensions and indicate these measurements on the site plan.

.5 Consultant should keep an accurate account of the actual quantities drilled during the field work to ensure that the lengths provided in the original estimate are not exceeded. If additional drilling is required in order to obtain the information needed for the report, then the consultant must call PWC prior to proceeding with the additional work. Do not exceed the estimated coring quantitites without PWC approval to proceed.

4.0 General Requirements

.1 All data and work must be in the metric system.

.2 Notify the Rideau Canal office in Smiths Falls three days in advance of starting geotechnical work on site.

.3 All drilling is to be supervised by an experienced Soils Technician, under the *general supervision* of a Geotechnical Engineer registered with the PEO.

.4 Restore all damage caused during this contract to match the original conditions, including re-sodding of grassed areas. All tire ruts in grass are to be re-sodded.

.5 Boreholes in masonry and concrete must be filled with a non-shrink cement grout. Holes in asphalt must be compacted and levelled with asphalt.

.6 Holes in soil and overburden must be backfilled, compacted and levelled to match the surrounding grade.

.7 Remove all debris from the site on a daily basis.

.8 Allow no oil, gasoline, grout, debris or other contaminant to enter the water. No burying or burning on site is allowed.

.9 *When drilling through the bridge deck into the concrete counterweight do not block traffic crossing the bridge. Use barriers, warning signs and a traffic control person (flagman) to direct on-coming vehicular traffic. Place the hole to one side of the bridge and allow traffic to pass on the other.*

5.0 Schedule

.1 Start work immediately upon notice of award of contract.

.2 All work should be completed and final copies of the report sent to Public Works no later than 6 weeks after site work was started.

.3 All work must be fully completed and invoiced by December 31, 1999.

.4 Make available to Public Works the drilling logs and results of both field and laboratory analysis upon request. Consultant may be asked to fax copies of the field results to PWGSC as they become available, as engineering design may be proceeding concurrent with the investigations.

6.0 Report

Provide four copies of a final report containing the following:

.1 a site plan indicating the exact location of each borehole and all dimensions and elevations recorded in the field. Indicate the location and elevation of the fixed site reference point.

The site plan must be prepared with AutoCAD version 12 or newer. A copy of the plan on a 3 1/2"diskette must be given to PWGSC upon completion of the project. The site plan must indicate all elevations and field measurements.

.2 borehole logs; results of lab testing

.3 compressive strength results of the samples taken and air entrainment content;

.4 comment on the severity of the alkali-aggregate reactivity within the concrete and its long term effect upon the deterioration of the abutments;

.5 discuss coring results, voids, seams and any discontinuities found during the drilling. Record locations where drill water is lost in cavities, voids, or where washouts occur during the drilling and comment on the possible cause of the loss;

.6 the report must be prepared by a Geotechnical Engineer registered in the province of Ontario with the PEO.;

.7 final copies of the report are to be sent to RPS CH/EC PWGSC in Cornwall. Forward copies to:

Public Works & Government Services Canada
Real Property Services
111 Water Street East, 3rd Floor
Cornwall, Ontario
K6H 6S3

Attention: James Richardson, P.Eng.

7.0 Quantities and Offer

.1 Be aware that coring may be in masonry, bedrock, reinforced concrete, overburden and backfill. Provide all casing as required for the investigation.

There will be a total of *five* vertical boreholes; estimated drilling in reinforced concrete is approximately *20 metres.* Note that the concrete is reinforced with steel rebar. Estimated drilling in bedrock is *4 metres.* *(Note: These quantities are approximations only, and the actual field drilling quantities may differ significantly).*

.2 The exact location of the boreholes will be left to the discretion of the consultant, but with approval from PWGSC. Public Works may alter the number, location and depth of the holes based upon initial coring results. Vertical holes in the abutments and pivot pier will each extend *one metre* into bedrock.

.3 The Consultant's offer to undertake the work should be presented in three parts:

.1 *Professional Fees*: (FIXED LUMP SUM PRICE) to include all professional fees; administration costs; engineering costs; report preparation; any CAD or drafting services; office costs; FAX and typing costs.

.2 *Disbursements*: (ESTIMATED COST) to include all expenses incurred by the consultant; actual drilling costs with no additional mark-up by the consultant; field and lab tests; travel expenses. Mileage will be paid at the rate of 38¢ per kilometre.

.3 *Supervision*: (ESTIMATED COST) to provide field supervision of the drilling contract, based upon an hourly rate for professional fees. Travel expenses while supervising the drilling will be paid under *Disbursements*.

.4 The drilling cost estimate should be based upon a unit price for drilling in concrete and a separate unit price for the supply and installation of casing.

.5 Consultant's offer is *not* to include GST and taxes.

.6 Note: that an amount of money equivalent to the cost of restoring any site damage may be withheld until the damage has been repaired to the Owner's satisfaction. This includes but is not limited to backfilling of cored holes and repair to sodded areas.

.7 (a) Professional Fees: FIXED LUMP SUM PRICE
(b) Disbursements : ESTIMATED COST
(c) Drilling Supervision: ESTIMATED COST

The supervision costs are to be invoiced based upon HOURLY RATES. Consultant is requested to submit his hourly-rate fee schedule with his offer. Provide receipts for all disbursements and expenses.

8.0 Contacts

.1 Rideau Canal Engineering:
- Joe Brown (613) 283-7199 ext.248

.2 PWGSC Engineering:
- Jim Richardson (613) 938-5957

.3 Senior Design Engineer, PWGSC:
- Eric Sunstrum (819) 997-6047

9.0 Information Available

.1 Geotechnical report from Site Investigation Services, January 1977, at the east abutment.

.2 Construction drawings of the swing bridge abutments and pivot pier foundation.

.3 Site plan of the area, indicating elevations and grades.

J. Richardson, P. Eng.
RPS CH/EC PWGSC
Ontario Region