PUBLIC SERVICES AND PROCUREMENT CANADA

KINGSTON MILLS LOCKS 46 TO 49 REHABILITATION

DRAFT GEOTECHINCAL REPORT

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1 INTRODUCTION

1.1 CONTEXT

WSP Canada Inc. (WSP) was retained by Public Services and Procurement Canada (PSPC) to complete an assessment of the Kingston Mills Lockstation, part of the Rideau Canal waterway system, and provide recommendations and design requirements to complete proposed rehabilitation work. As part of the assessment and design process, a review of the available geotechnical information was undertaken and supplementary geotechnical field investigations were completed. Results of a detailed review of all available historical information, a supplemental field investigation and design analysis are presented in the ensuing paragraphs

This report was prepared in accordance with professional services agreement No. EQ754-171828/A with PSPC as the intended recipient. A disclosure of this report to third parties can only be made by the intended recipient who will assume responsibility for such a disclosure. The information, data, and opinions expressed in this report reflect WSP's best judgement in light of the information available at the time of preparation of the report. Any use of the report by third parties reliance upon, or decisions made based upon information provided in this report, are the responsibility of such third parties and specifically WSP accepts no responsibility for damages, if any, suffered by any third parties as a result of decisions made or actions taken based on information contained in this report. This limitations statement is considered part of this report.

1.2 PROJECT AND SITE DESCRIPTION

1.2.1 PROJECT DESCRIPTION

The project site is located approximately in the county of Frontenac, 7.8 km northeast of The City of Kingston near Kingston Mills, Ontario. Access to project site is north of Highway 401, west of Highway 15, westerly on Kingston Mills Road (County Road 21) to the project site

WSP's overall scope includes design of rehabilitation works for the locks and associated structures which have been identified as being in relatively poor condition.

WSP's geotechnical scope of work included the following:

- Review of existing historical geotechnical information;
- Drilling of six exploratory boreholes within the study area;
- Drilling four exploratory boreholes and two hand augers holes within the project area;
- Excavating three exploratory test pits near selected retaining and chamber walls;
- Coring the exterior walls of selected retaining walls;
- In-situ soil sampling and testing, including Standard Penetration Testing (SPT);
- Obtaining soil samples and rock core samples for additional review and laboratory testing;
- Laboratory testing;
- Geotechnical analysis; and
- Compiling all results into a single, concise report.

1.2.2 SITE DESCRIPTION

The existing lock station consists of a set of four locks, divided into two areas separated by a turning basin. The northern most lock (Lock 46), acts as a turning basin from Cranberry Lake while the south set of locks (Lock 47 through 49), provide access to the St. Lawrence River.

The topography of the land consists of steep hills with flat areas cut into or filled on top of the existing hillsides. The ground slopes downward to the south from approximately elevation 90.5 m to elevation 87.0 m at the top of the first set of locks. The ground continues to slope downwards to the south at the second set of locks to approximately elevation 86.1 m downwards past the railroad bridge to approximately elevation 78.8 m.

1.2.3 SUMMARY OF ORIGINAL LOCK CONSTRUCTION

Upon review of historical documents and site observations from supplemental work completed by WSP staff, the lock components are generally constructed on bedrock with some sections partially constructed on oak sills. Additional discussion on lock construction is presented in the ensuing paragraphs.

Lock 46 are built partially on bedrock and partially on oak sills. The oak sills support components of the structure upstream of the breast wall, where bedrock is not present. Site observations note that timbers are visible in the stream adjacent to the shore at the west upstream embankment, possibly from original construction of the Lockstation. The section of chamber floor in front of the breast wall towards the downstream gate was noted to be constructed on bedrock. Bedrock is also present behind the chamber walls. A centre bearing swing bridge is present and installed over this lock, with the centre pier and east abutment on bedrock immediately behind the west and east walls of the chamber lock, respectively. The east rest abutment that is utilized when the bridge is open to traffic is partially constructed on top of the east chamber wall.

Locks 47 and 48 are constructed within a blasted/excavated bedrock outcrop area. The west side walls of Locks 47 and 48 are built directly against the bedrock while the east side walls of Locks 47 and 48 were built elevated on bedrock and have the back of the walls exposed. The original bedrock floors in Locks 47 and 48 were overlaid in the past with concrete directly over bedrock in an apparent attempt to seal the bedrock and prevent water infiltration towards the breast walls.

At Lock 49, available information indicates that more than 70% of the lock is constructed on timber rafts placed on top of the fill with the north section of the lock supported by bedrock. The downstream end of Lock 49 was modified and repaired very shortly after original construction with some very significant structural interventions required in the years following construction. The past repairs appear to focus on the concern of global stability and resilience to settlement. The most recent major intervention was in 1972 when lower wingwalls and monoliths were dismantled, backfill replaced and masonry grouted.

2 INVESTIGATIVE METHODLOGY

2.1 DESKTOP STUDY

2.1.1 PUBLISHED SURFICIAL AND BEDROCK GEOLOGY MAPS

A review of available geological mapping (GSC Map M2227) noted that native soil in the general study area consists of shallow till and rock ridges. Precambrian bedrock in the area is of the Grenville Formation consisting of granite-gneiss.

2.1.2 PREVIOUS GEOTECHNICAL INVESTIGATIONS

To develop a thorough understanding of the existing site conditions in addition to determining where additional boreholes were to be placed, the following reports information was provided to WSP by PSPC for review:

<u>Site Investigation Services Limited, 1977 – Swing Bridge at Kingston Mills</u>

A geotechnical investigation was carried out to assess the soil and rock conditions behind the east abutment of the swing bridge at the Kingston Mills Lock station. A total 10 boreholes were advanced at selected locations in near the east abutment. Borehole locations from the investigation are presented on a borehole plan in Appendix A and also included on a site plan appended to the full report included in Appendix D.

Based on the results of the investigation, the following observations were reported:

South of the Swing Bridge:

- Overburden near the lock wall consisted of topsoil over clayey silt fill to a depth of 910 mm
- Bedrock was encountered within 910 mm of the top of the lock wall.
- Bedrock is exposed 4.57 to 6.1m from the wall line

North of the Swing Bridge

- At boreholes E and J, overburden consisted of topsoil over a native stony sandy clayey silt till.
- Bedrock was encountered at elevation 87.93 (2.9 m below the top of the concrete wall at boreholes E and J)
- At boreholes G, H, and I rockfill was encountered at approximately 1.5 m below grade.
- At borehole J, bedrock was observed at elevation 87.5.

<u>Golder Associates, 1979 – Borehole Results - Kingston Mills Lock Station Rideau Canal, Kingston Ontario</u>

The purpose of the geotechnical investigation was to provide supplemental information to Public Services and Procurement Canada on the condition of the soils underlying Lock 49.

Two boreholes were advanced within Lock 49:

- One at the upstream Sill.
- One at the downstream Sill.

<u>Trow Ontario Ltd., 1990 – Geotechnical Investigation – Grouting Test Program, Locks 47 and 48, Kingston Mills Locks, Kingston, Ontario</u>

A geotechnical investigation was carried out to assess the construction of lock 47 and 48. A total of 20 horizontal and 6 vertical boreholes were advanced through the structures as part of the investigation. A grouting program was also completed to assess the condition of the grouting techniques used in the rehabilitation of the locks. A detailed discussion on the extent of the grouting program can be found in Appendix D.

Quontacon Associates, 1999 – Kingston Mills Swing Bridge, Rideau Canal, Geotechnical Investigation

A geotechnical investigation was commissioned by Public Services and Procurement Canada to obtain an assessment of the foundations of the existing swing bridge (Lock 46) at the Kingston Mills Lockstation. A total of 14 vertical boreholes and cores were advanced within the study limits. The boreholes were placed at select locations as indicated below:

- 6 boreholes to obtain core samples within the abutments and pivot pier foundations
- 1 borehole through the bridge deck and concrete counter weight
- 4 boreholes near the bridge abutments (1 per quadrant of the bridge)
- 2 boreholes near the concrete pivot pier adjacent to the circular track
- 1 borehole through the concrete counterweight at the west end of the bridge.

Further details on the investigation and recommendations can be found in Appendix D.

Jacques Whitford, 2005 – Geotechnical Investigation for the Kingston Mills Lock No. 46, Kingston, Ontario

A geotechnical investigation was completed at Lock 46. A total of 11 cores were extracted at select locations to assess the material properties of the structure.

Golder Associates, 2015 – Geotechnical Investigation, Proposed Structural Rehabilitation – Kingston Mills Swing Bridge, Kingston, Ontario

Addition investigative work was completed to provide recommendations for the proposed rehabilitation of the swing bridge. A desktop study was completed which included a review of additional historical reports (not documented above):

- J.D. Lee Engineering Limited, 1976 – The foundation condition of the swing bridge at Kingston Mills, Ontario
- J.D. Lee Engineering Limited, 1977 – Swing Bridge at Kingston Mills

The field program consisted of

- Three horizontal boreholes
- One vertical borehole
- Four test pits
- In situ testing and a laboratory testing program

In general, soil conditions encountered consisted of 2.5 m to 3 m of fill over bedrock. A more detailed explanation of site conditions within the limits of the lock station is presented in Section 3.

Please refer to the full report presented in Appendix D.

2.2 SUPPLEMENTAL INVESTIGATION

A geotechnical investigation was completed by WSP in April and October 2017. The investigation included selective coring of the existing stone, borehole drilling, test pitting, installation of monitoring wells, laboratory testing of selected soil samples, geotechnical analysis and preparation of this report.

2.2.1 CORING INVESTIGATION

A total of 33 core holes (CH 1 through CH 33) were placed at selected locations within the study area as shown in Appendix A. Cores were advanced with diamond tipped coring equipment supplied by CCC Geotechnical and Environmental Drilling Ltd. Of Ottawa, Ontario.

During the field investigation, all drilling operations were supervised on a full-time basis by a member of WSP's geotechnical staff who logged the depths at which different soil strata were encountered and processed and transported samples to our accredited laboratory facilities in Ottawa.

2.2.2 BOREHOLE AND TEST PIT INVESTIGATION

A total of 4 boreholes (BH 17-1 through 3 and BH 17-6), 2 hand auger holes (HA 17-4 and 17-5) and 3 test pits (TP 17-1 through 17-3) where placed at key locations within the study areas in consultation with the structural team as shown in Appendix A.

Prior to the start of drilling and excavating activities, utility clearances were obtained for all borehole and test pitting locations. Boreholes were advanced with CME hydraulic drilling equipment. Test pits were excavated using a rubber tire backhoe. All equipment and operating staff was supplied by Canadian Environmental Drilling and Contracting Inc. of Ivernary, Ontario. Soil samples were obtained at selected intervals using split spoon sampling techniques in conjunction with Standard Penetration Testing (SPT). Field shear vane testing was completed in areas where cohesive soil was encountered.

Standpipe piezometers were installed at all borehole locations to permit the ongoing measurement of stabilized groundwater levels within the study area. Records of each piezometer installation completed are presented as drawings in the attached borehole logs presented in Appendix C.

During the field investigation, all drilling operations were supervised on a full-time basis by a member of WSP's geotechnical staff who logged the depths at which different soil strata were encountered as well as processed and transported samples to our accredited laboratory facilities in Ottawa.

2.2.3 LABORATORY TESTING PROGRAM

All recovered samples were visually reviewed and a laboratory testing program was carried out on selected soil samples which included natural moisture content, particle size analysis and Atterberg limits (plasticity) testing. Laboratory index testing results are presented on the individual borehole logs and are included in Appendix C.

Samples of the stone masonry and rock from the current WSP coring investigation was also tested for the following parameters:

- Percent Absorption
- Bulk Specific Gravity.
- Compressive Strength
- Uniaxial Compression

3 SUBSURFACE CONDITIONS

3.1 SOIL CONDITIONS

A summary of subsurface conditions encountered within the study limits is presented in the ensuing sections. A detailed description of the soil stratigraphy encountered at each borehole location is shown on the borehole log sheets shown in Appendix C and D. Please note that the factual descriptions shown in each borehole logs takes precedence over the generalized (and simplified) descriptions presented below.

3.1.1 TOPSOIL AND ORGANICS

Topsoil was encountered at the ground surface at the majority of all the boreholes and test pit locations. The topsoil thickness varied from 30 mm to 330 mm.

The measured topsoil thicknesses where encountered are summarized below.

Table 3.1 Measured Topsoil Thickness

LOCATION	TOPSOIL THICKNESS
Lock 46	30 mm to 330 mm
Lock 47	80 mm to 270 mm
Lock 48	160 mm to 240 mm
Lock 49	170 mm

3.1.2 PAVEMENT STRUCTURE – KINGSTON MILLS ROAD

Historical boreholes were advanced through the pavement structure of Kingston Mills Road during the 1977 investigation completed by Site Investigation Service, the 2000 investigation completed by Quontacon Associates and the 2015 investigation completed by Golder Associates.

East of Lock 46, four boreholes (77 G through I and 15-101) were advanced through the existing pavement platform. Field observations noted a flexible pavement structure (asphalt over granular fill). The asphalt thickness was observed to be a consistent 100 mm and generally supported by sand fill over clayey sandy silt to silty clay some sand fill that extended to depths ranging from 1.2 to 1.5m from surface within the westbound lane and extended to 2.7 m within the eastbound lane.

West of Lock 46, two boreholes were advanced through the existing pavement structure (00 C3 and C4). Conditions observed during the field investigation noted the presence of a composite pavement structure (asphalt over concrete). Asphalt thickness was observed to vary from 38 mm to 50 mm. The concrete base extended to depths ranging from 300 to 400 mm below the surface.

3.1.3 FILL

Fill material was also encountered beyond Kingston Mills Road throughout the project limits in all the current boreholes and in most of the previous boreholes. The depth of fill encountered in the various boreholes (both current and previous) ranged from surface to 4.6 m from surface.

The fill material includes a range of soil and rock materials, but is most typically described as ranging from silt with clay and varying amounts of gravel and sand to silty sand. Fill is often, by nature, a heterogeneous material and has likely been placed as part of multiple previous projects in the area. It should, therefore, be anticipated that variability will exist in the fill material (i.e. materials other than those described in the borehole logs could be encountered during construction).

SPT 'N' values within the fill material typically ranged from 3 blows to 29 blows per 305 mm of penetration through most of the project, indicating a loose to compact consistency soil.

Grain size curves for selected samples of fill material from the 2017 WSP Investigation is presented in Appendix B. A summary of these grain size distributions is also presented in the table below.

Table 3.2 Results of Grain Size Analyses for Fill

BOREHOLE NO.	SAMPLE NO.	GRAIN SIZE DISTRIBUTION			
		% Gravel	% Sand	% Silt	% Clay
15-1A*	2	0	8	92	
	3	2	6	92	
	4	0	7	93	
17-1	SS4	9	14	56	21
17-2	SS3	9	47	37	7
17-3	SS3	6	24	60	9
17-6	SS4	9	44	41	6
	SS6	2	22	58	18
TP17-2	GRAB 2	2	89	9	
	GRAB 3	3	33	53	12
TP17-3	GRAB 3	48	34	18	

Note: Approximate grain size distribution values interpreted from review of graphical chart available.*

3.1.4 TILL

Glacial till was reported to have been encountered during the 1977 Site Investigation Services. The glacial till consists of a heterogeneous mixture of gravel, sandy clayey silt till.

Table 3.3 Glacial Till

Investigation	Borehole	Depth Encountered (m)	Thickness (m)
Site Investigation Service - 1977	77-E	250 mm - 2.7 m	2.45
	77-F	230 mm - 2.3 m	2.07
	77-J	1.8 m - 2.7 m	0.90

3.1.5 AUGER REFUSAL AND BEDROCK

Auger refusal was encountered at 14 boreholes (12 at Lock 46, 1 at Lock 48 and 1 at Lock 49) drilled as part of previous investigations at depths ranging from of 100 mm to 2.7 m from surface. Bedrock was cored at boreholes 17- 1, 17-2, and 17-3 during the WSP investigation using 'NQ' sized diamond coring equipment. A summary of the elevation as which bedrock was encountered in the WSP investigations as well as within historical investigations is presented in Appendix F.

The rock encountered in the cored holes consisted of fresh granite. Rock Quality Designation (RQD) ranged from 0 to 100% (indicating a rock quality of "very poor" to "excellent"). Generally, the RQD values increase with depth (i.e. is typically "very poor" to "poor" quality near surface, and becomes "fair" to "excellent" quality with depth. When analyzing the rock quality results within the Kingston Mills Lock station, the RQD values were general observed to be good to excellent once encountered (one exception was at BH 17-2 where the RQD was observed to be 27%) and decreased in RQD value with depth.

Results of the testing are presented in Appendix C and D respectively.

3.1.6 SUMMARY

A summary of the sub-surface conditions noted in the historical information review encountered at the various boreholes is presented in Appendix G.

3.2 GROUNDWATER CONDITIONS

Groundwater measurements were obtained (by others) in various boreholes in 1979 and 2015. In addition, monitoring wells and data loggers were installed in the boreholes drilled as part of this investigation, and stabilized groundwater levels were obtained from during drilling and from site visits completed between the months of October and November 2017. All data logging devices have been left in the drilled boreholes as part of an ongoing groundwater monitoring program to assess fluctuations in the stabilized groundwater levels over the winter months. To date, groundwater data has been collected and processed from October to November 2017 and from November 2017 to May 2018. A final round of data collection will be completed in August 2018. A graphical representation of the groundwater information collected to date is presented in Appendix B.

A summary of the groundwater levels measured at the various boreholes and monitoring wells (in the current investigation as well as reported in previous investigations) is presented in the table below.

Table 3.4 Groundwater Level Data By Investigation

BH	Lock	Ground Surface Elevation (m)	Installation Depth (m)	Soil Type at Response Zone	Measured Groundwater Elevation/Depth (m)					
					1979	2015	2017			2018
					14 - May	14 - May	4-Oct	12-Oct	30-Nov	17-May
15-101	46	91.9	N/A	Bedrock		88.65/ 3.25				
17-1	46	90.5	1.93 – 3.45	Silt to Silty Sand Fill			86.92/ 3.58	88.33/ 2.17	88.49/ 2.01	88.67/ 1.83
17-2	46	90.5	1.21 – 2.43	Silty Sand Fill			87.73/ 2.77	88.33/ 2.17	88.19/ 2.31	
17-3	47	86.1	1.01 – 2.28	Silt Fill			83.02/ 3.08	84.12/ 1.98	84.12/ 1.98	85.19/ 0.91
17-6	49	78.8	5.2 – 6.7	Silt Fill/Sand Till			72.22/ 6.58	74.96/ 3.84	75.06/ 3.74	75.61/ 3.19
79-1	49	75.18	N/A	Silty Sand/Gravel/Boulder	76.20/ 1.02					

A summary of depth and dates where a notable change in static water levels were recorded is shown in the table below:

Table 3.5 Data Logger Information – Changes In Water Level

Borehole Location	Lock	Stabilized Static Water Level (m)	Date	Depth of Fluctuation (Elevation)	Change Water Level (m)
17-1	46	87.87	Oct 19/17 to Oct 20/17	88.29 - 87.90	0.39
		88.43	Octo 29/17 to Oct 30/17	87.90 – 88.87	0.97
		87.46	Nov 8/17 to Nov 14/17	88.87 – 87.82	1.05
		87.14	Dec 10/17 to Dec 11//17	87.25 – 87.72	0.47
			Jan 11/18 to Jan 12/18	87.07 - 88.07	1.00
		87.08	Feb 21/18 to Feb 26/18	87.14 - 87.77	0.63
		87.89	Mar 29/18 to Apr 5/18	87.10 – 87.81	0.71
17-2	46	88.34	Oct 29/17 to Oct 31/17	88.34 – 88.08	0.26
17-3	47	84.00	Oct 29/17 to Oct 31/17	83.99 - 84.95	0.96
			Nov 2/17 to Nov 3/17	83.99 – 84.61	0.62
			Nov 6/17 to Nov 7/17	83.99 – 84.68	0.69
			Dec 5/18	84.03 – 84.28	0.25
			Jan 11/18 to Jan 12/18	84.00 - 85.10	1.10
			Jan 24/18	84.00 - 84.47	0.47
			Jan 27/18	84.00 - 84.67	0.67
			Feb 14/18	84.00 - 84.66	0.66
			Feb 19/18 to Feb 28/18	84.01 – 84.86	0.85
			Mar 30/18 to Mar 31/18	84.02 – 84.31	0.29
			Apr 3/18 to Apr 5/18	83.99 – 84.71	0.72
			Apr 12/18 to Apr 19/18	84.02 – 84.78	0.76
			Apr 28/18	84.03 – 84.28	0.25
			May 17/18	83.99 – 85.66	1.67
17-6	49	74.48	Oct 29/17 to Oct 30/17	74.48 – 74.85	0.37
		74.55	Nov 9/17 to Nov 12/17	74.85 – 74.55	0.30
		74.94	Jan 10/18 to Jan 25/18	74.81 – 75.12	0.31
		75.12	Feb 18/18 to Mar 14/18	74.96 – 75.18	0.22
		74.87	Apr 4/18	74.92 – 75.37	0.45
		75.27	Apr 8/18 to Apr 17/18	75.00 – 75.27	0.27
		75.24	May 4/18	75.34 – 75.73	0.39
			May 17/18	75.28 – 75.55	0.27

Note: Maximum change in water level shown within date range indicated.
Gap in data from November 15 to November 30 due to removal and installation of data logger

4 DISCUSSION

4.1 GENERAL

This section of the report provides engineering guidelines and assessment related to the geotechnical design aspects of the project based on our interpretation of the available information described herein and project requirements. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information available for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities. Reference should be made to the Limitations of this Report, attached in Appendix H, which follows the text but forms an integral part of this document.

4.2 CLAY PUDDLE

An assessment of the presence of a clay puddle behind the lock walls was completed within the study limits as part of the geotechnical mandate for this assignment. The purpose of the clay puddle is to provide an impervious barrier to limit the water from travelling from the chamber of a lock into the surrounding fill. A clay puddle material can be defined as follows:

- Fill material directly behind the chamber wall for a thickness of 600 mm that consists of clay, coarse sand and/or fine gravel that is predominantly homogenous but may contain lenses of pure clay along with occasional small impurities such as smaller stones and pieces of masonry rubble and is fully and intentionally consolidated and generally free of organic matter.

It was noted during this assessment that soils information along the perimeter of the lock structures was limited primarily since the majority of the lock station is built within a bedrock outcrop. It was also noted that there was generally no laboratory testing data available to substantiate the soil descriptions provided in each historical report except for some limited testing completed as part of the 2015 Golder investigation. For ease of presentation, the results of the assessment are presented in the ensuing sections below, separated by lock.

Borehole locations are presented in Appendix A with historical soils reports provided in Appendix D.

4.2.1 LOCK 46 AND WINGWALLS

A total five boreholes were found to be within the estimated vicinity of where the clay puddle is expected to be located. It was also noted that of the historical information reviewed, there was no laboratory testing data available to substantiate the soil descriptions provided. A summary of the applicable borehole information is presented in the table below with borehole locations presented:

Table 4.1 Lock 46 – Clay Puddle Assessment

Investigation	Borehole	Location	Clay Puddle Material	
			Main Constituents	Y/N
1977 Site Investigation Services	77-J	Fill behind east chamber wall	Clay/Silt	Y
	77-K	Fill behind east chamber wall	Clay/Silt	Y
2015 Golder Associates	TP 15-1A	Fill behind east chamber wall	Clay/Silt	Y
2017 WSP	17-1	Fill behind northwest wingwall	Silt/Clay	Y
	17-2	Fill behind east monolith	Silt/Sand	N

Note: Y=Yes, N=No [indicates the material description is (or is not) a plausible match for the clay puddle material as described in historical sources; it is not possible to confirm with certainty given the uncertainty with construction and likely high variability of the original construction).

Of the locations where soils information is available around the perimeter of Lock 46 and relatively close to the structure, it is concluded that materials exhibiting the properties of a clay puddle are generally presented along the perimeter of the structure except for within the limits of the east monolith and northeast wingwall. Soil information in this area is limited to one borehole consisting of silty sand material with trace amounts of gravel and clay.

4.2.2 LOCK 47 AND WINGWALLS

Two boreholes and one test pit were observed to be advanced within the general vicinity of where the clay puddle was anticipated. No laboratory testing information was available to substantiate the soil descriptions provided in the historical reports. No assessment was completed within the limits of the east chamber wall due to lack of soils information and the majority of the structure is above grade.

A summary of applicable borehole locations is presented in the following table:

Table 4.2 Lock 47 – Clay Puddle Assessment

Investigation	Borehole/ Test pit	Location	Clay Puddle Material	
			Main Constituents	Y/N
1990 Trow Ontario Ltd	90-H1	West Chamber Wall	Silt/Clay	Y
	90-H2	West Chamber Wall	Silt/Clay	Y
2017 WSP	TP17-1	West Chamber Wall	Silt/Clay	Y

Note: Y=Yes, N=No [indicates the material description is (or is not) a plausible match for the clay puddle material as described in historical sources; it is not possible to confirm with certainty given the uncertainty with construction and likely high variability of the original construction).

Based on the limited data available, it is likely that a clay puddle is present along the western perimeter of Lock 47 from the northwestern wingwall to the downstream monolith.

4.2.3 LOCK 48

A review of past and current information noted three boreholes were advanced within the area where a clay puddle was anticipated. In general, a clay puddle may be present along the east side of Lock 48 and likely not present along the west side of the lock as the chamber wall rests against a bedrock outcrop.

A summary of relevant locations where a clay puddle should be present is shown in the table below.

Table 4.3 Lock 48 – Clay Puddle Assessment

Investigation	Borehole/ Test pit	Location	Clay Puddle Material	
			Main Constituents	Y/N
1990 Trow Ontario Ltd.	90-H17	East Chamber Wall (Near Stairs)	Silt/Clay	Y
2017 WSP	TP 17-3	Downstream Monolith	Sand/Gravel	N
	HA-17-4	West Chamber Wall	Sand/Gravel	N

Note: Y=Yes, N=No [indicates the material description is (or is not) a plausible match for the clay puddle material as described in historical sources; it is not possible to confirm with certainty given the uncertainty with construction and likely high variability of the original construction).

4.2.4 LOCK 49 AND WINGWALLS

Three boreholes were advanced within the vicinity of where a clay puddle is anticipated. As with the other locks, most of the soils information available outside of the 2017 investigation completed by WSP did not have any laboratory testing results available to confirm the soil descriptions provided. Along the west section of the wall, it information the limited to a shallow hand auger and site observations that the lock likely rests against a bedrock outcrop due to the exposed rock face visible throughout. Within the limits of the east chamber wall, the information available is limited to a single borehole and test pit completed during the 2017 WSP investigation. Available information within the east section of the structure shows clay fill material under the structure at the downstream monolith, upstream breast wall, and midpoint of the east chamber wall which would suggest the presence of a clay puddle. However, a sand and gravel based material was encountered between the midpoint of the east chamber wall and the southeast wingwall. While a clay puddle may be present beneath the structure, and the north section of the east chamber wall, it is unclear if the clay puddle extends southerly towards the southeast wingwall at Lock 49.

A summary of relevant locations where a clay puddle should be present is shown in the table below.

Table 4.4 Lock 49 – Clay Puddle Assessment

Investigation	Borehole/ Test pit	Location	Clay Puddle Material	
			Main Constituents	Y/N/M
2017 WSP	HA-5	West Chamber Wall	Silt/Sand/Gravel	N
	BH 17-6	East Chamber Wall	Silt/Sand	N
	TP 17-2	East Chamber Wall	Silt/Sand/Clay	M

Note: Y=Yes, N=No, M=Maybe [indicates the material description is (or is not) a plausible match for the clay puddle material as described in historical sources; it is not possible to confirm with certainty given the uncertainty with construction and likely high variability of the original construction).

4.3 SEISMIC CONSIDERATIONS

4.3.1 LIQUIFACTION POTENTIAL

Based on the available information, the structure foundations are resting on bedrock and are seismic liquefaction is therefore not an issue.

4.3.2 SEISMIC SITE CLASSIFICATION

Based on available information, it is understood that the structures are founded on Bedrock. A Site Class C may be assumed for the design of upgraded structures. It is possible that the site classification could be upgraded to a Site Class A or Site Class B (given the shallow bedrock) however this would require a site-specific measurement of shear wave velocities.

4.4 FOUNDATIONS

4.4.1 FROST PROTECTION

The depth of frost penetration for the site may be assumed to be 1.5 m. All foundation elements should therefore have a permanent soil cover of at least 1.5 m (or its thermal equivalent if artificial insulation is used).

4.4.2 BEARING CAPACITY

The following bearing resistances on bedrock may be assumed:

- The unfactored ultimate geotechnical bearing resistance can be taken as 3 kPa. A resistance factor of 0.5 should be applied to this value, yielding a factored bearing resistance of 1.5 kPa at ULS (Ultimate Limit States).
- The settlement of sound rock at normal bearing pressures is typically less than the normally accepted 25 mm limits and therefore SLS conditions generally do not govern the design of foundations constructed on rock.

If there are new structures (or portions of existing structures) founded on soil, the following parameters may be assumed:

- The unfactored ultimate geotechnical bearing resistance can be taken as 300 kPa. A resistance factor of 0.5 should be applied to this value, yielding a factored bearing resistance of 150 kPa at ULS (Ultimate Limit States).
- The geotechnical resistance at the Serviceability Limit State (SLS) can be taken as 100 kPa.

All bearing surfaces should be checked, evaluated and approved at the time of construction by a geotechnical engineer who is familiar with the findings of this investigation and the design and construction of similar projects prior to placement of any concrete, back fill, etc.

4.4.3 LATERAL EARTH PRESSURES

Lateral Earth Pressure

The lateral earth pressure acting on retaining walls, etc. may be calculated using the following expression:

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

Where:

P = lateral earth pressure (kPa) acting at depth h

K = earth pressure coefficient; for unrestrained walls and structures where some movement is acceptable (such as retaining walls) use a coefficient of active earth pressure (K_a) equal to 0.27, for restrained walls use the coefficient of earth pressure at rest (K_0) equal to 0.42

γ = the density of the backfill; use 21.5 kN/m^3 for compacted granular backfill

h = the depth to the point of interest (m)

q = the magnitude of any design surcharge at the ground surface;

The above values assume free-draining granular backfill will be used. If this is not the case then the above values may need to be adjusted based on the soil type used, and water pressures should be considered in the calculation of lateral pressures. WSP can provide additional guidance based on actual plans if required.

The passive resistance offered by the foundation wall backfill soils could also be considered in evaluating the lateral resistance applied to the foundations. The magnitude of that lateral resistance will depend on the backfill materials and backfill conditions adjacent to the foundation walls. If the backfill materials consist of compacted sand or sand and gravel (OPSS Granular B Type I) as discussed herein, then the passive resistance acting on the foundation wall may be taken as:

$$\sigma_h(z) = K_p\,(\gamma\, z+q)$$

where:

$\sigma_h(z)$= lateral earth resistance applied to the foundation wall at depth z, kilopascals

K_p = passive earth pressure coefficient, use 3.7

γ = unit weight of retained soil, use 21.5 kN/m^3

z = depth below top of wall, metres

q = the magnitude of any design surcharge at the ground surface;

This resistance is provided in unfactored format. Factoring of the calculated resistance value will be required if the design is being carried out using Limit States Design.

Movement of the backfill and wall is required to mobilize the passive resistance. As a guideline, approximately 75 mm of movement would be required to mobilize the passive resistance.

Seismic Earth Pressure

Earth pressures will be higher under seismic loading conditions. In order to account for seismic earth pressures the total earth pressure during a seismic event (including both the seismic and static components) may be assumed to be:

$$\sigma h(z) = K_a \gamma z + (K_{AE} - Ka)\, \gamma\, (H\text{-}z)$$

Where:

- $\sigma h(z)$ = the total earth pressure at depth z (kPa);
- K_a = the active earth pressure coefficient (0.27);
- γ= the unit weight of soil (21.5 kN/m^3 for granular fill or 19 kN/m^3 for native soils);
- K_{AE} = the combined active earth pressure and seismic earth pressure coefficient (use 0.8);
- H = the total height of the wall (m)
- z = the depth below the top of the wall (m)

The above earth pressure values (both static and seismic) are unfactored values.

The sliding resistance can be calculated using the following unfactored friction coefficients given in the table below:

Table 4.5 Unfactored Friction Coefficients

Condition	Unfactored Friction Coefficient, tan σ
Between concrete and clean bedrock	0.7
Between concrete and native granular soils	0.5

4.5 CONSTRUCTION CONSIDERATIONS

Is it understood that some excavation work is required as part of the overall rehabilitation work proposed at the Kingston Mills Lock station. Where work is required at or near the lock structures to expose structural elements, the following recommendations are provided:

4.5.1 TEMPORARY DEWATERING

The loose sand and silt based fill material (which will be present below the water table through much of the excavation) has the potential to behave as “flowing” soils with virtually no stand-up time if not adequately supported or dewatered in advance of excavation. For the purposes of planning where open excavation work will be required, it should be assumed that the groundwater level will need to be drawn down to at least one meter below the excavation invert (again by means of an active dewatering system such as wells or well points) where the base is in soil.

Allowing groundwater to simply seep uncontrolled into the excavation to be pumped out as work proceeds is likely to result in excessive base disturbance, heaving, sloughing of excavation sidewalls, extensive over-excavation and replacement, and potentially unsafe working conditions.

Bedrock is unlikely to be sensitive to groundwater seepage, and in areas where the base and sidewalls of the excavation below the water table are primarily in rock, pumping from suitably filtered sumps is likely to be adequate.

4.5.2 TEMPORARY EXCAVATIONS

All excavations should be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). Part III of Ontario Regulation 213/91 deals with excavations.

The soils within the expected excavation near the lockstation include granular and cohesive fill, native sands with silty clay. These soils can be classified as a Type 3 Soil above the groundwater table (or depth of de-watering) and Type 4 soils below the groundwater table (or depth of de-watering). These classifications must be reviewed and confirmed by a qualified person during excavation. Excavations within Type 3 soil typically require side slopes with a minimum gradient of 1 horizontal to 1 vertical and excavations within Type 4 soil typically require side slopes of 3 horizontal to 1 vertical.

If limited space is available then a temporary shoring system may be required. Once the location and elevations of the excavations are determined, the need for vertical shoring should be reviewed. The type of shoring to be used depends on the permissible movement of the shoring. The design of any the shoring system must be carried out by a professional engineer and take into consideration the effect of the excavation upon the neighboring buildings and structures. The contractor is typically responsible for the detailed design of temporary shoring.

If required, WSP can provide additional guidance based on preliminary excavation plans, depths, etc. during the detailed design phase of the project.

4.5.3 SUBGRADE PREPARATION

The geotechnical bearing resistances provided in Section 4.6 assume that the foundation soils will not be disturbed by construction activities. Proper de-watering and protection of the exposed subgrade will be important where the existing facilities require exposure below the ground surface to complete rehabilitation work. All excavated surfaces should be kept free of frost, water, etc. during construction. All excavated surfaces should be inspected by a qualified geotechnical engineer who is familiar with the findings of this investigation and the design and construction of similar structures.

The foundations soils at the site are expected to be sensitive to disturbance from ponded water and construction traffic if the subgrade for the foundations is exposed for a prolonged duration and/or exposed to construction traffic then placement of a mud slab directly on the subgrade may be required to protect the subgrade from these elements.

4.5.4 BACKFILLING AND COMPACTION

Should excavation of fill be required as a component of the rehabilitation work, it is recommended that the clay puddle be reinstated where required using suitable clay fill material as defined in Section 4.2 with a low susceptibility to frost heave. Upon completion of the reinstatement of the clay puddle where required, remaining fill may include suitable portions of the existing soils that meet OPSS Select Subgrade Material gradation requirements or improved imported fill material. Backfill should be placed in shallow lifts, not exceeding 200 mm loose thickness, and compacted to 98% SPMDD where it is supporting any structures or services, or 95% in other areasf required, the suitability of imported materials should be confirmed prior to placement from both a geotechnical and environmental perspective. Portions of the existing soils at the site are adequate for use as general earth fill, but may require moisture conditioning (either wetting or drying) prior to placement and compaction.

To avoid damaging or laterally displacing the structures, care should be exercised when compacting fill adjacent to the existing lock structures. Heavy equipment should be kept a minimum of 1 m away from the lock structure during backfilling. It is recommended that within a 1m width adjacent to the chamber walls should be compacted using hand-operated equipment unless otherwise authorized.

Only material with a low susceptibility to frost heave should be used as backfill against exterior or unheated foundation elements (e.g., footings, foundation walls, pile caps, etc.). Remaining fill should consist of the following:

- Non-frost-susceptible sand and/or gravel which meets that gradation requirements for OPSS Granular A or Granular B;
- 19-millimetre clear crushed stone having a unit weight not exceeding 21.5 kN/m^3, which is separated from other soils with a Class II non-woven geotextile having an FOS not exceeding 100 microns to prevent loss of adjacent sand, or silty soils into the clear stone. It should be noted that the use of clear stone as foundation backfill may lead to unfavourable growing conditions for plant matter placed in overlying topsoil.

4.5.5 WINTER CONSTRUCTION

Should construction be carried out during freezing temperatures, exposed frost susceptible subgrade fill should be protected immediately from freezing using one or a combination of straw, propane heaters, polystyrene insulation, insulated tarpaulins, or other suitable means that prevent the underlying soil from freezing, which could cause frost heave.

5 CLOSURE

WSP should be retained for a general review of the final design and specifications to verify that this report has been properly interpreted and implemented. If not accorded this opportunity to make this review, WSP assumes no responsibility for the interpretation of the recommendations contained in this report.

Recommendations presented in the body of this report have been made based on our present understanding of the project requirements. We trust this information satisfies the requirements of the Public Services and Procurement Canada at this time. Should you have any questions on the contents of this report, please do not hesitate to contact the undersigned.

Sincerely,

Shawn Lapain, P.Eng.
Geotechnical Engineer

Chris Hendry, P.Eng.
Senior Geotechnical Engineer

APPENDIX

A LOCATION OF BOREHOLES

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

VERTICAL HISTORIC TEST HOLE
1977 SITE INVESTIGATION SERVICES VERTICAL BOREHOLE
2000 QUONTACON ASSOCIATES VERTICAL BOREHOLES
2005 JACQUES WHITFORD VERTICAL BOREHOLES
2015 GOLDER ASSOCIATES VERTICAL BOREHOLES AND TEST PITS

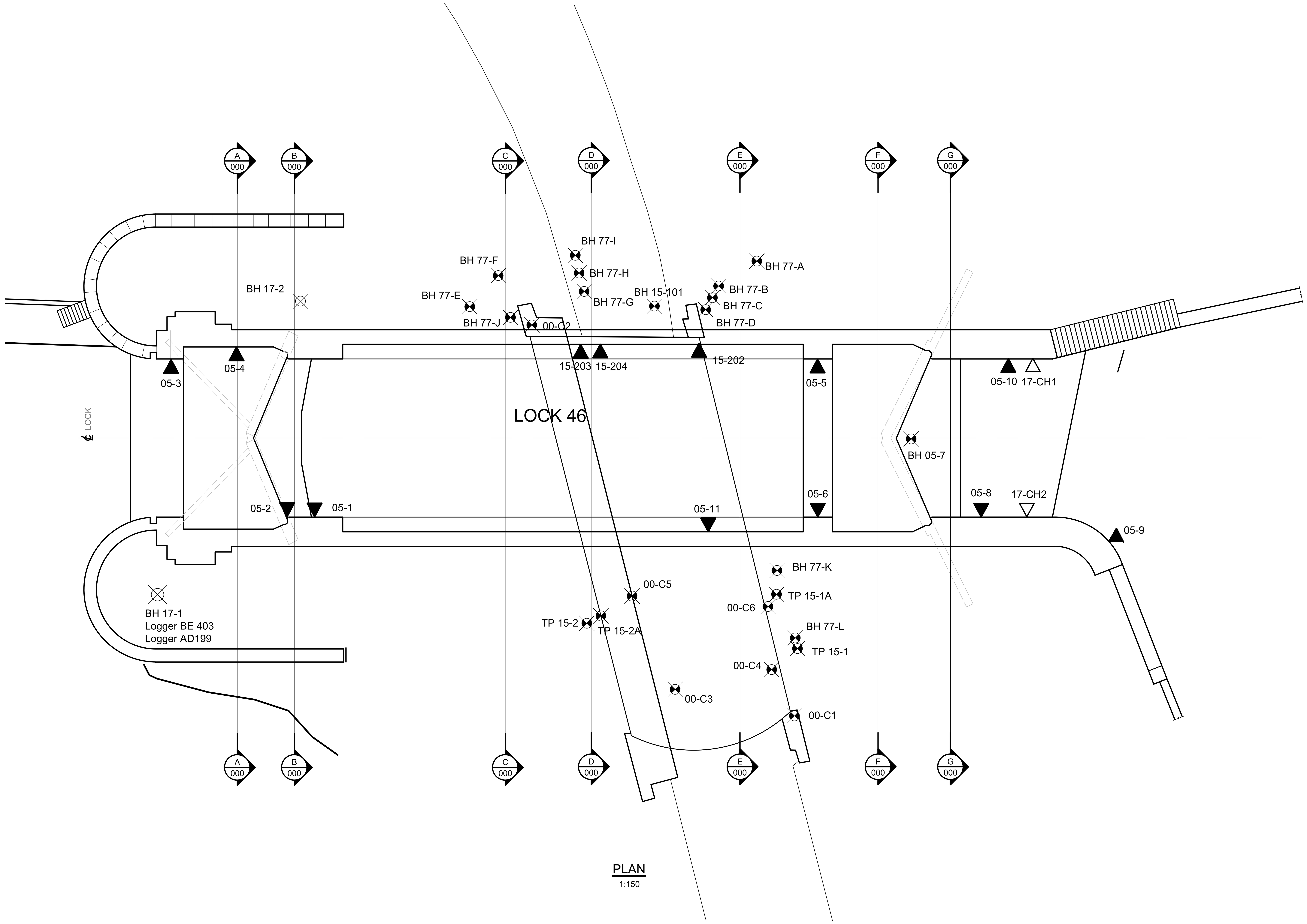
HORIZONTAL HISTORIC TEST HOLE
2005 JACQUES WHITFORD HORIZONTAL BOREHOLES
2015 GOLDER ASSOCIATES HORIZONTAL BOREHOLES

VERTICAL 2017 WSP INVESTIGATION
2017 VERTICAL BOREHOLE
2017 TEST PIT
2017 VERTICAL CORE

HORIZONTAL 2017 WSP CORES

XX XX - XX
TEST HOLE ID
YEAR OF STUDY
TYPE OF TEST HOLE:
BH BOREHOLE
TP TEST PIT
HA HAND AUGER
- CORE HOLE

NOTE:
ALL LOCATIONS ARE APPROXIMATE.



PLAN
1:150

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Services publics et Approvisionnement Canada

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Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

Parks Canada
Parcs Canada

Canadä

wsp

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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet
RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin
LOCK 46
BOREHOLE AND TEST PIT LOCATIONS

Drawn by / Dessiné par: BN/AM/JH
Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON
Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION
Drawing Number / Numéro du Dessin: GR-02
Project Number / Numéro du projet: R.079796.009
Sheet / Feuille 2 of / du 3

PWGSC-B1 (1000 x 707) PLOTTED BY: SHAWN.LAPAIN DATE PLOTTED: Mar 09, 2018 LAYOUT NAME: 01 FILE NAME: 171-02359-00_46-49Plan_v0.3_20180215.dwg

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

VERTICAL HISTORIC TEST HOLE
1979 GOLDER ASSOCIATES BOREHOLE
1990 TROW ASSOCIATES VERTICAL BOREHOLE

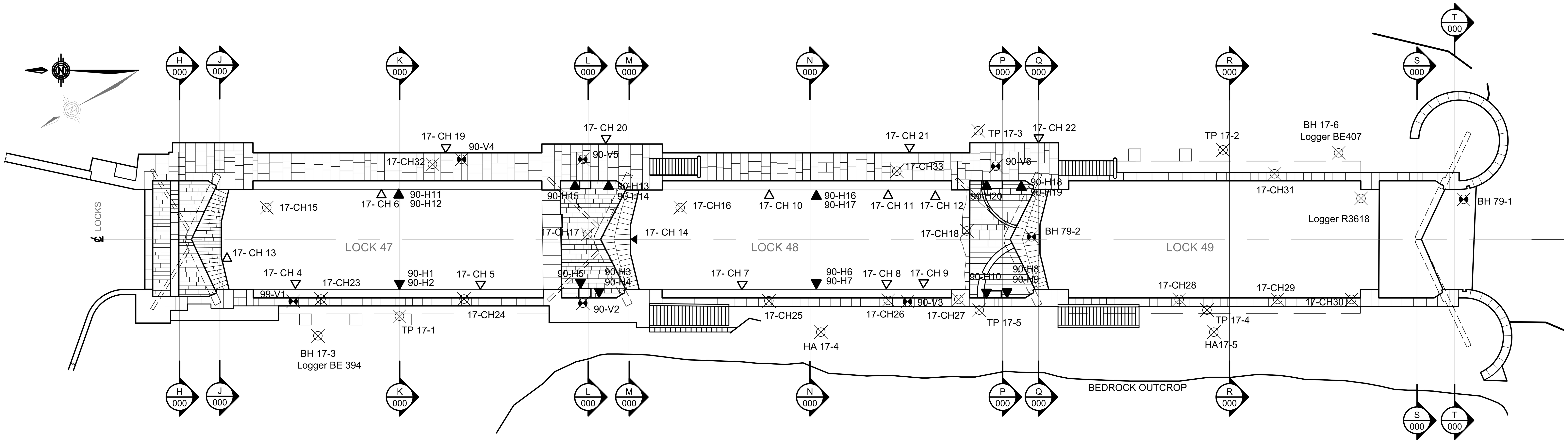
HORIZONTAL HISTORIC TEST HOLE
1990 TROW ASSOCIATES HORIZONTAL BOREHOLE

VERTICAL 2017 WSP INVESTIGATION
2017 VERTICAL BOREHOLE AND VERTICAL CORE
2017 HAND AUGER
2017 TEST PIT

HORIZONTAL 2017 WSP CORES

XX XX - XX
TEST HOLE ID
YEAR OF STUDY
TYPE OF TEST HOLE:
BH BOREHOLE
TP TEST PIT
HA HAND AUGER
- CORE HOLE

NOTE:
ALL LOCATIONS ARE APPROXIMATE.



PLAN
1:250

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Parks Canada Infrastructure Directorate

Groupe Canaux historiques et travaux d'ingénierie
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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number
Numéro du détail
B Location dwg. number
Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

LOCK 47, 48, & 49
BOREHOLE AND TEST PIT
LOCATIONS

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-02
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 3 of / du 3

APPENDIX

B PHOTOS

Lock 46
East Side of Lock Station – Lower Gate/Monolith

Lock 46
West Side of Lock Station – Lower Gate/Monolith/Buttress

Lock 46
Chamber – East Wall

Lock 46
Chamber – West Wall

Lock 46
East Side – Sluice Tunnel

Lock 46
West Side – Sluice Tunnel

Lock 46
Breast Wall

Lock 46
Floor

Lock 46
East Side of Lock – Southeast Wing wall

Lock 46
West Side of Lock – Southwest Wing wall

Turning Basin
East Wall

Turning Basin
West Wall

Turning Basin
Floor

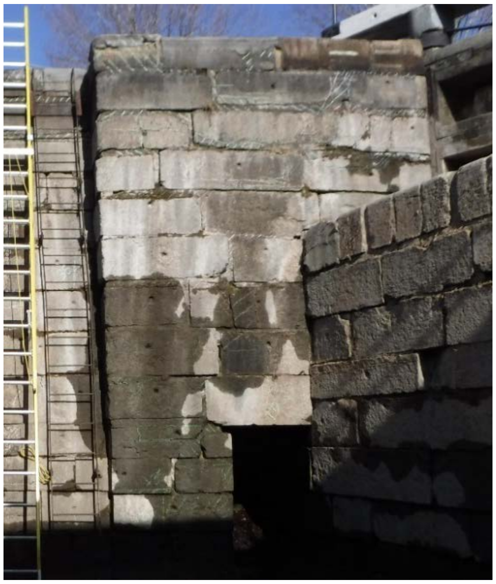
Turning Basin
Basin Sluice

Lock 47
East Upstream Walls

Lock 47
West Upstream Walls

Lock 47
East Side – Monolith

Lock 47
West Side – Monolith



Lock 47
East Side – Chamber Wall

Lock 47
West Side – Chamber Wall

Lock 47
East Side – Sluice Tunnel

Lock 47
West Side – Sluice Tunnel

Lock 47
Breast Wall

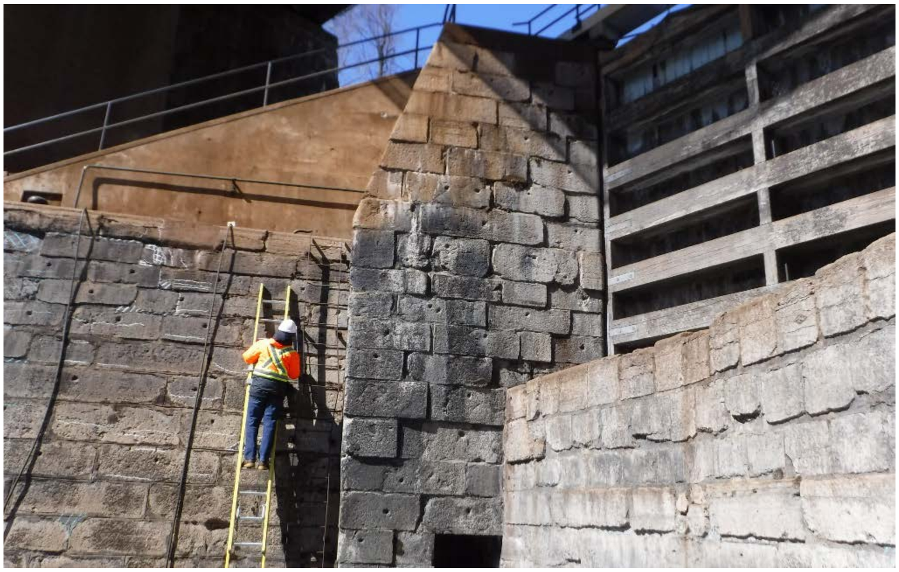
Lock 47
Floor

Lock 47
East Side – Buttress/Vertical Wall

Lock 47
West Side – Buttress/Vertical Wall

Lock 48
East Side –Monolith

Lock 48
West Side – Lower Gate/Monolith



Lock 48
East Side – Chamber Wall

Lock 48
West Side – Chamber Wall

Lock 48
East Side – Sluice Tunnel

Lock 48
West Side – Sluice Tunnel

Lock 48
Breast Wall

Lock 48
Floor

Lock 48
East Side – Buttress and Vertical Wall

Lock 48
West Side – Buttress and Vertical Wall

Lock 49
East Side – Chamber Wall

Lock 49
West Side – Chamber Wall

Lock 49
Wingwall

Lock 49
Breast Wall

APPENDIX

C WSP INVESTIGATION

APPENDIX

C-1 *CORING INVESTIGATION*

Core Location at East Pier

ID	CH 1
Date Cored	April 21, 2017
Lock	46
Location	Pier
Section	46E-PIE10
Direction Core Extracted	Horizontal
Tested	No
Total Hole Depth	490 mm
General Notes: Concrete 0 – 490 mm	

Core Location at West Pier

ID	CH 2
Date Cored	April 21, 2017
Lock	46
Location	Pier
Section	46W-PIE9
Direction Core Extracted	Horizontal
Tested	No
Total Hole Depth	470 mm
General Notes: 0 – 470 mm	

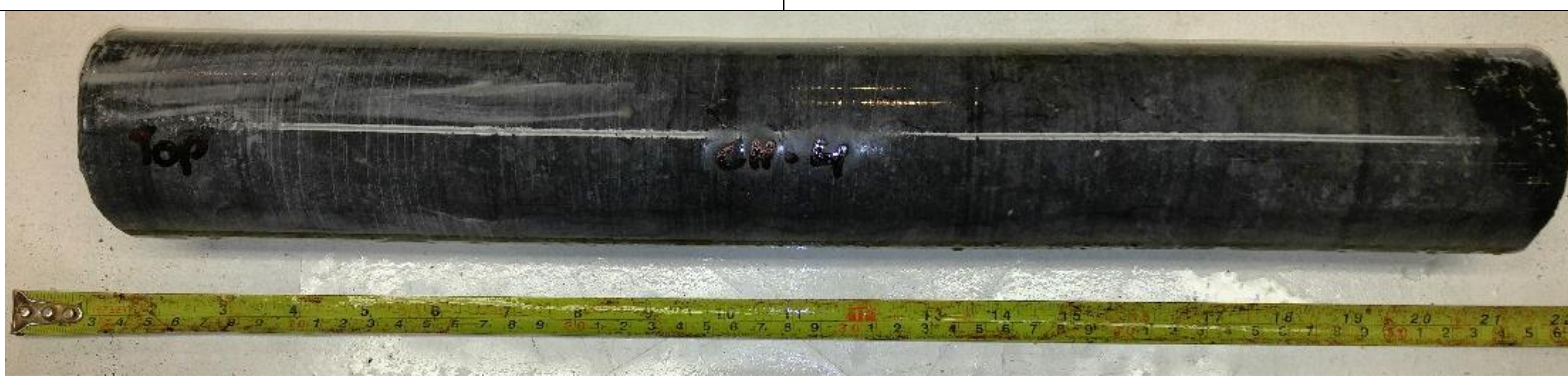
Core Location at North Basin Wall

ID	CH 3
Date Cored	April 21, 2017
Lock	Turning Basin
Location	Basin Wall
Section	BAS-NBW1
Direction Core Extracted	Horizontal
Tested	No
Total Hole Depth	410 mm
General Notes: Concrete 0 -410 mm	

Core Location at West Chamber Wall

ID	CH 4
Date Cored	April 21, 2017
Lock	47
Location	Chamber Wall
Section	47W-CHW2
Direction Core Extracted	Horizontal
Tested	No
Total Hole Depth	470 mm

General Notes:
Limestone 0 – 470 mm



Core Location at West Chamber Wall

ID	CH 5
Date Cored	April 21, 2017
Lock	47
Location	Chamber Wall
Section	47W-CHW1
Direction Core Extracted	Horizontal
Tested	Yes
Total Hole Depth	480 mm

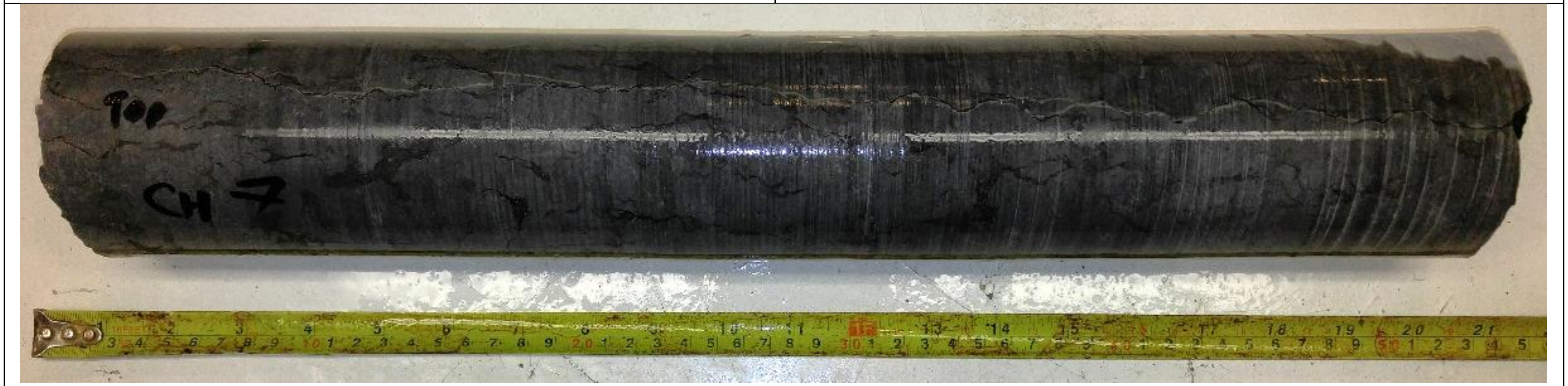
General Notes:
Limestone 0 – 480 mm

Core Location at East Chamber Wall

ID	CH 6
Date Cored	April 21, 2017
Lock	47
Location	Chamber Wall
Section	47E-CHW1
Direction Core Extracted	Horizontal
Tested	No
Total Hole Depth	500 mm
General Notes: Limestone 0 – 500 mm	

Core Location at West Chamber Wall

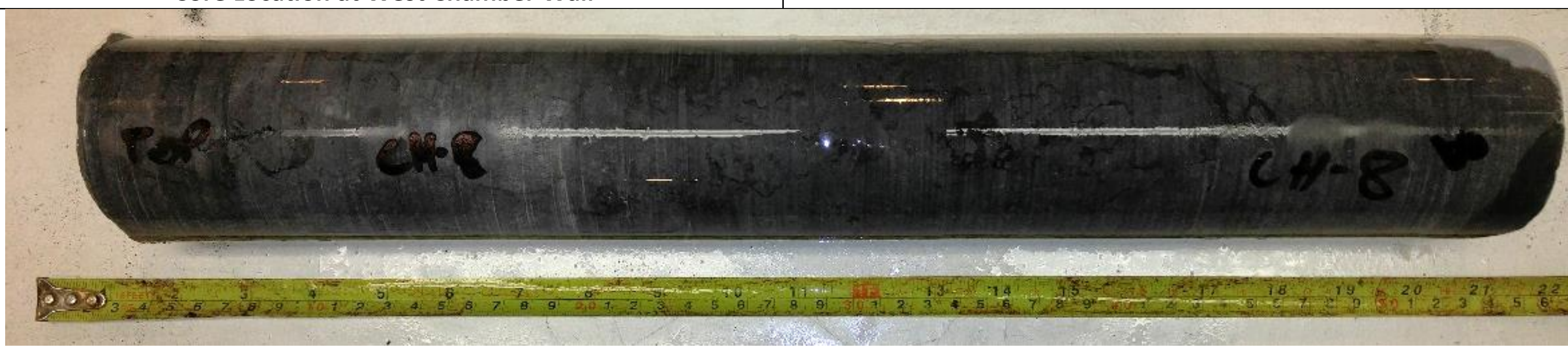
ID	CH 7
Date Cored	April 21, 2017
Lock	Chamber Wall
Location	48
Section	Chamber Wall
Direction Core Extracted	Horizontal
Tested	No
Total Hole Depth	480 mm
General Notes: Limestone 0 – 480 mm	



Core Location at West Chamber Wall

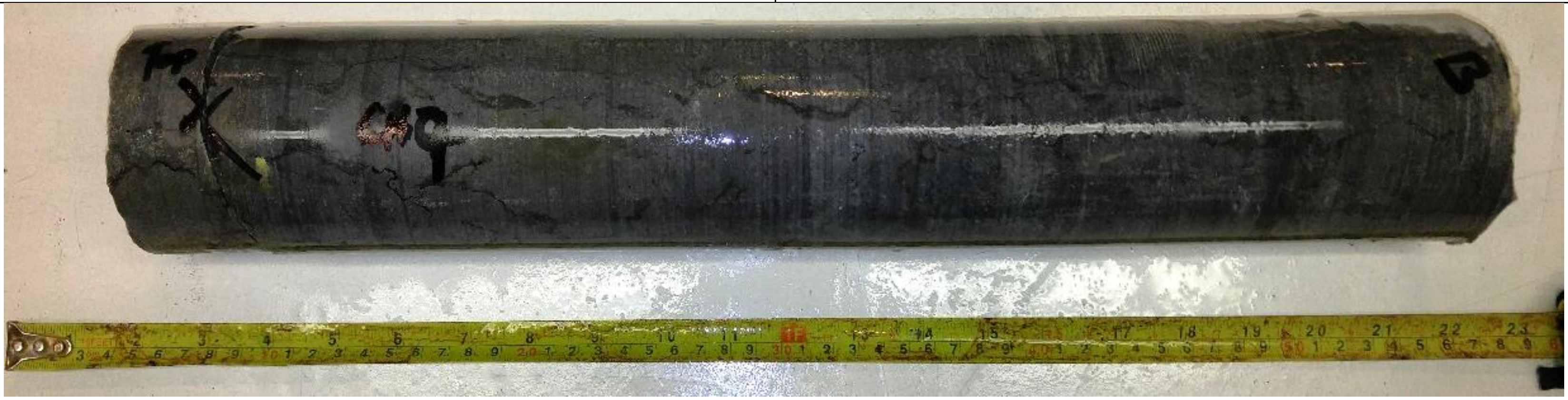
ID	CH 8
Date Cored	April 21, 2017
Lock	48
Location	Chamber Wall
Section	48W-CHW1
Direction Core Extracted	Horizontal
Tested	Yes
Total Hole Depth	480 mm

General Notes:
Limestone 0 – 480 mm



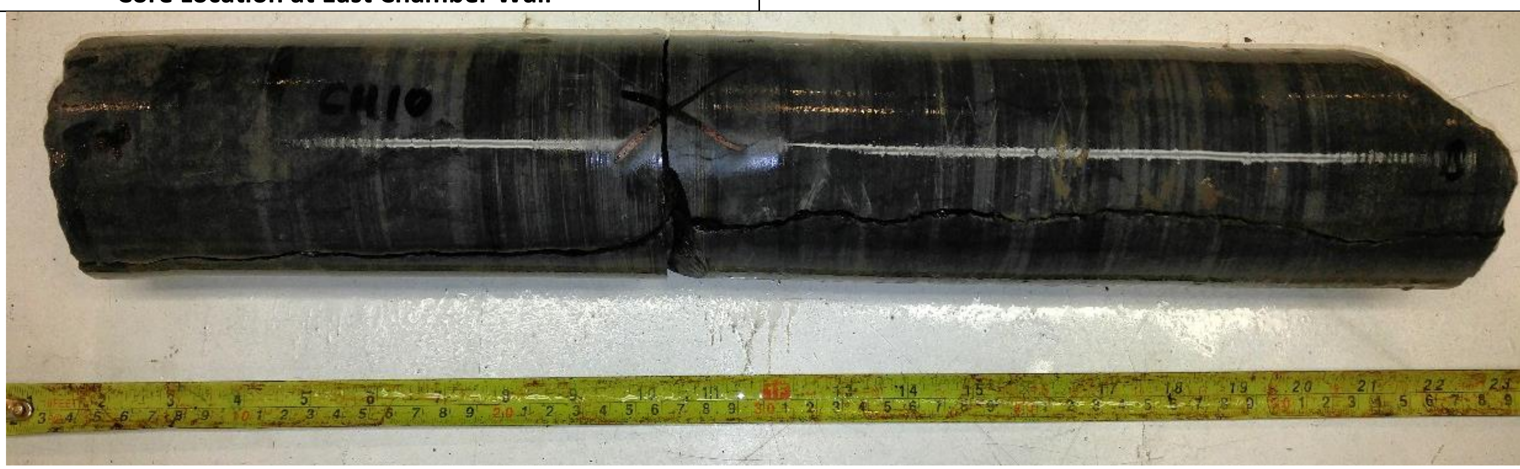
Core Location at West Chamber Wall

ID	CH 9
Date Cored	April 21, 2017
Lock	48
Location	Chamber Wall
Section	48W-CHW1
Direction Core Extracted	Horizontal
Tested	Yes
Total Hole Depth	480 mm
General Notes: Limestone 0 – 480 mm	



Core Location at East Chamber Wall

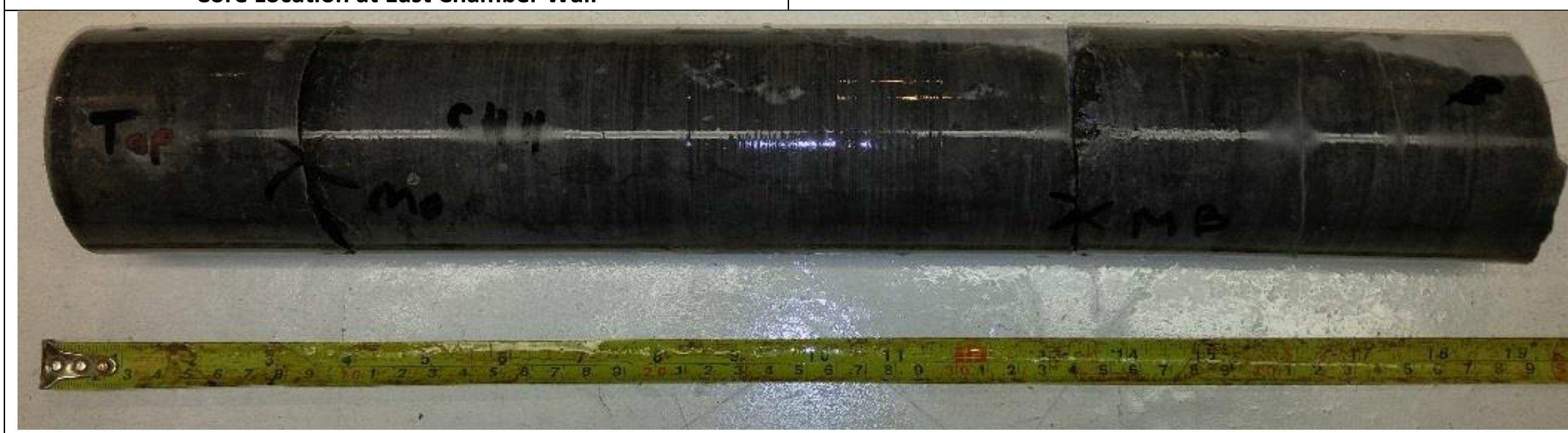
ID	CH 10
Date Cored	April 21, 2017
Lock	48
Location	Chamber Wall
Section	48E-CHW1
Direction Core Extracted	Horizontal
Tested	No
Total Hole Depth	480 mm
General Notes: Limestone 0 – 480 mm	



Core Location at East Chamber Wall

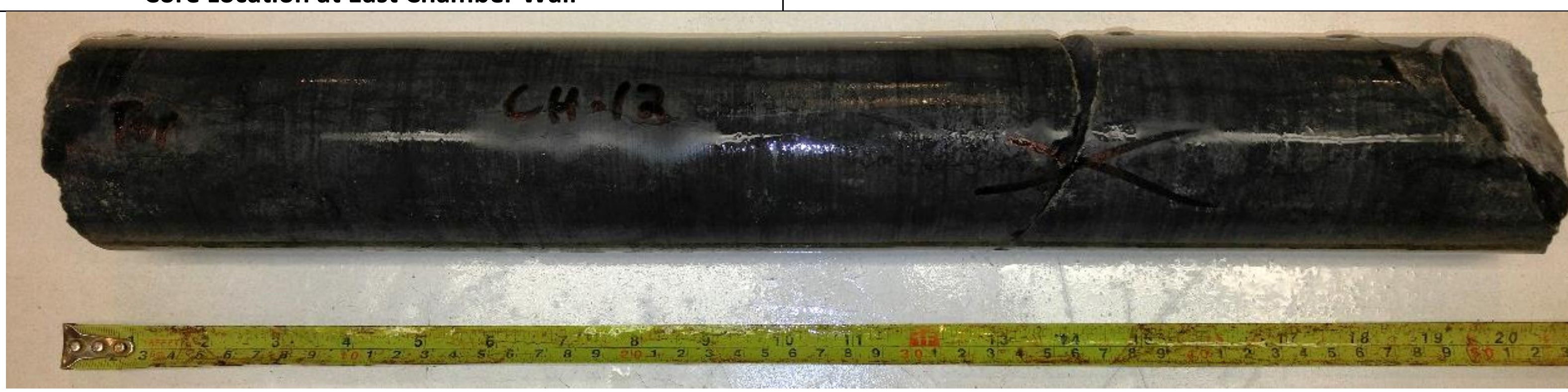
ID	CH 11
Date Cored	April 21, 2017
Lock	48
Location	Chamber Wall
Section	48E-CHW2
Direction Core Extracted	Horizontal
Tested	Yes
Total Hole Depth	440 mm

General Notes:
Limestone 0 – 440 mm



Core Location at East Chamber Wall

ID	CH 12
Date Cored	April 21, 2017
Lock	48
Location	Chamber Wall
Section	48E-CHW2
Direction Core Extracted	Horizontal
Tested	Yes
Total Hole Depth	480 mm
General Notes: Limestone 0 – 480 mm	



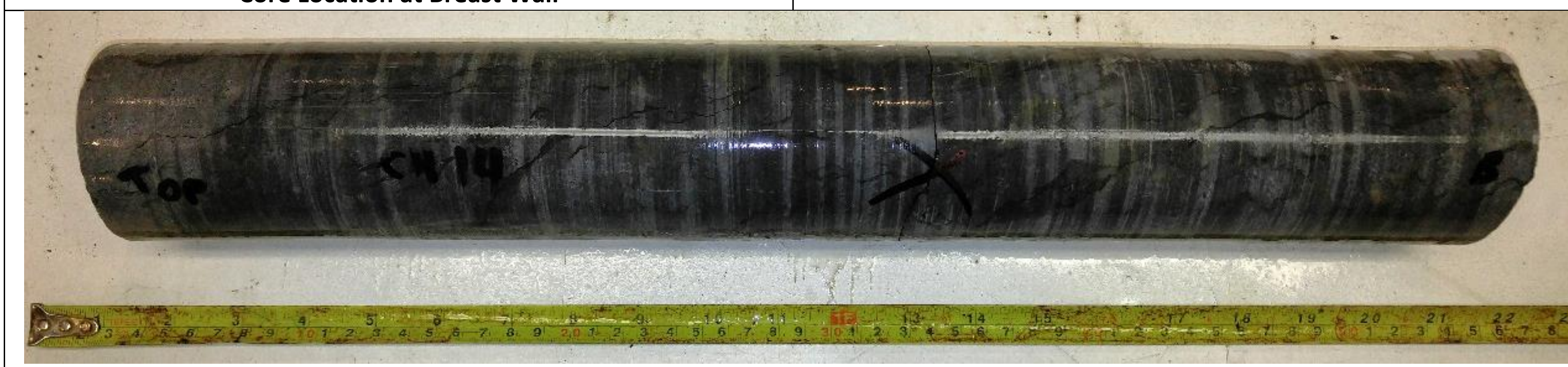
Core Location at Breast Wall

ID	CH 13
Date Cored	April 21, 2017
Lock	47
Location	Breast Wall
Section	47-BRE4
Direction Core Extracted	Horizontal
Tested	No
Total Hole Depth	480 mm

General Notes:
Limestone 0 – 480 mm

Core Location at Breast Wall

ID	CH 14
Date Cored	April 21, 2017
Lock	48
Location	Breast Wall
Section	48-BRE3
Direction Core Extracted	Horizontal
Tested	Yes
Total Hole Depth	480 mm
General Notes: Limestone 0 – 480 mm	



Core Location at Lock Floor

ID	CH 15
Date Cored	April 21, 2017
Lock	47
Location	Floor
Section	47 FLR
Direction Core Extracted	Vertical
Tested	NO
Total Hole Depth	480 mm

General Notes:
Concrete 0 – 270 mm
Granite 0 – 480 mm

Core Location at Lock Floor

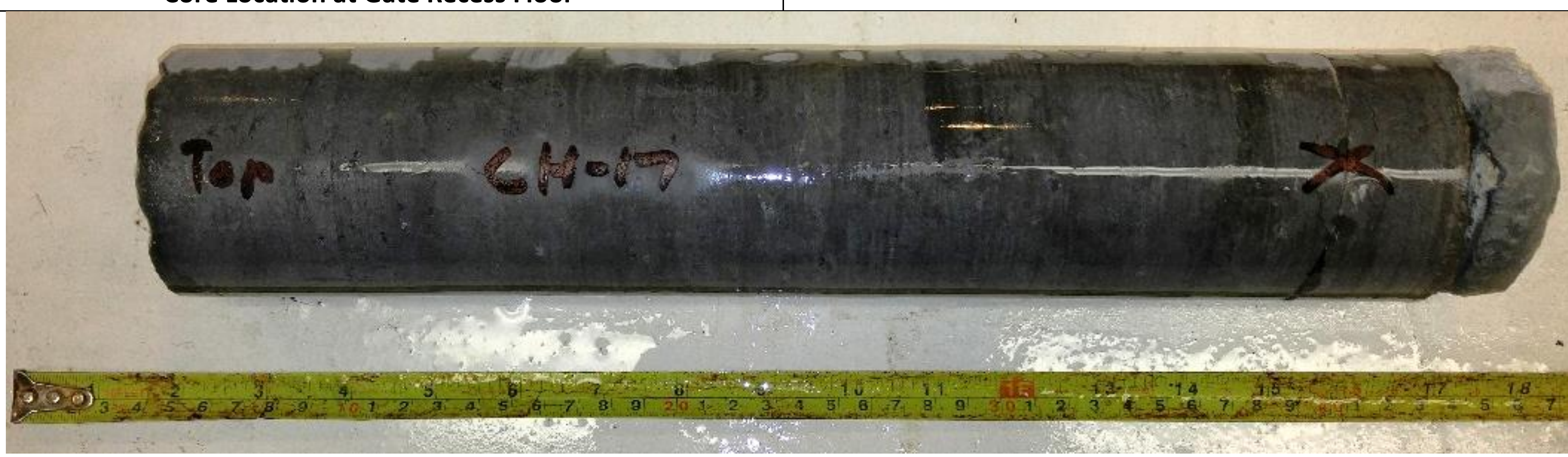
ID	CH 16
Date Cored	April 21, 2017
Lock	48
Location	Floor
Section	48 FLR
Direction Core Extracted	Vertical
Tested	No
Total Hole Depth	590 mm

General Notes:
Concrete 0 -240 mm
Granite 240 – 590 mm

Core Location at Gate Recess Floor

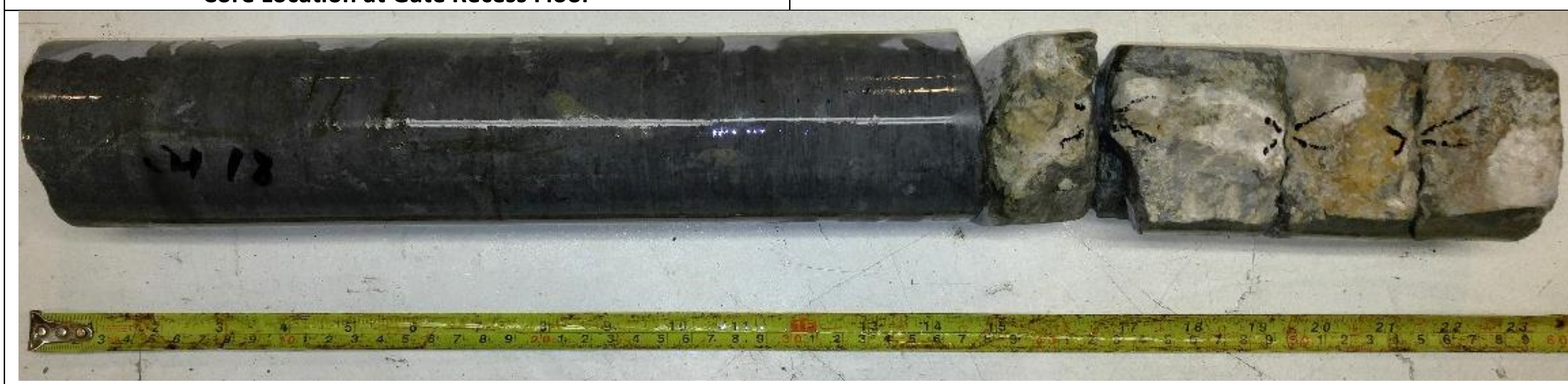
ID	CH 17
Date Cored	April 21, 2017
Lock	47
Location	Gate Recess Floor
Section	47-GRF3
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	370

General Notes:
Limestone 0 – 370 mm



Core Location at Gate Recess Floor

ID	CH 18
Date Cored	April 21, 2017
Lock	48
Location	Gate Recess Floor
Section	48-GRF2
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	540 mm
General Notes: Limestone 0 – 540 mm	



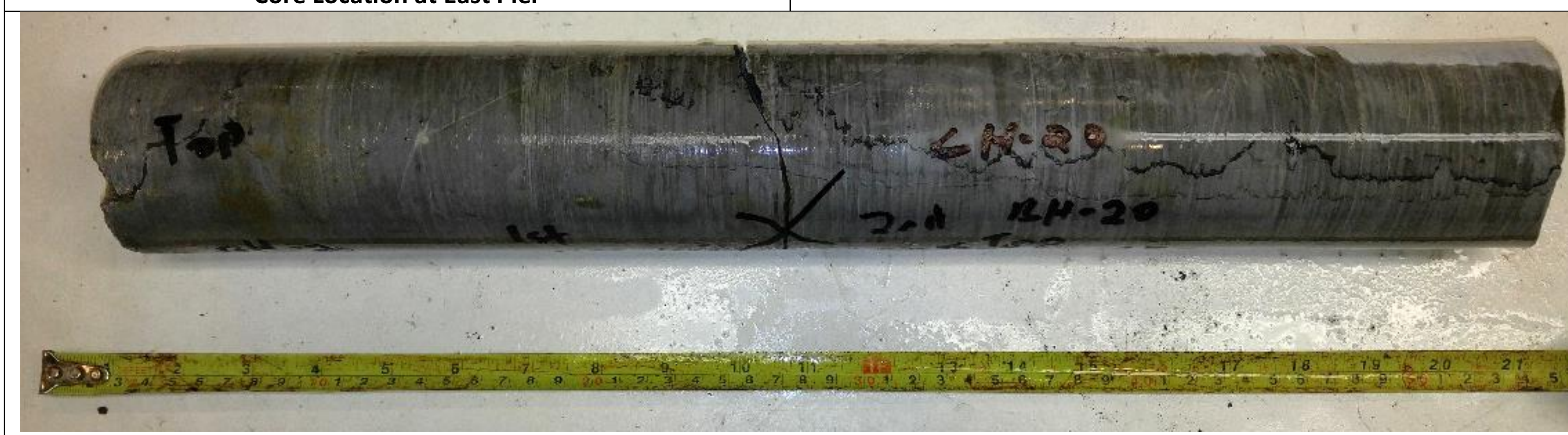
Core Location at East Chamber Wall

ID	CH 19
Date Cored	April 21, 2017
Lock	47
Location	Chamber Wall
Section	47E-CHW1
Direction Core Extracted	Horizontal
Tested	No
Total Hole Depth	440 mm
General Notes: Limestone 0 – 440 mm	

Core Location at East Pier

ID	CH 20
Date Cored	April 21, 2017
Lock	47
Location	Pier
Section	47E-PIE6
Direction Core Extracted	Horizontal
Tested	Yes
Total Hole Depth	480 mm

General Notes:
Limestone 0 – 480 mm

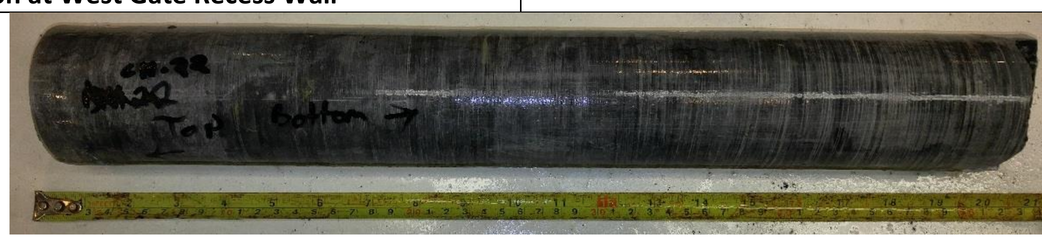


Core Location at East Chamber Wall

ID	CH 21
Date Cored	April 21, 2017
Lock	48
Location	Chamber Wall
Section	48E-CHW2
Direction Core Extracted	Horizontal
Tested	Yes
Total Hole Depth	470 mm
General Notes: Limestone 0 – 470 mm	

Core Location at West Gate Recess Wall

ID	CH 22
Date Cored	April 21, 2017
Lock	48
Location	Gate Recess Wall
Section	48E-GRW4
Direction Core Extracted	Horizontal
Tested	Yes
Total Hole Depth	480 mm
General Notes: Limestone 0 – 480 mm	



Core Location at West Chamber Wall

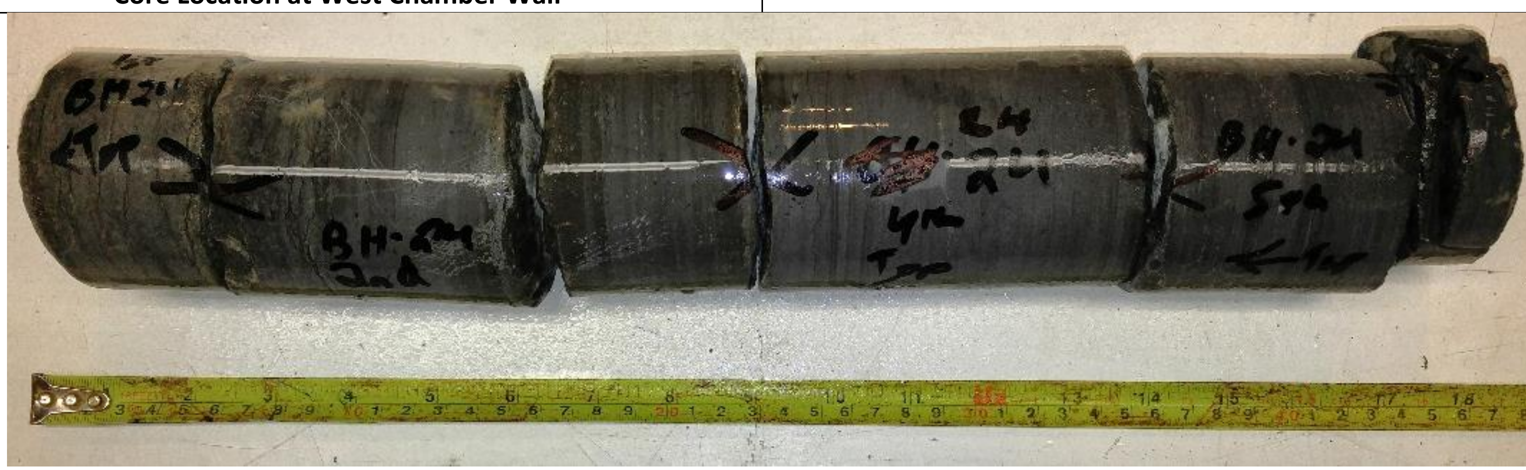
ID	CH 23
Date Cored	April 21, 2017
Lock	47
Location	Chamber Wall
Section	47W-CHW2
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	460 mm

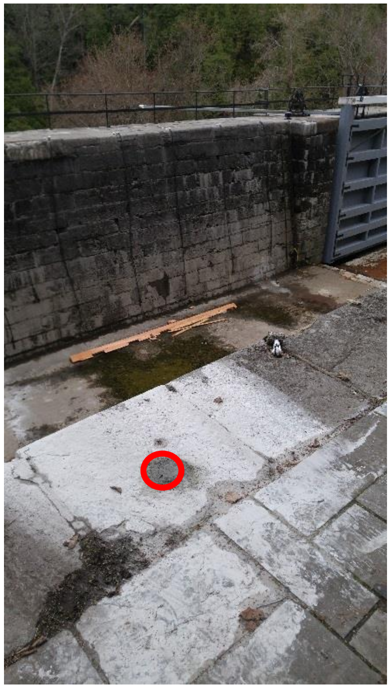
General Notes:
Limestone 0 – 420 mm
Mortar 420 – 460 mm

Core Location at West Chamber Wall

ID	CH 24
Date Cored	April 21, 2017
Lock	47
Location	Chamber Wall
Section	47W-CHW1
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	410 mm

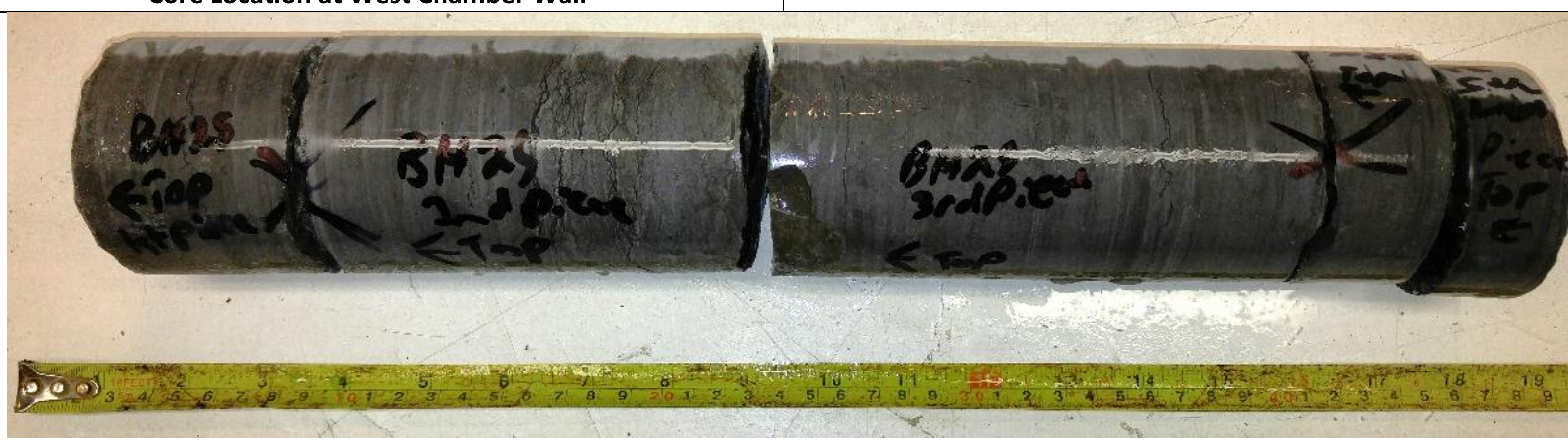
General Notes:
Limestone 0 – 410 mm





Core Location at West Chamber Wall

ID	CH 25
Date Cored	April 21, 2017
Lock	48
Location	Chamber Wall
Section	48W-CHW1
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	400 mm
General Notes: Limestone 0 – 400 mm	



Core Location at West Chamber Wall

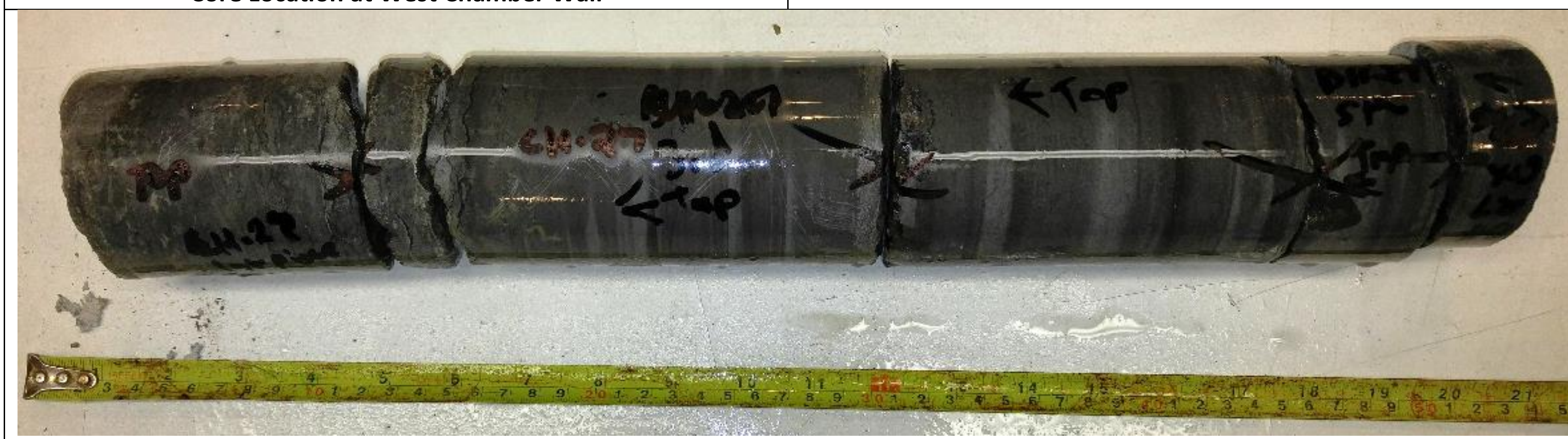
ID	CH 26
Date Cored	April 21, 2017
Lock	48
Location	Chamber Wall
Section	48W-CHW1
Direction Core Extracted	Vertical
Tested	No
Total Hole Depth	430 mm

General Notes:
Limestone 0 – 410 mm
Mortar/concrete 410 – 430 mm

Core Location at West Chamber Wall

ID	CH 27
Date Cored	April 21, 2017
Lock	48
Location	Chamber Wall
Section	48W-CHW1
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	500 mm

General Notes:
Limestone 0 – 500 mm



Core Location at West Chamber Wall

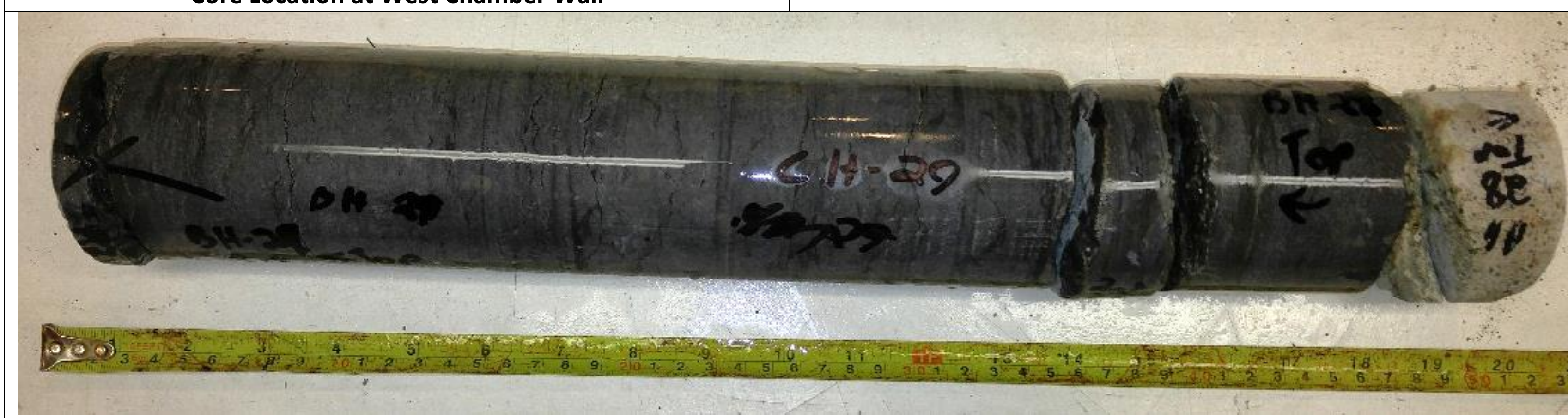
ID	CH 28
Date Cored	April 21, 2017
Lock	49
Location	Chamber Wall
Section	49W-CHW2
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	460mm

General Notes:
Limestone 0 – 400 mm
Mortar/concrete 400 – 460 mm

Core Location at West Chamber Wall

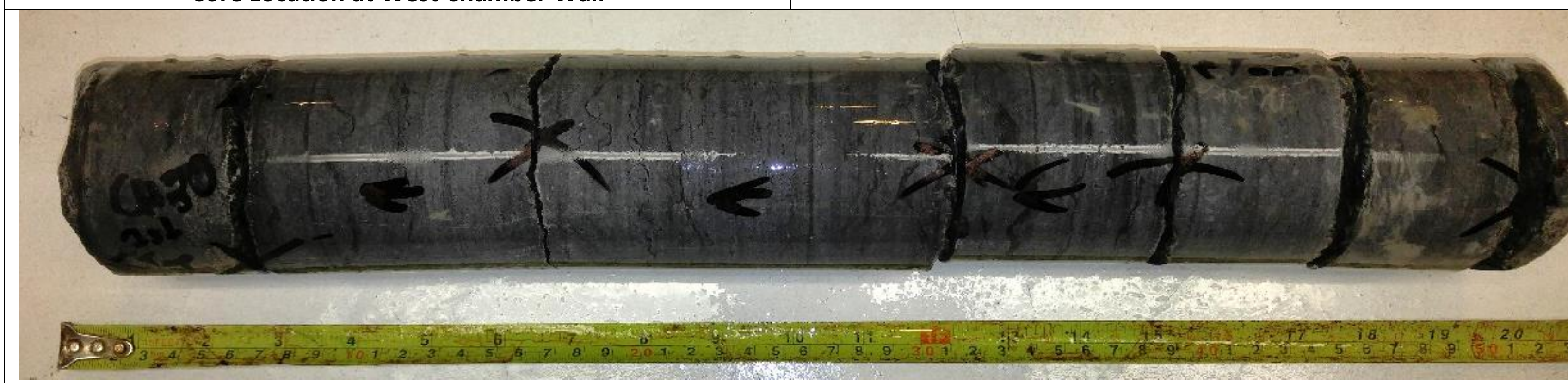
ID	CH 29
Date Cored	April 21, 2017
Lock	49
Location	Chamber Wall
Section	49W-CHW1
Direction Core Extracted	Vertical
Tested	No
Total Hole Depth	460 mm

General Notes:
Limestone 0 – 400 mm
Mortar 400 – 460 mm



Core Location at West Chamber Wall

ID	CH 30
Date Cored	April 21, 2017
Lock	49
Location	Chamber Wall
Section	49W-CHW1
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	460 mm
General Notes: Limestone 0 – 460 mm	



Core Location at East Chamber Wall

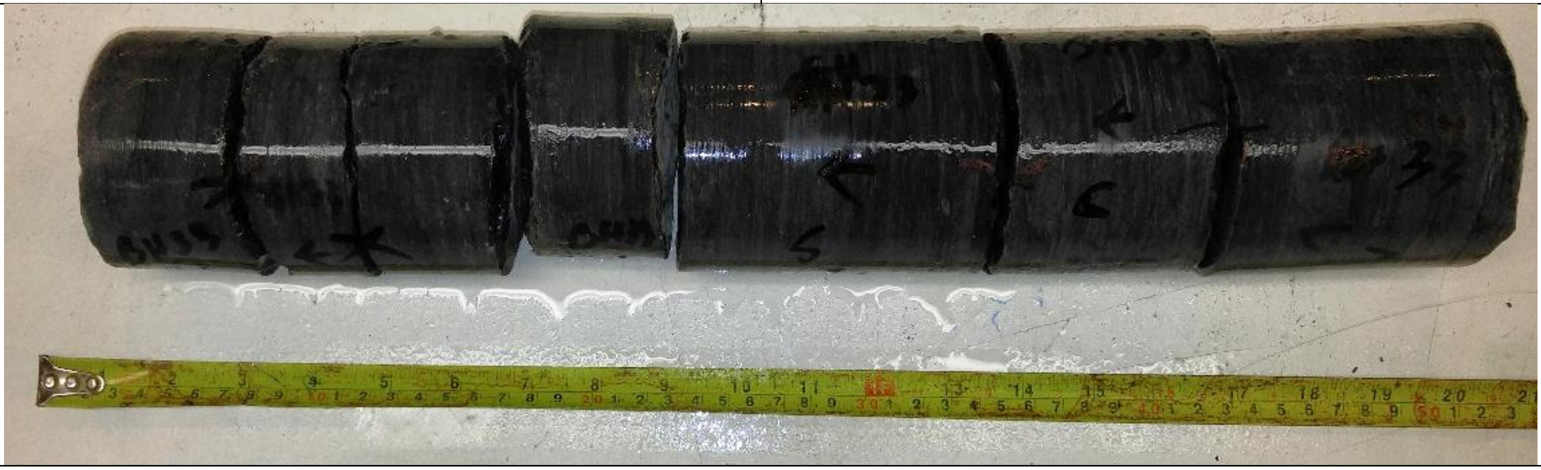
ID	CH 31
Date Cored	April 21, 2017
Lock	49
Location	Chamber Wall
Section	49E-CHW2
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	460 mm
General Notes: Limestone 0 – 460 mm	

Core Location at East Chamber Wall

ID	CH 32
Date Cored	April 21, 2017
Lock	47
Location	Chamber Wall
Section	47E-CHW2
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	430 mm
General Notes: Limestone 0 – 430 mm	

Core Location at East Chamber Wall

ID	CH 33
Date Cored	April 21, 2017
Lock	48
Location	Chamber Wall
Section	48E-CHW2
Direction Core Extracted	Vertical
Tested	Yes
Total Hole Depth	460 mm
General Notes: Limestone 0 – 460 mm	



Kingston Mills Lockstation
Rehabilitation of Locks 46 to 49
Results of Laboratory Testing

ID	Core Orientation	Percent Water Absorption (%)																							Comments
		C	Block 1	C	Block 2	C	Block 3	C	Block 4	C	Block 5	C	Block 6	C	Block 7	C	Block 8	C	Block 9	Min	Max	Avg	Std Dev	Classification	
CH 8	Horizontal	W	0.09	W	0.11	W	0.09	W	0.10	W	0.10	W	0.12	-	-	-	-	-	-	0.09	0.12	0.10	0.01	Type III	
CH 9	Horizontal	W	0.03	W	0.05	W	0.06	W	0.06	W	0.06	W	0.07	W	0.07	-	-	-	-	0.03	0.07	0.06	0.01	Type III	
CH 11	Horizontal	W	0.08	W	0.11	W	0.08	W	0.08	W	0.07	-	-	-	-	-	-	-	-	0.07	0.11	0.08	0.01	Type III	
CH 12	Horizontal	D	0.09	D	0.10		0.09	D	0.07	-	0.08	D	0.12	W	0.12	-	-	-	-	0.07	0.12	0.10	0.02	Type III	
CH 14	Horizontal	-	-	-	-	W	0.13	W	0.12	W	0.07	W	0.06	W	0.07	-	-	-	-	0.06	0.13	0.09	0.03	Type III	
CH 17	Vertical	D	0.09	D	0.08	D	0.06	D	0.06	D	0.08		0.12	-	-	-	-	-	-	0.06	0.12	0.08	0.02	Type III	
CH 18	Vertical	W	0.09	W	0.10	W	0.11	W	0.11	W	0.09	-	-	-	-	-	-	-	-	0.09	0.11	0.10	0.01	Type III	
CH 21	Horizontal	D	0.08	D	0.07	D	0.06	D	0.08	D	0.08	-	-	-	-	-	-	-	-	0.06	0.08	0.07	0.01	Type III	
CH 22	Horizontal	W	0.16	W	0.15	W	0.15	W	0.15	W	0.13	-	0.15	-	-	-	-	-	-	0.13	0.16	0.15	0.01	Type III	
CH 27	Vertical	W	0.14	W	0.11	W	0.11	W	0.04	W	0.09	-	-	-	-	-	-	-	-	0.04	0.14	0.10	0.03	Type III	
CH 31	Vertical	D	0.22	D	0.09	D	0.13	D	0.08	D	0.13	-	-	-	-	-	-	-	-	0.08	0.22	0.13	0.05	Type III	
																All Samples			Min	0.0	0.1	0.1		Type III	
																			Max	0.1	0.2	0.1			
																			Avg	0.1	0.1	0.1			

ID	Core Orientation	Bulk Specific Gravity (kg/m³)																							Comments
		C	Block 1	C	Block 2	C	Block 3	C	Block 4	C	Block 5	C	Block 6	C	Block 7	C	Block 8	C	Block 9	Min	Max	Avg	Std Dev	Classification	
CH 8	Horizontal	W	2730	W	2750	W	2740	W	2730	W	2730	W	2740	-	-	-	-	-	-	2730	2750	2737	7	Type III	
CH 9	Horizontal	W	2370	W	2370	W	2730	W	2710	W	2720	-	2730	-	2720	-	-	-	-	2370	2730	2621	159	Type III	
CH 11	Horizontal	W	2740	W	2710	W	2740	W	2730	W	2720	-	-	-	-	-	-	-	-	2710	2740	2728	12	Type III	
CH 12	Horizontal	D	2720	D	2720	-	2710	D	2730	-	2710	D	2720	W	2720	-	-	-	-	2710	2730	2719	6	Type III	
CH 14	Horizontal	-	-	-	-	W	2720	W	2720	W	2730	W	2860	W	2720	-	-	-	-	2720	2860	2750	55	Type III	
CH 17	Vertical	D	2740	D	2720	D	2710	D	2720	D	2710		2710	-	-	-	-	-	-	2710	2740	2718	11	Type III	
CH 18	Vertical	W	2720	W	2720	W	2720	W	2730	W	2740	-	-	-	-	-	-	-	-	2720	2740	2726	8	Type III	
CH 21	Horizontal	D	2720	D	2720	D	2720	D	2720	D	2720	-	-	-	-	-	-	-	-	2720	2720	2720	0	Type III	
CH 22	Horizontal	W	2730	W	2730	W	2720	W	2730	W	2720	-	2740	-	-	-	-	-	-	2720	2740	2728	7	Type III	
CH 27	Vertical	W	2730	W	2750	W	2770	W	2730	W	2720	-	-	-	-	-	-	-	-	2720	2770	2740	18	Type III	
CH 31	Vertical	D	2710	D	2730	D	2720	D	2720	D	2740	-	-	-	-	-	-	-	-	2710	2740	2724	10	Type III	
																All Samples			Min	2370	2720	2621		Type III	
																			Max	2730	2860	2750			
																			Avg	2685	2751	2719			

Note: C Test Condition (W=Wet, D = Dry)

Kingston Mills Lockstation
Rehabilitation of Locks 46 to 49
Results of Laboratory Testing

ID	Core Orientation	Compressive Strength (MPa)														
		C	T	Block 1	C	T	Block 2	C	T	Block 3	C	T	Block 4	C	T	Block 5
CH 8	Horizontal	W	**	146	W	**	123	W	**	112	W	**	121	W	**	99
CH 9	Horizontal	W	**	118	W	**	157	W	**	136	W	**	180	W	**	137
CH 11	Horizontal	W	**	97	W	**	19	W	*	42	W	**	88	W	**	66
CH 12	Horizontal	D	**	113	D	**	192	-	-	-	D	**	167	-	-	-
CH 14	Horizontal	-	-	-	-	-	-	W	**	7	W	**	135	W	**	228
CH 17	Vertical	D	*	38	D	**	149	D	**	140	D	**	75	D	*	149
CH 18	Vertical	W	**	144	W	**	166	W	*	135	W	*	115	W	*	45
CH 21	Horizontal	D	**	154	D	**	136	D	**	121	D	**	103	D	*	18
CH 22	Horizontal	W	**	155	W	**	135	W	**	128	W	**	92	W	**	110
CH 27	Vertical	W	**	98	W	*	31	W	*	47	W	*	35	W	**	90
CH 31	Vertical	D	**	133	D	**	95	D	*	9	D	*	51	D	*	42

ID	Compressive Strength (MPa)																	Comments
	C	T	Block 6	C	T	Block 7	C	T	Block 8	C	T	Block 9	Min	Max	Avg	Std Dev	Classification	
CH 8	-	-	-	-	-	-	-	-	-	-	-	-	99	146	120	15	Type III	
CH 9	-	-	-	-	-	-	-	-	-	-	-	-	118	180	146	21	Type III	
CH 11	-	-	-	-	-	-	-	-	-	-	-	-	19	97	62	29	Type III	
CH 12	D	**	114	W	**	189	-	-	-	-	-	-	113	192	155	35	Type III	
CH 14	W	**	213	W	*	216	-	-	-	-	-	-	7	228	160	83	Type III	
CH 17	-	-	-	-	-	-	-	-	-	-	-	-	38	149	110	45	Type III	
CH 18	-	-	-	-	-	-	-	-	-	-	-	-	45	166	121	41	Type III	
CH 21	-	-	-	-	-	-	-	-	-	-	-	-	18	154	106	47	Type III	
CH 22	-	-	-	-	-	-	-	-	-	-	-	-	92	155	124	22	Type III	
CH 27	-	-	-	-	-	-	-	-	-	-	-	-	31	98	60	28	Type III	
CH 31	-	-	-	-	-	-	-	-	-	-	-	-	9	133	66	43	Type III	

All Samples		Min	Max	Avg	Std Dev	Classification
	Min	7	97	60		Type III
	Max	118	228	160		
	Avg	54	154	112		

Note:
- C Test Condition (W=Wet, D = Dry)
- T Test Orientation
- * Sample Tested Parallel to Core Axis
- ** Sample Tested Perpendicular to Core Axis

ASTM C170 / C170M-17 includes testing parallel and perpendicular to the rift of the stone. The rift could not be identified in the sandstone samples. Testing was therefore completed both parallel and perpendicular to the core axis.

APPENDIX

C-2 BOREHOLE AND TEST PIT INVESTIGATION

LOG OF BOREHOLE 17-1

Project: Kingston Mills Lockstation Rehabilitation
Client: Parks Canada
Project Location: Kingston Mills, Ontario
Datum: Geodetic
BH Location: N 4905453 E 384954

DRILLING DATA
Rig Type: Portable
Method: Solid Stem Auger
Borehole Diameter: 150 mm
Core Diameter:

Project No.: 171-012359-00
Date Started: 10/4/2017
Supervisor: D.R
Reviewer: C.H.

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	WATER CONTENT	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.5								
90.4 / 0.1	**TOPSOIL** (70 mm)							
	SILT with clay, some sand, trace gravel, brown, moist, compact to dense (FILL)	1	SS	10	Backfill	90	o	
		2	SS	10	Bentonite		o	
		3	SS	5	Sand; W. L. 88.5 m Oct 12, 2017	89	o	
		4	SS	6	Screen	88	o	9 14 56 21
87.5 / 3.1	**SILTY SAND** trace gravel and clay, some organics, grey, wet, dense (FILL)	5	SS	100 over 25 mm			o	
87.0 / 86.9 / 3.6	**BEDROCK** fresh, close jointing, thin bedding TCR - 100% SCR - 100% RQD - 80%	1	CORE		Bentonite			

END OF BOREHOLE

Notes:

1) Borehole terminated at 3.6 m below the existing surface elevation
2) 50 mm monitoring well installed at 3.45 m below the existing ground surface.
3) Date / Groundwater Depth

Date	Groundwater Depth
10/4/2017	3.58 m
10/12/2017	2.17 m
11/30/2017	2.01 m

Column headers: SOIL PROFILE; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE (25 50 75 100 125); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25 50 75); POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (KN/m³).

WSP SOIL LOG - OTTAWA KINGSTON MILLS.GPJ SPL.GDT 4/26/18

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

LOG OF BOREHOLE 17-2

Project: Kingston Mills Lockstation Rehabilitation
Client: Parks Canada
Project Location: Kingston Mills, Ontario
Datum: Geodetic
BH Location: N 4905454 E 384975

DRILLING DATA
Rig Type:CME 55
Method: Solid Stem Auger
Borehole Diameter: 150 mm
Core Diameter:

Project No.: 171-012359-00
Date Started: 10/6/2017
Supervisor: D.R
Reviewer: C.H.

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.5								
90.4 / 0.1	**TOPSOIL** (90 mm)							
	SILTY SAND trace gravel and clay, brown, moist, soft to stiff (FILL)	1	SS	9	Bentonite	90	o	
		2	SS	3	Sand		o	
						89		
		3	SS	4	Screen		o	9 47 37 7
					W. L. 88.3 m Oct 12, 2017			
88.1		4	SS	100 over 25 mm			o	
2.4	**GRANITE** slightly weathered to fresh, thin to medium bedding, close jointing					88		
	TCR - 100%	1	CORE		Bentonite			
	SCR - 42%					87		
	RQD - 27%							
86.5	Concrete/Masonry observed							
4.0	**END OF BOREHOLE**							

Notes:

1) Borehole terminated at 3.96 m below the existing surface elevation
2) 50 mm monitoring well installed at 2.43 m below the existing ground surface.
3) Date / Groundwater Depth

Date	Groundwater Depth
10/6/2017	2.77 m
10/12/2017	2.17 m
11/30/2017	2.31 m

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE & Sensitivity, × LAB VANE: 25 50 75 100 125
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
POCKET PEN. (Cu) (kPa) — NATURAL UNIT WT (KN/m^3)

WSP SOIL LOG - OTTAWA KINGSTON MILLS.GPJ SPL.GDT 4/26/18

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

LOG OF BOREHOLE 17-3

Project: Kingston Mills Lockstation Rehabilitation	**DRILLING DATA**
Client: Parks Canada	Rig Type: CME 55 — Project No.: 171-012359-00
Project Location: Kingston Mills, Ontario	Method: Solid Stem Auger — Date Started: 10/4/2017
Datum: Geodetic	Borehole Diameter: 150 mm — Supervisor: D.R
BH Location: N 4905334 E 384932	Core Diameter: — Reviewer: C.H.

ELEV (m) / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	WATER CONTENT	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
86.1								
86.0 / 0.1	**TOPSOIL** (80 mm)					86		
	SILT with sand, trace clay and gravel, brown, moist, loose to compact (FILL)	1	SS	5	Bentonite		o	
		2	SS	4		85	o	
	occasional cobbles	3	SS	100 over 50 mm	Screen		o	6 24 60 9
					W. L. 84.1 m Oct 12, 2017			
83.8 / 2.3	**GRANITE** Fresh, Moderately close jointing, Medium to thin bedding	4	SS	100 over 0 mm				
	TCR - 100% SCR - 89% RQD - 94%	1	CORE		Bentonite			
	GRANITE Fresh, Moderately close jointing, Medium to thin bedding	2	CORE					
82.2 / 3.9	TCR - 100% SCR - 92% RQD - 75%							
	END OF BOREHOLE							

Column headings also include: STRATA PLOT; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) (○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (kN/m^3).

Notes:

1) Borehole terminated at 3.88 m below the existing surface elevation
2) 50 mm monitoring well installed at 1.77 m below the existing ground surface.
3)

Date	Groundwater Depth
10/5/2017	3.08 m
10/12/2017	1.98 m
11/30/2017	1.98 m

WSP SOIL LOG - OTTAWA KINGSTON MILLS.GPJ SPL.GDT 4/26/18

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

LOG OF BOREHOLE HA17-4

Project: Kingston Mills Lockstation Rehabilitation	**DRILLING DATA**	
Client: Parks Canada	Rig Type:Hand Dug	Project No.: 171-012359-00
Project Location: Kingston Mills, Ontario	Method: Solid Stem Auger	Date Started: 10/6/2017
Datum: Geodetic	Borehole Diameter: 150 mm	Supervisor: D.R
BH Location: N 4905285 E 384907	Core Diameter:	Reviewer: C.H.

(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE (25 50 75 100 125)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.6												
82.6 / 0.0	**SILTY SAND AND GRAVEL** some organics, brown, moist, loose (160 mm) (FILL)	⊠	1	GRAB					○			
0.2	**END OF TEST PIT**											

Notes:

1) Hand dug test pit terminated at 160 mm below the existing surface elevation on inferred boulder

WSP SOIL LOG - OTTAWA KINGSTON MILLS.GPJ SPL.GDT 4/26/18

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

LOG OF BOREHOLE HA17-5

Project: Kingston Mills Lockstation Rehabilitation
Client: Parks Canada
Project Location: Kingston Mills, Ontario
Datum: Geodetic
BH Location: N 4905251 E 384887

DRILLING DATA
Rig Type:Hand Dug
Method: Solid Stem Auger
Borehole Diameter: 150 mm
Core Diameter:

Project No.: 171-012359-00
Date Started: 10/6/2017
Supervisor: D.R
Reviewer: C.H.

(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE & Sensitivity ● QUICK TRIAXIAL × LAB VANE 25 50 75 100 125	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 25 50 75	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
78.8												
78.8 0.1	**SILTY SAND AND GRAVEL** some organics, brown, moist, loose (100 mm) (FILL)		1	GRAB					○			
	END OF TEST PIT											
	Notes:											
	1) Hand dug test pit terminated at 100 mm below the existing surface elevation on inferred boulder											

GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

Sheet No. 1 of 1

LOG OF BOREHOLE 17-6

Project: Kingston Mills Lockstation Rehabilitation
Client: Parks Canada
Project Location: Kingston Mills, Ontario
Datum: Geodetic
BH Location: N 4905251 E 384897

DRILLING DATA
Rig Type: CME 55
Method: Solid Stem Auger
Borehole Diameter: 150 mm
Core Diameter:

Project No.: 171-012359-00
Date Started: 10/5/2017
Supervisor: D.R
Reviewer: C.H.

ELEV (m) / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	Water content	GR	SA	SI	CL
78.8										
78.7 / 0.1	**TOPSOIL** (100mm)									
	SAND AND SILT trace gravel and clay, grey brown, moist to wet, loose (wood fragement at 5.79m) (FILL)	1	SS	13	Backfill	o				
		2	SS	9		o				
		3	SS	7		o (with limits)				
		4	SS	9		o	9	44	41	6
		5	SS	5		o				
75.0 / 3.8	**SILT** with sand, some clay, trace gravel, brown, wet, loose to very dense (FILL)	6	SS	3	W. L. 75.0 m Oct 12, 2017	o	2	22	58	18
		7	SS	12	Bentonite / Sand	o				
		8	SS	21	Screen	o				
72.7 / 6.1	**SILTY SAND** trace gravel and clay, grey wet, loose to dense	9	SS	10		o (with limits)				
		10	SS	4	Bentonite	o				
70.6 / 8.2	**END OF BOREHOLE**									

Elevation scale: 78, 77, 76, 75, 74, 73, 72, 71

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100
Shear Strength (kPa): ○ Unconfined, + Field Vane & Sensitivity, ● Quick Triaxial, × Lab Vane; 25 50 75 100 125
Plastic Limit W_P, Natural Moisture Content W, Liquid Limit W_L; Water Content (%): 25 50 75
Pocket Pen. (Cu) (kPa); Natural Unit Wt (kN/m³); Remarks and Grain Size Distribution (%)

Notes:
1) Borehole terminated at 8.22 m below the existing surface elevation
2) 50 mm monitoring well installed at 6.7 m below the existing ground surface.
3)

Date	Groundwater Depth
10/5/2017	6.58 m
10/12/2017	3.84 m
11/30/2017	3.74 m

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▼ ▼

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

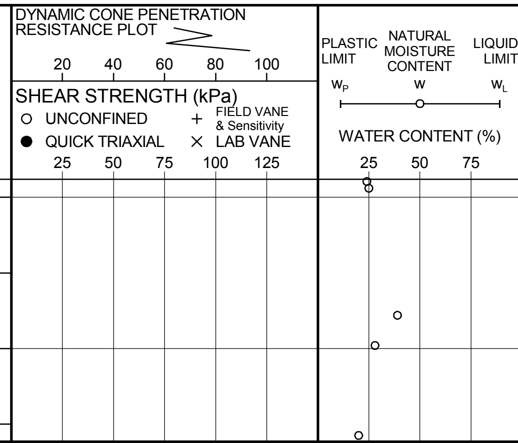
LOG OF BOREHOLE TP17-1

Project: Kingston Mills Lockstation Rehabilitation
Client: Parks Canada
Project Location: Kingston Mills, Ontario
Datum: Geodetic
BH Location: N 4905328 E 384930

DRILLING DATA
Rig Type: Backhoe
Method: Rubber Tire Backhoe
Borehole Diameter: 150 mm
Core Diameter:

Project No.: 171-012359-00
Date Started: 10/18/2017
Supervisor: D.R
Reviewer: C.H.

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
86.1 / 0.0	**TOPSOIL** (70mm)	2	GRAB			
86.0 / 0.1	**SILT** with clay some sand trace gravel, brown, moist	1	GRAB			
		5	GRAB			
		3	GRAB			
84.4 / 1.7		4	GRAB			
	END OF TESTPIT					



Notes:

1) Borehole terminated at 1.74 m below the existing surface elevation
2) Bedrock encountered at 1.74 m below surface

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation

GRAPH NOTES: + 3, × 3: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

Sheet No. 1 of 1

WSP SOIL LOG - OTTAWA KINGSTON MILLS.GPJ SPL.GDT 4/26/18

LOG OF BOREHOLE TP17-2

Project: Kingston Mills Lockstation Rehabilitation
Client: Parks Canada
Project Location: Kingston Mills, Ontario
Datum: Geodetic
BH Location: N 4905266 E 384916

DRILLING DATA
Rig Type: Backhoe
Method: Rubber Tire Backhoe
Borehole Diameter: 150 mm
Core Diameter:

Project No.: 171-012359-00
Date Started: 10/18/2017
Supervisor: D.R
Reviewer: C.H.

ELEV (m) / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	WATER CONTENT (%)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
78.7								
78.9 / 0.0	**TOPSOIL** (170 mm)							
0.2	**SILTY SAND** some clay, some cobble, brown, moist	1	GRAB				o	
78.2 / 78.1 / 0.5	**SAND** trace silt and gravel, brown, moist	2	GRAB				o	2 89 (9)
0.6	**SANDY SILT** some clay, trace gravel, some sand, frequent cobbles and boulders, brown, moist	3	GRAB			78	o	3 33 53 12
77.6 / 1.1	**END OF TESTPIT**							

Notes:

1) Testpit terminated at 1.1 m below the existing surface elevation
2) bedrock encountered at 1.1 m below surface

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75
POCKET PEN. (Cu) (kPa); NATURAL UNIT WT (KN/m³)

GROUNDWATER ELEVATIONS
Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

LOG OF BOREHOLE TP17-3

Project: Kingston Mills Lockstation Rehabilitation
Client: Parks Canada
Project Location: Kingston Mills, Ontario
Datum: Geodetic
BH Location: N 4905249 E 384901

DRILLING DATA
Rig Type:Backhoe
Method: Rubber Tire Backhoe
Borehole Diameter: 150 mm
Core Diameter:

Project No.: 171-012359-00
Date Started: 10/18/2017
Supervisor: D.R
Reviewer: C.H.

ELEV (m) / DEPTH	DESCRIPTION	STRATA PLOT	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (KN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
78.8 / 0.0	**TOPSOIL** (160 mm)		1	GRAB					o			
78.6 / 0.2	**SANDY GRAVEL** some silt, brown, moist		2	GRAB			78		o			
			3	GRAB					o			48 34 (18)
77.2 / 1.6	**END OF TEST PIT**											

Notes:

1) Borehole terminated at 1.6 m below the existing surface elevation on bedrock

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE & Sensitivity, ● QUICK TRIAXIAL, × LAB VANE; 25 50 75 100 125. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75.

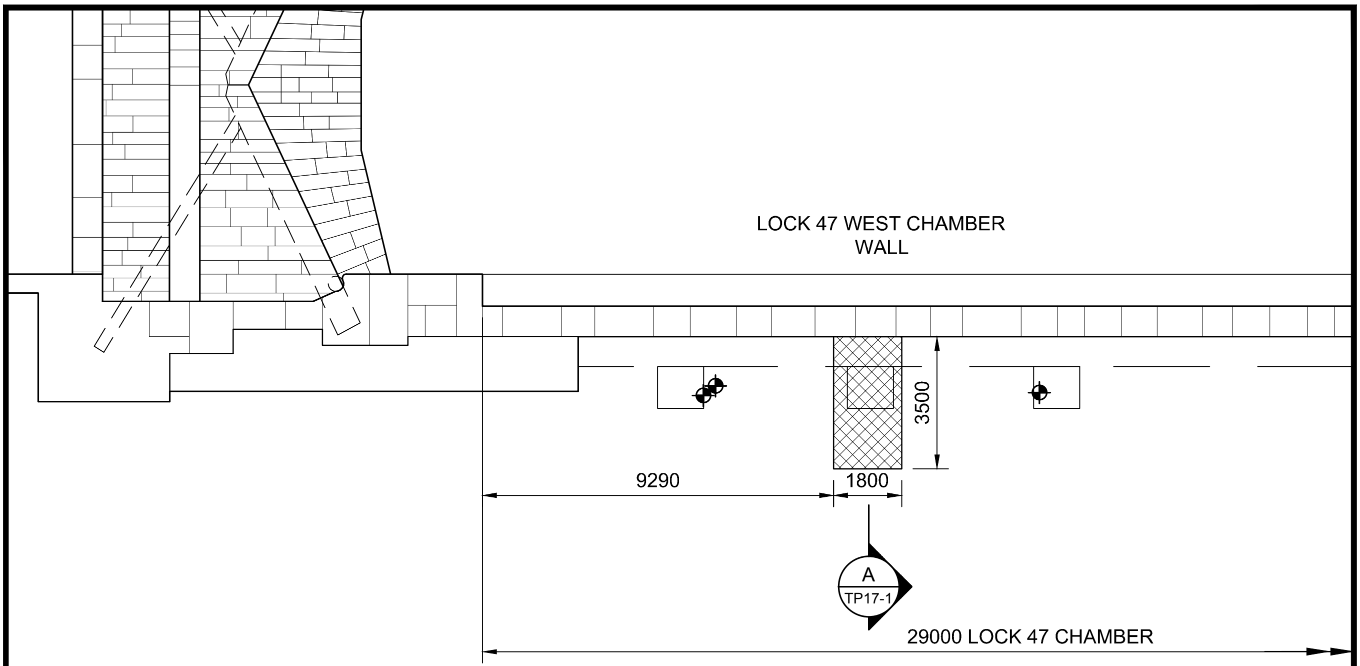
GROUNDWATER ELEVATIONS

Shallow/ Single Installation ▽ ▼ Deep/Dual Installation ▽ ▼

GRAPH NOTES: $+^3, \times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

APPENDIX

C-3 *TEST PIT SKETCHES AND PHOTOS*



PLAN

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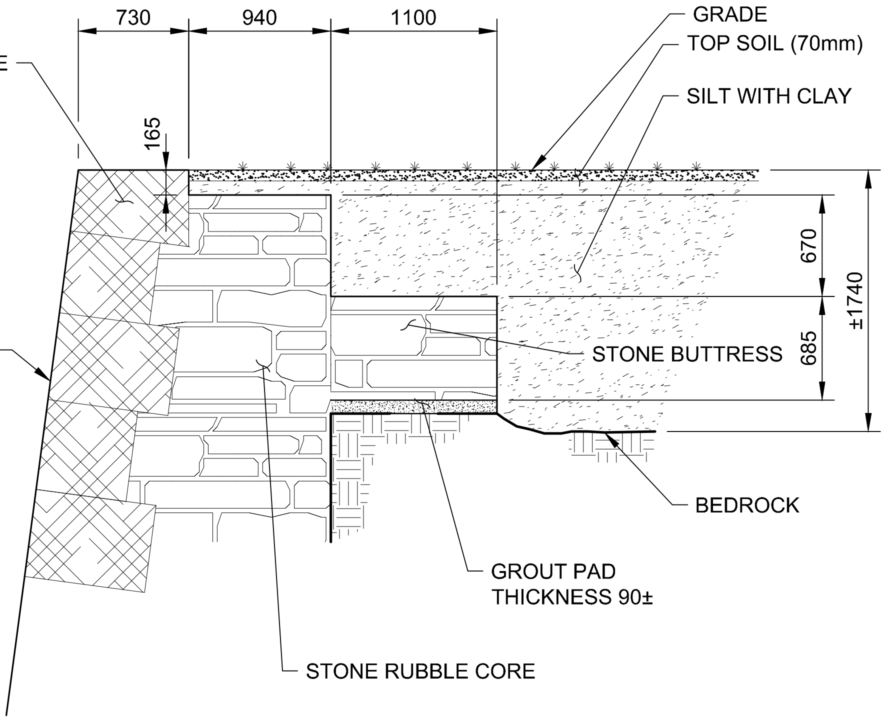


SECTION A TP17-1

1:50

wsp
300-2611 QUEENSVIEW DRIVE
OTTAWA (ONTARIO)
CANADA K2B 8K2
TELEPHONE: 613-829-2800 FAX: 613-829-8299
WWW.WSPGROUP.COM

TITLE:	KINGSTON MILLS LOCK STATION TEST PIT 17-1
SCALE:	AS SHOWN
DATE:	7-Feb-2018
PROJECT NO:	171-02359-00
REVISION:	
DRAWING NO:	TP17-1

WSP-A4Vrt PLOTTED BY: ADRIAN.MEUNIER DATE PLOTTED: Feb 12, 2018 FILE NAME: 500 Test Pits 01.dwg

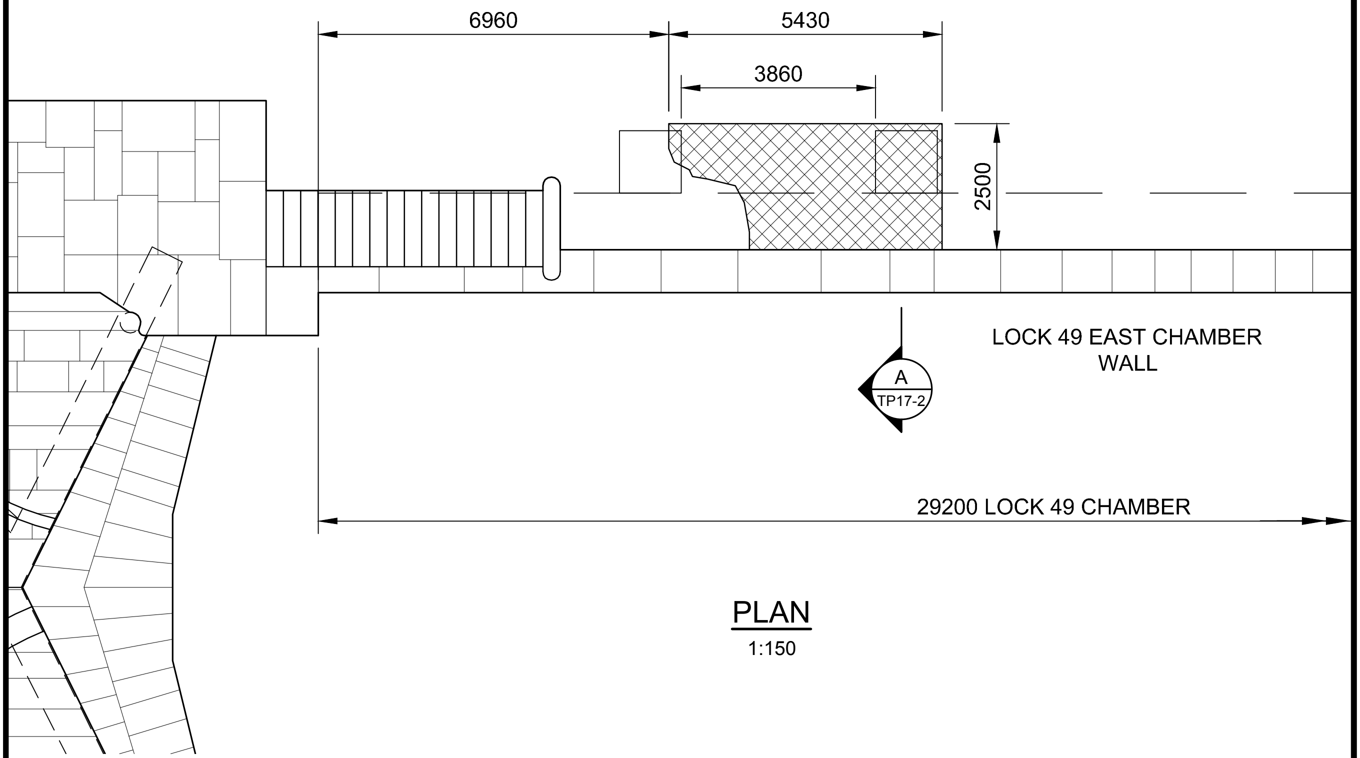
PHOTOS – TEST PIT #1

Test Pit #1 – Top of wall and buttress

Test Pit #1 – Top of wall and buttress

PHOTOS – TEST PIT #1

Test pit #1 – Layer of grout at bedrock



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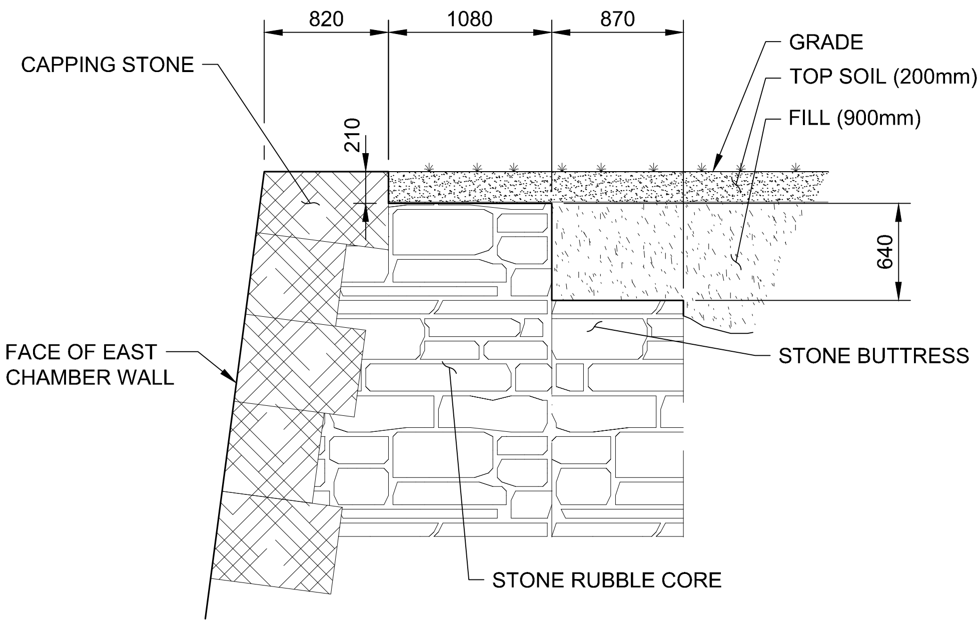


SECTION A TP17-2
1:50

WSP
300-2611 QUEENSVIEW DRIVE
OTTAWA (ONTARIO)
CANADA K2B 8K2
TELEPHONE: 613-829-2800 FAX: 613-829-8299
WWW.WSPGROUP.COM

TITLE: KINGSTON MILLS LOCK STATION
TEST PIT 17-2

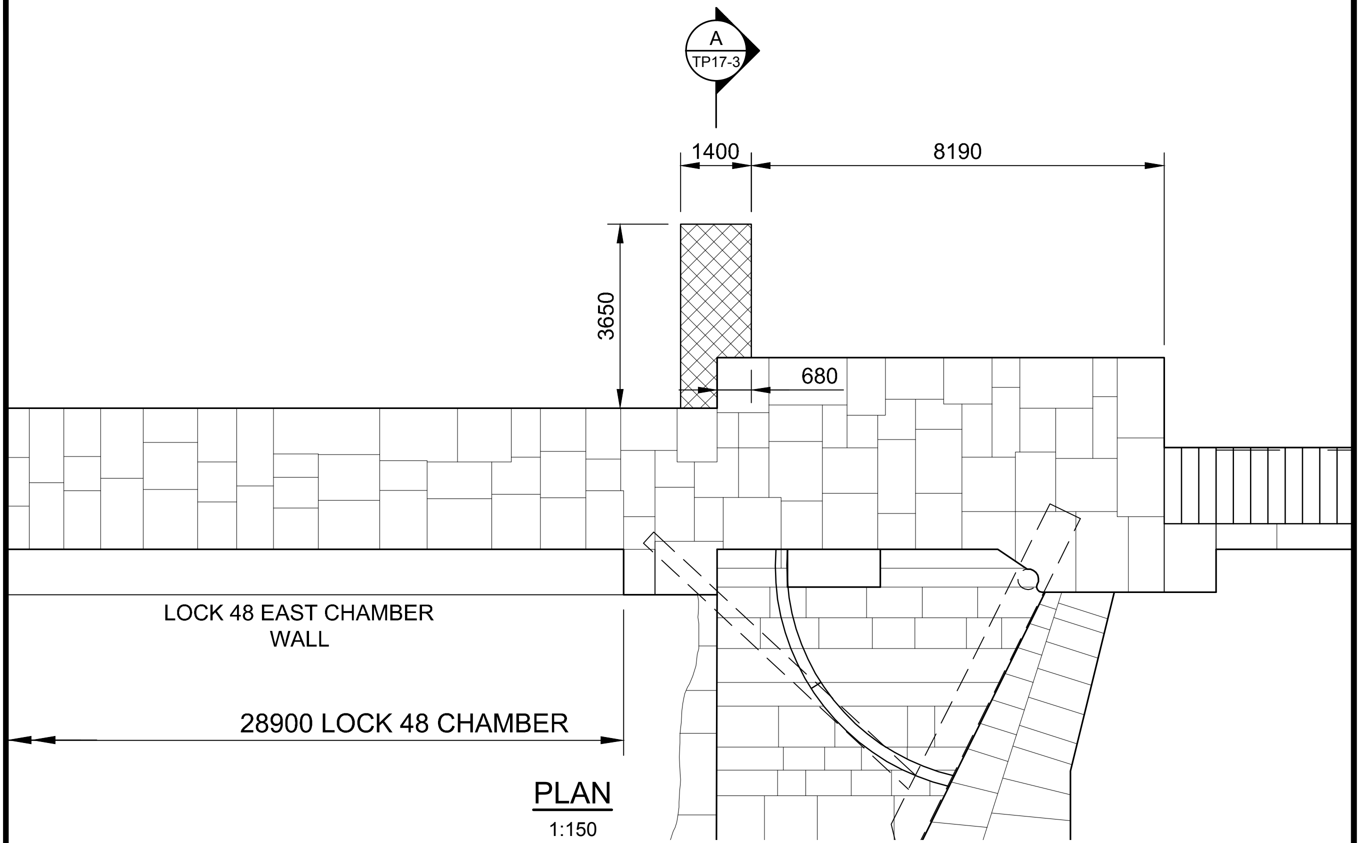
SCALE: AS SHOWN
DATE: 7-Feb-2018
PROJECT NO: 171-02359-00
REVISION:
DRAWING NO: TP17-2

WSP-A4Vrt PLOTTED BY: ADRIAN.MEUNIER DATE PLOTTED: Feb 12, 2018 FILE NAME: 500 Test Pits 01.dwg

PHOTOS – TEST PIT #2

Test Pit #2 – Buttress and back of chamber wall

Test Pit #2 – Buttress and back of chamber wall



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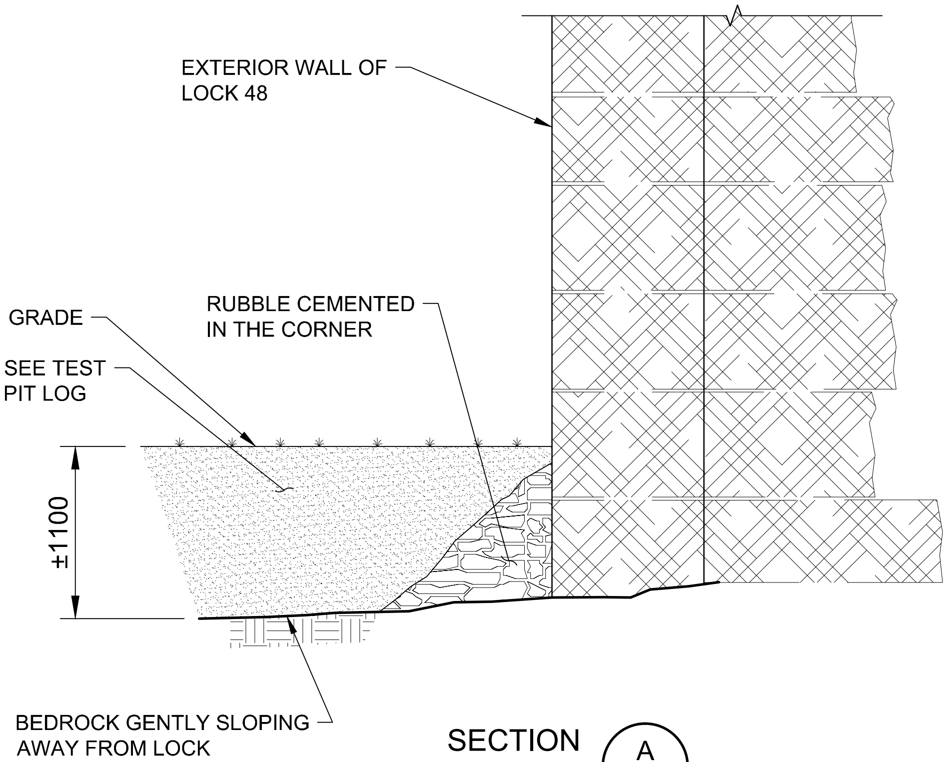
1:150



SECTION A TP17-3
1:50

wsp
300–2611 QUEENSVIEW DRIVE
OTTAWA (ONTARIO)
CANADA K2B 8K2
TELEPHONE: 613–829–2800 FAX: 613–829–8299
WWW.WSPGROUP.COM

TITLE:	KINGSTON MILLS LOCK STATION TEST PIT 17–3
SCALE:	AS SHOWN
DATE:	7–Feb–2018
PROJECT NO:	171–02359–00
REVISION:	
DRAWING NO:	TP17-3

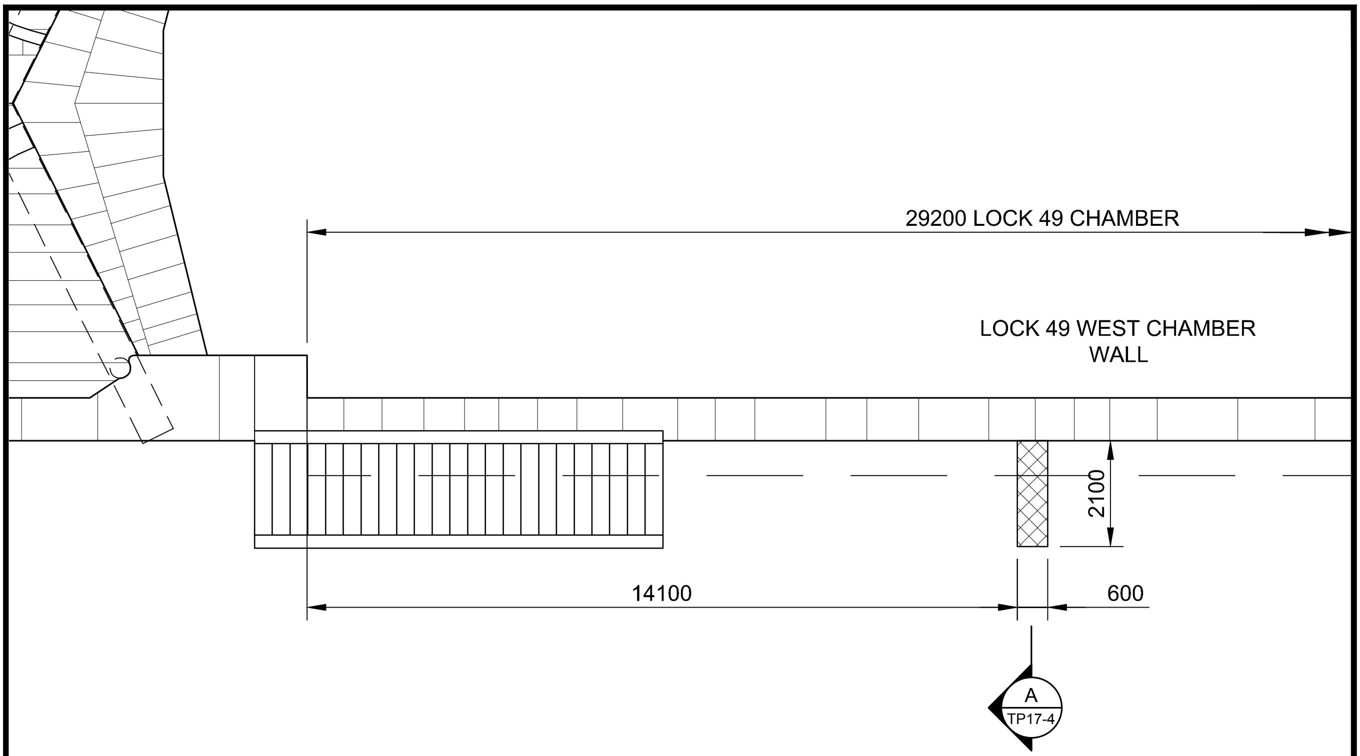
WSP–A4Vrt PLOTTED BY: ADRIAN.MEUNIER DATE PLOTTED: Feb 12, 2018 FILE NAME: 500 Test Pits 01.dwg

PHOTOS – TEST PIT #3

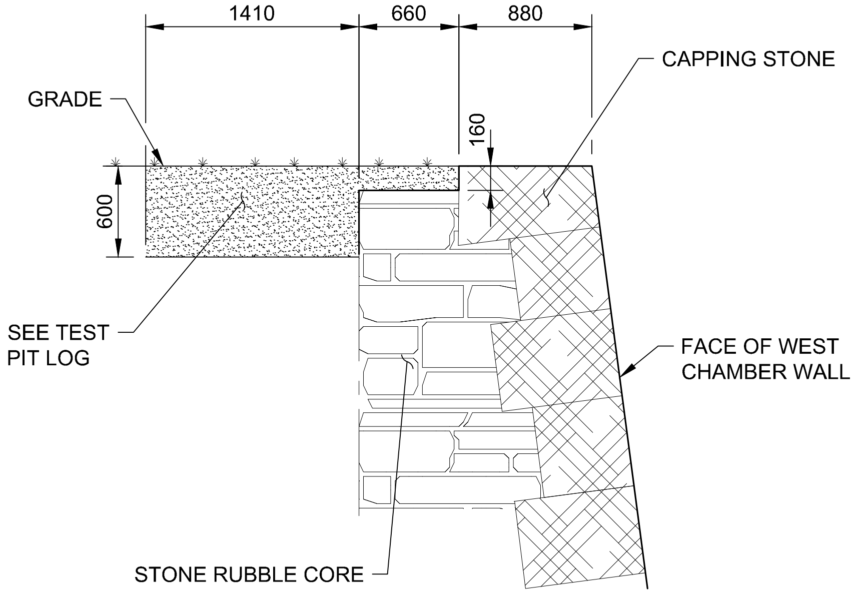
Test Pit #3 – Grouted rubble at base of chamber wall

PHOTOS – TEST PIT #3

Test Pit #3 – Monolith #4 and back of lock chamber wall



PLAN
1:150



SECTION A TP17-4
1:50

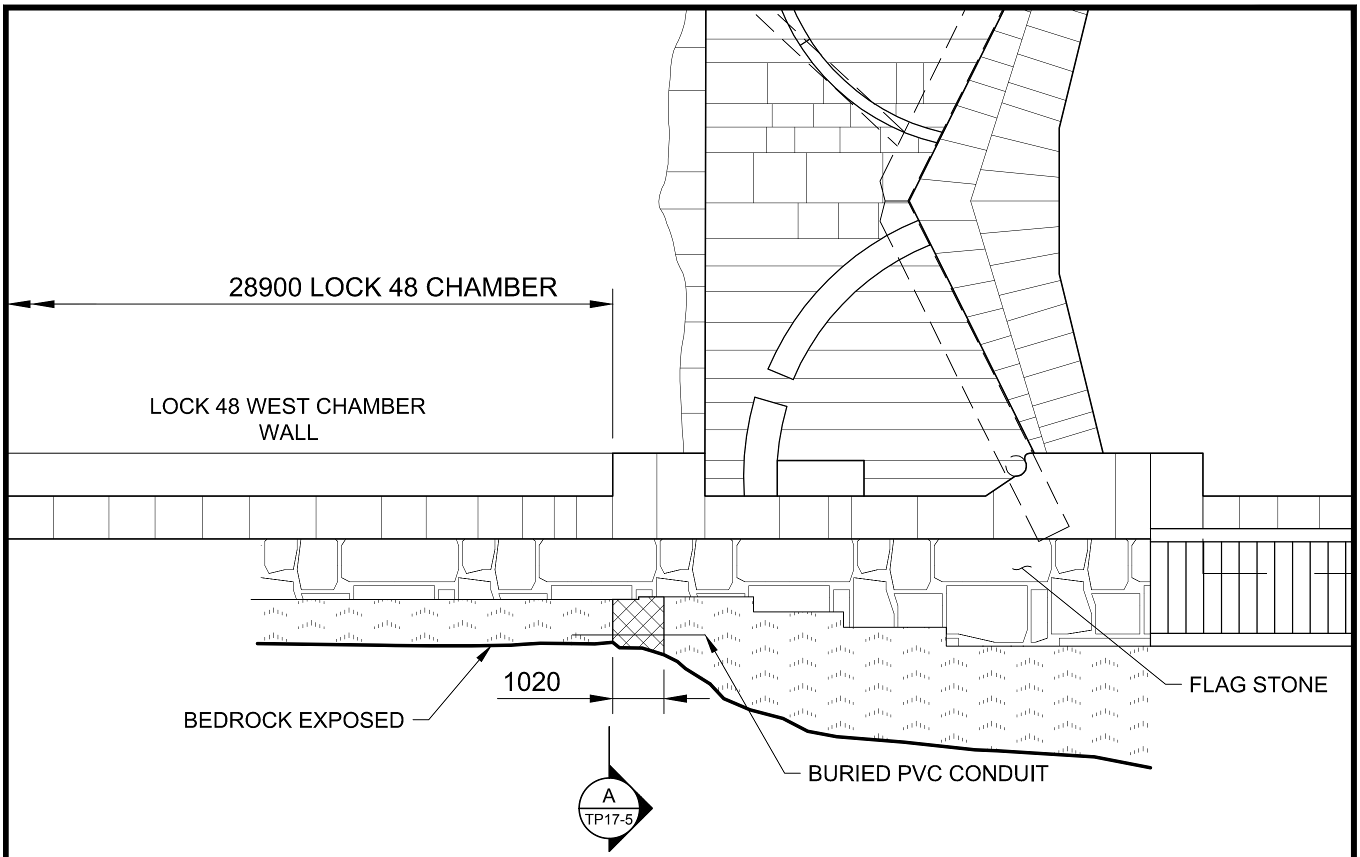
WSP
300-2611 QUEENSVIEW DRIVE
OTTAWA (ONTARIO)
CANADA K2B 8K2
TELEPHONE: 613-829-2800 FAX: 613-829-8299
WWW.WSPGROUP.COM

TITLE:	KINGSTON MILLS LOCK STATION TEST PIT 17-4
SCALE:	AS SHOWN
DATE:	7-Feb-2018
PROJECT NO:	171-02359-00
REVISION:	
DRAWING NO:	TP17-4

PHOTOS – TEST PIT #4

Test Pit #4 – Back of chamber wall

Test Pit #4 – Back of chamber wall



PLAN
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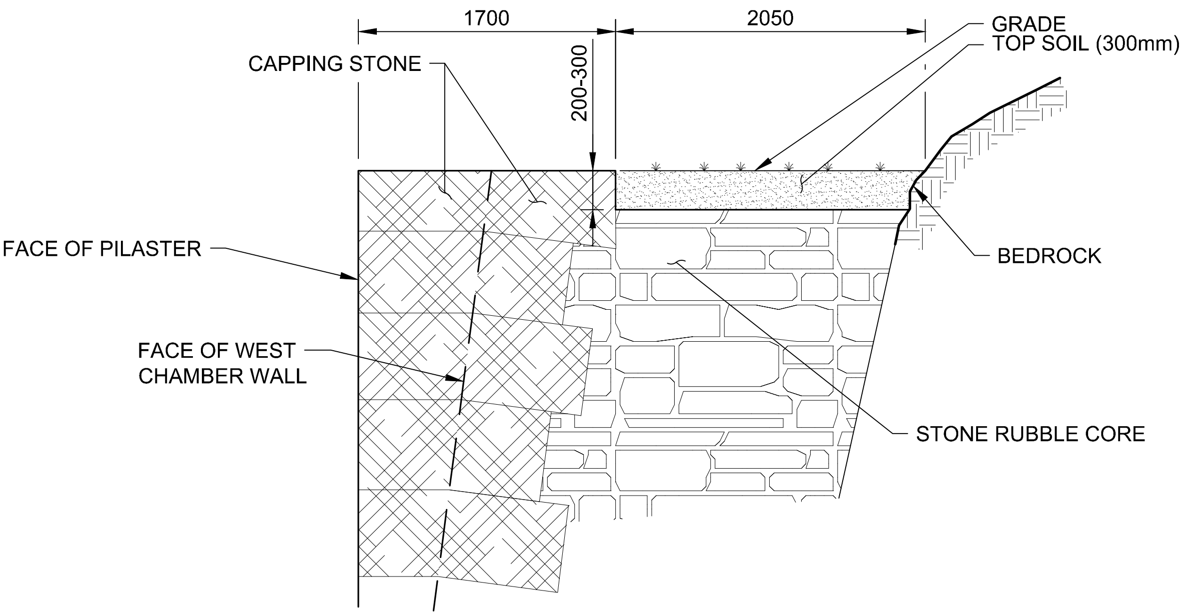


SECTION A TP17-5
1:50

300-2611 QUEENSVIEW DRIVE
OTTAWA (ONTARIO)
CANADA K2B 8K2
TELEPHONE: 613-829-2800 FAX: 613-829-8299
WWW.WSPGROUP.COM

TITLE:	KINGSTON MILLS LOCK STATION TEST PIT 17-5
SCALE:	AS SHOWN
DATE:	7-Feb-2018
PROJECT NO:	171-02359-00
REVISION:	
DRAWING NO:	TP17-5

PHOTOS – TEST PIT #5

Test Pit #5 – Rubble behind wall to face of bedrock outcrop

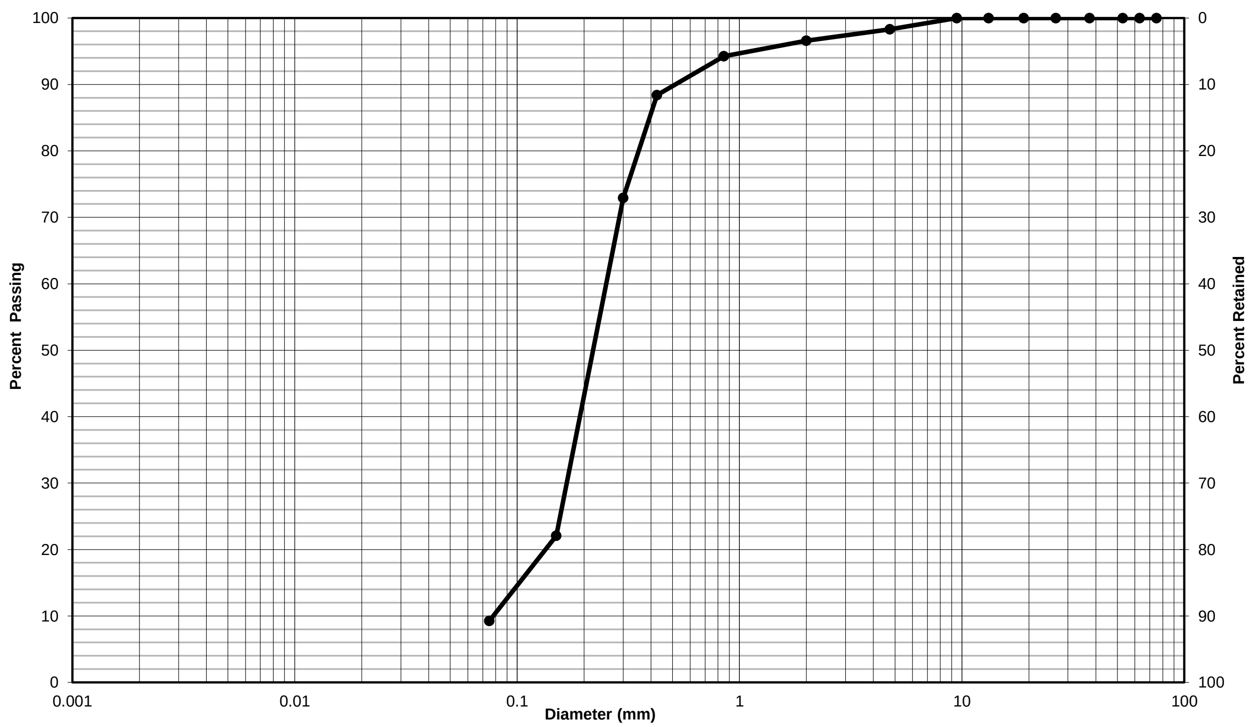
Test Pit #5 – Rubble behind wall to face of bedrock outcrop

APPENDIX

C-4 LABORATORY TESTING RESULTS

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL 229-6
Project/Site:	Kingston Mills Rehabilitation	**Project no.:**	171-02359-00
Borehole no.:	TP 2	Sample no.:	GS 2
Depth:	0.47-0.57m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

	Gravel	Sand	Clay & Silt	Silt	Clay
Percent %	1.7	89.1	9.3	-	-

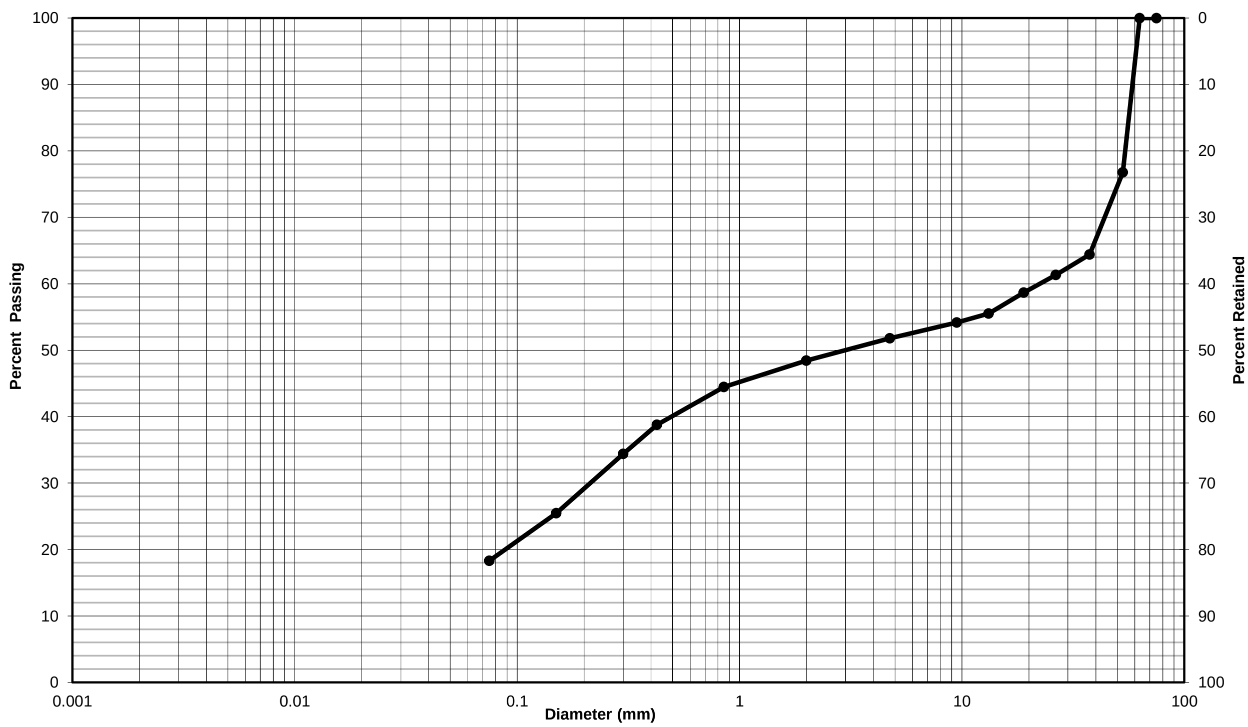
Remarks:

Performed by:	N.Krebs	**Date:**	October 31, 2017
Verified by:	N.Krebs	**Date:**	October 31, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL 229-8
Project/Site:	Kingston Mills Rehabilitation	**Project no.:**	171-02359-00

Borehole no.:	TP 3	Sample no.:	GS 3
Depth:	1-2m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent %	Gravel	Sand	Clay & Silt	Silt	Clay
	48.2	33.5	18.3	-	-

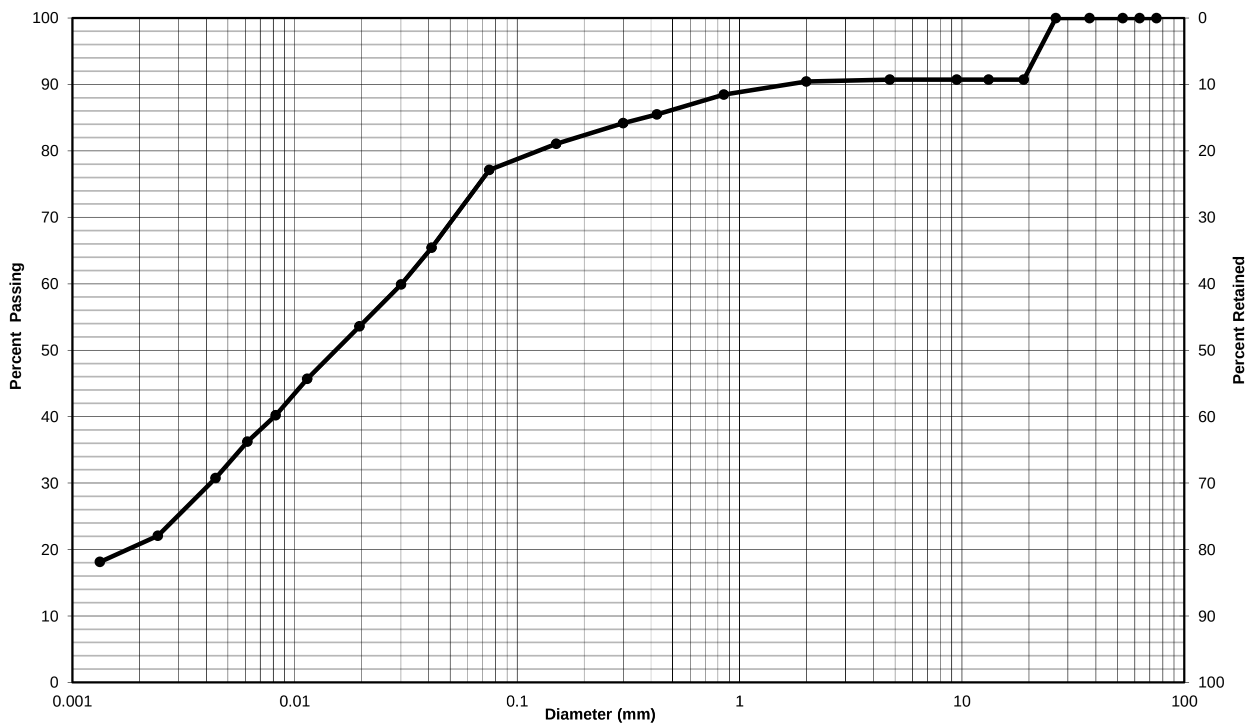
Remarks: 1 large, un-representative 3" stone omitted from sample

Performed by:	N.Krebs	**Date:**	October 31, 2017
Verified by:	N.Krebs	**Date:**	October 31, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL 229-1
Project/Site:	Kingston Mills Rehabilitation	**Project no.:**	171-02359-00

Borehole no.:	17-1	Sample no.:	SS4
Depth:	2.25-2.85m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent %	Gravel	Sand	Clay & Silt	Silt	Clay
	9.3	13.6	77.1	56.1	21.0

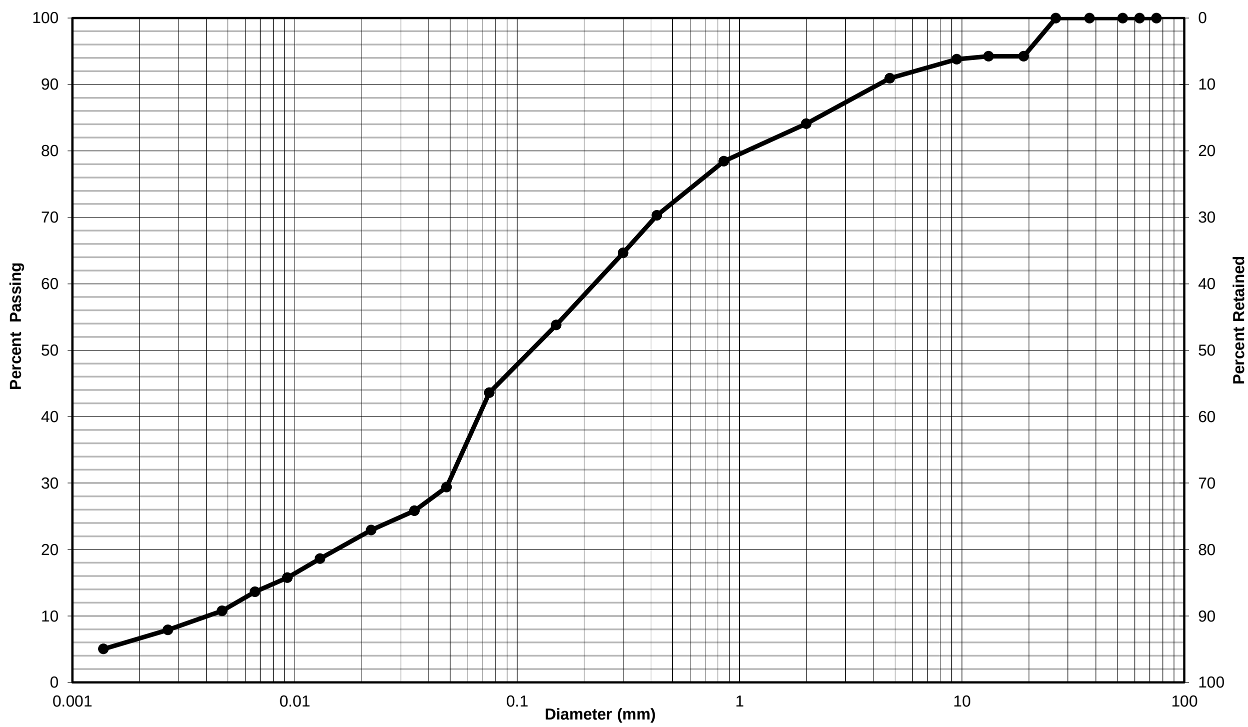
Remarks:

Performed by:	N.Krebs	**Date:**	November 7, 2017
Verified by:	N.Krebs	**Date:**	November 7, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL 229-2
Project/Site:	Kingston Mills Rehabilitation	**Project no.:**	171-02359-00

Borehole no.:	17-2	Sample no.:	SS3
Depth:	1.5-2.1m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent %	Gravel	Sand	Clay & Silt	Silt	Clay
	9.1	47.3	43.6	36.6	7.0

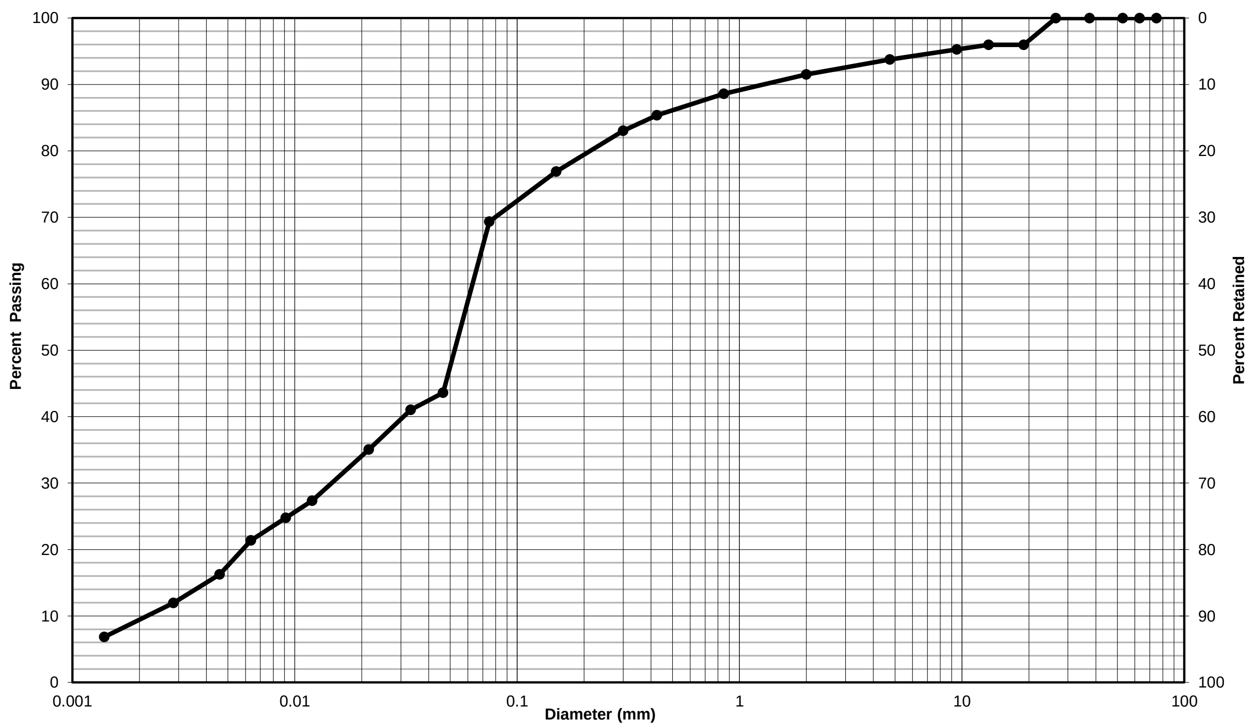
Remarks:

Performed by:	N.Krebs	**Date:**	November 7, 2017
Verified by:	N.Krebs	**Date:**	November 7, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL 229-3
Project/Site:	Kingston Mills Rehabilitation	**Project no.:**	171-02359-00

Borehole no.:	17-3	Sample no.:	SS3
Depth:	1.5-2.1m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

Percent %	Gravel	Sand	Clay & Silt	Silt	Clay
	6.3	24.4	69.3	60.3	9.0

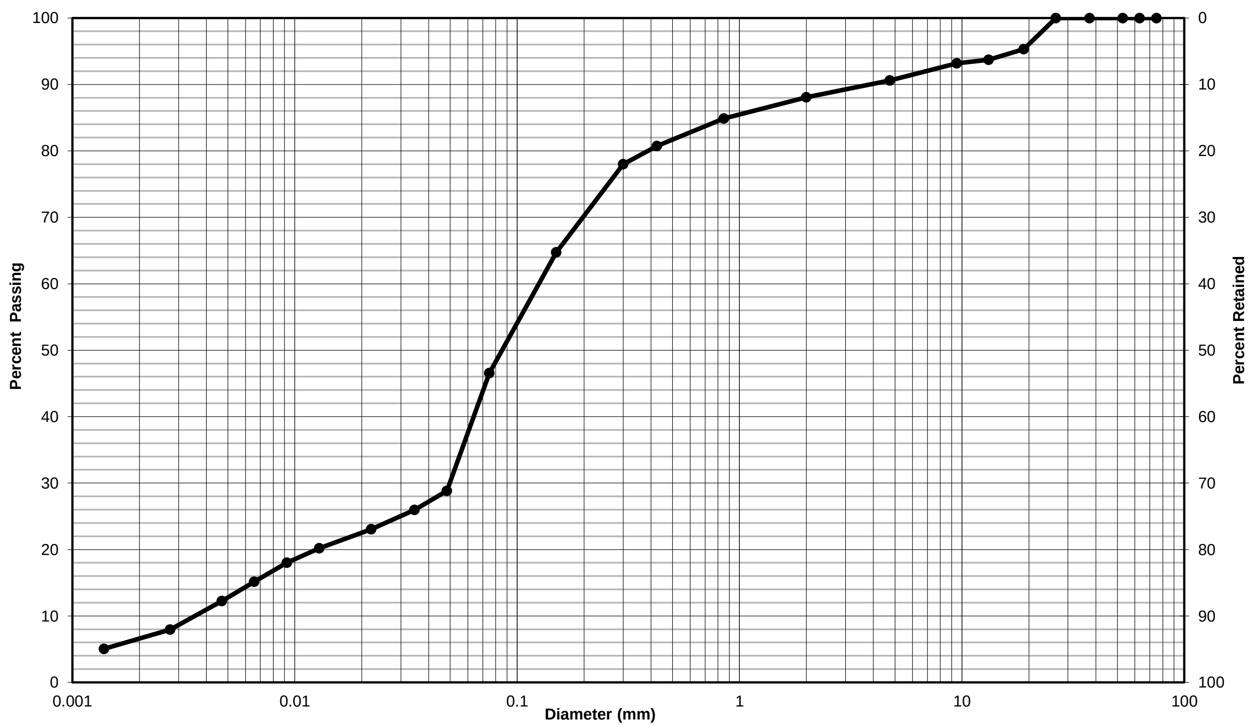
Remarks:

Performed by:	N.Krebs	**Date:**	November 8, 2017
Verified by:	N.Krebs	**Date:**	November 8, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL 229-4
Project/Site:	Kingston Mills Rehabilitation	**Project no.:**	171-02359-00

Borehole no.:	17-6	Sample no.:	SS4
Depth:	2.25-2.85m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

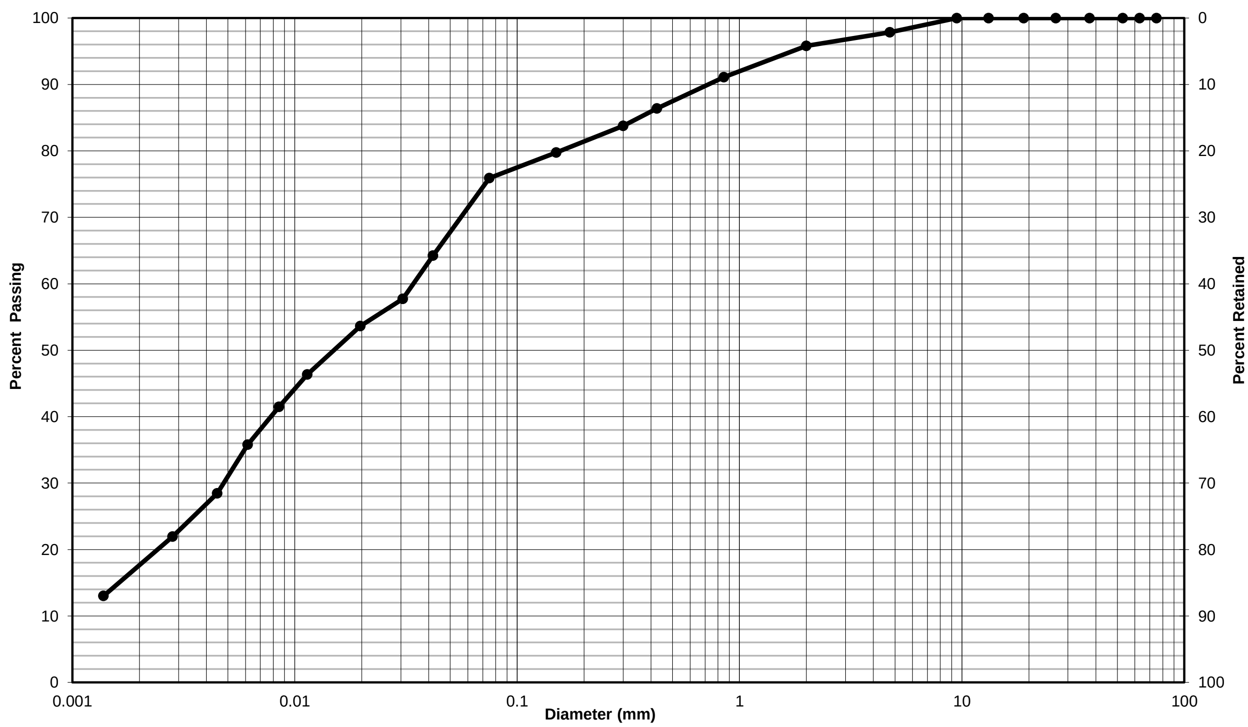
	Gravel	Sand	Clay & Silt	Silt	Clay
Percent %	9.4	44.0	46.5	40.5	6.0

Remarks:

Performed by:	N.Krebs	**Date:**	November 8, 2017
Verified by:	N.Krebs	**Date:**	November 8, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL 229-5
Project/Site:	Kingston Mills Rehabilitation	**Project no.:**	171-02359-00
Borehole no.:	17-6	Sample no.:	SS9
Depth:	6.1-6.7m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

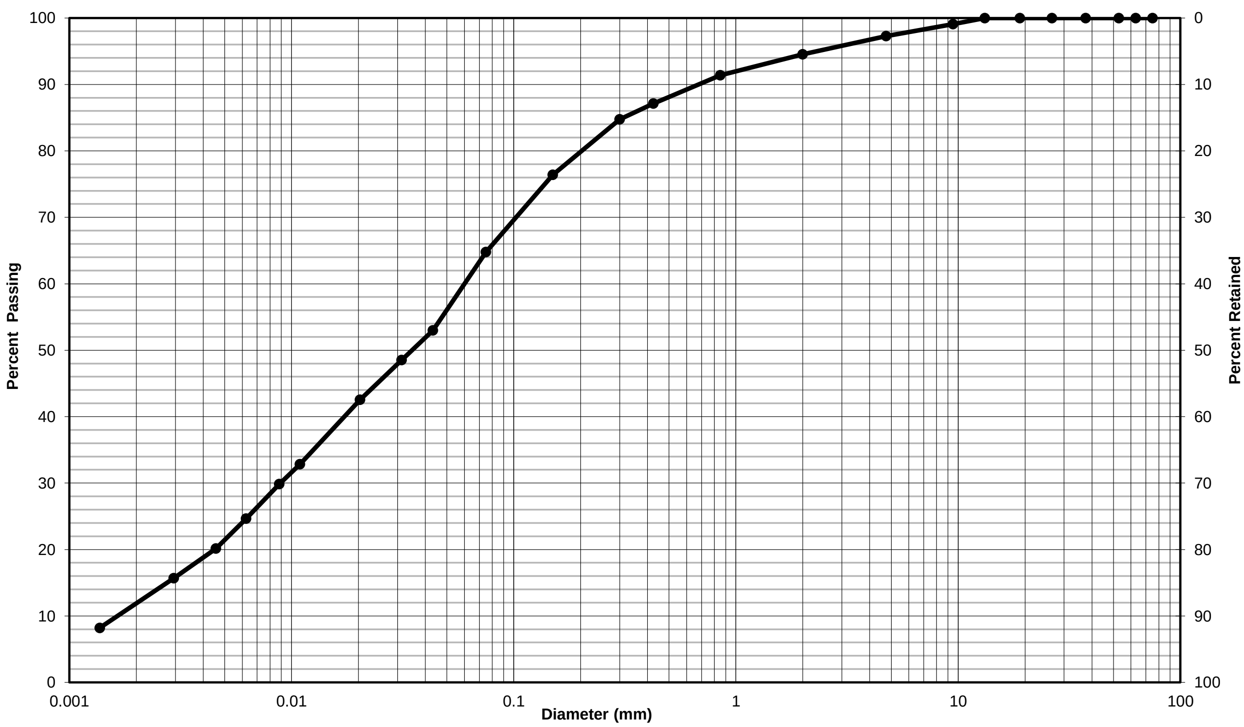
	Gravel	Sand	Clay & Silt	Silt	Clay
Percent %	2.1	22.0	75.9	57.9	18.0

Remarks:

Performed by:	N.Krebs	**Date:**	November 8, 2017
Verified by:	N.Krebs	**Date:**	November 8, 2017

Particle-Size Analysis of Soils
(ASTM D422)

Client:	Parks Canada	**Lab no.:**	OL 229-7
Project/Site:	Kingston Mills Rehabilitation	**Project no.:**	171-02359-00
Borehole no.:	TP 2	Sample no.:	GS 3
Depth:	0.7-1m		



Clay & Silt	Sand			Gravel	
	Fine	Medium	Coarse	Fine	Coarse
Unified Soil Classification System					

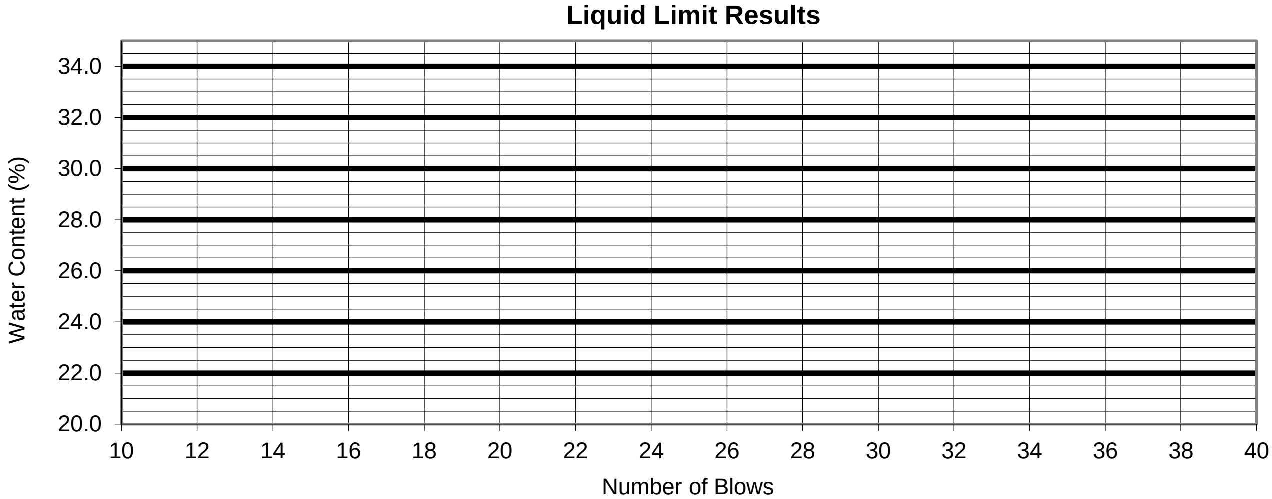
Percent %	Gravel	Sand	Clay & Silt	Silt	Clay
	2.7	32.5	64.8	52.8	12.0

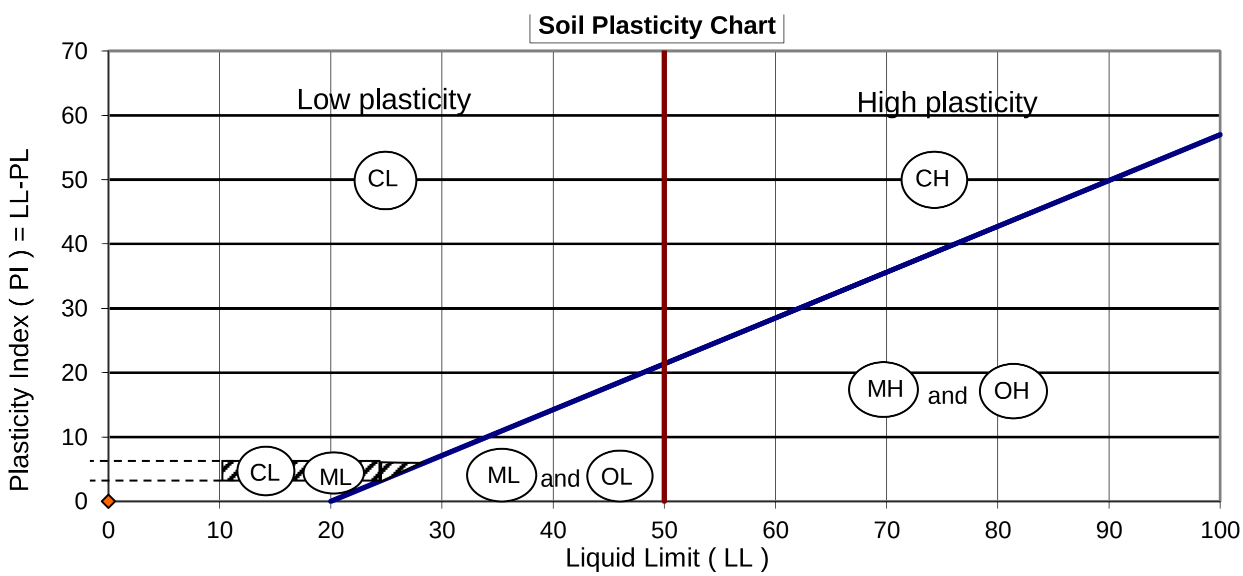
Remarks:

Performed by:	N.Krebs	**Date:**	November 8, 2017
Verified by:	N.Krebs	**Date:**	November 8, 2017

Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	Parks Canada	**Lab No.:**	OL 229-2
Project/Site:	Kingston Mills Lockstation Rehabilitation	**Project No.:**	171-02359-00
Borehole No.:	17-2	Sample No.:	SS 3
Sample Depth:	1.5-2.1m		





Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
0	0	0	18.0

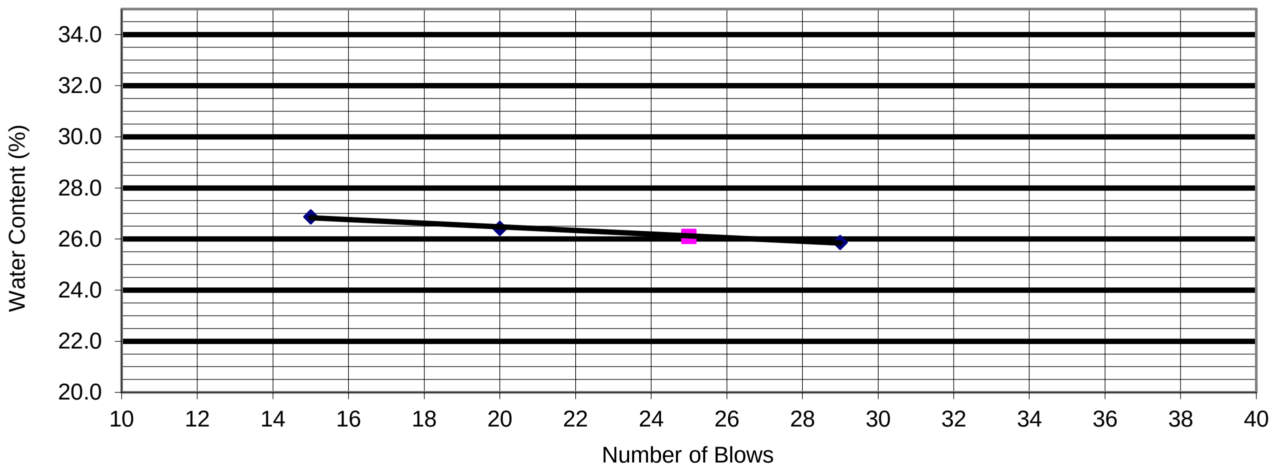
Sample Description: **Non-Plastic**

Performed By:	J.Meehan	**Date:**	November 8, 2017
Verified By:	N.Krebs	**Date:**	November 9, 2017

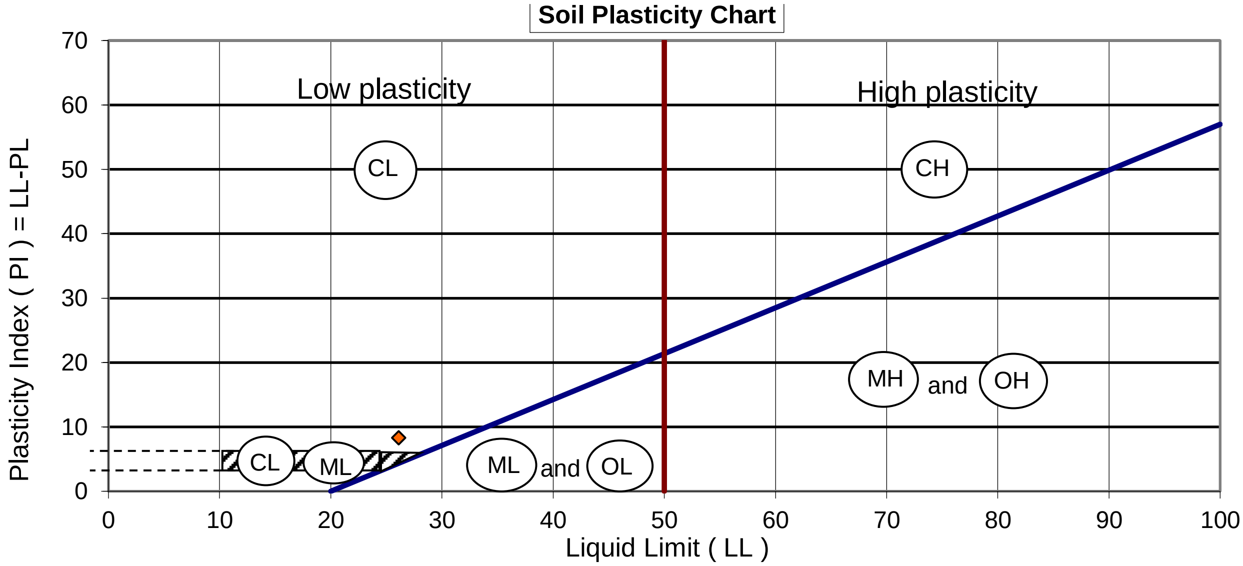
Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	Parks Canada	**Lab No.:**	OL 229-3
Project/Site:	Kingston Mills Lockstation Rehabilitation	**Project No.:**	171-02359-00
Borehole No.:	17-3	Sample No.:	SS 3
Sample Depth:	1.5-2.1m		

Liquid Limit Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
26	18	8	29.0

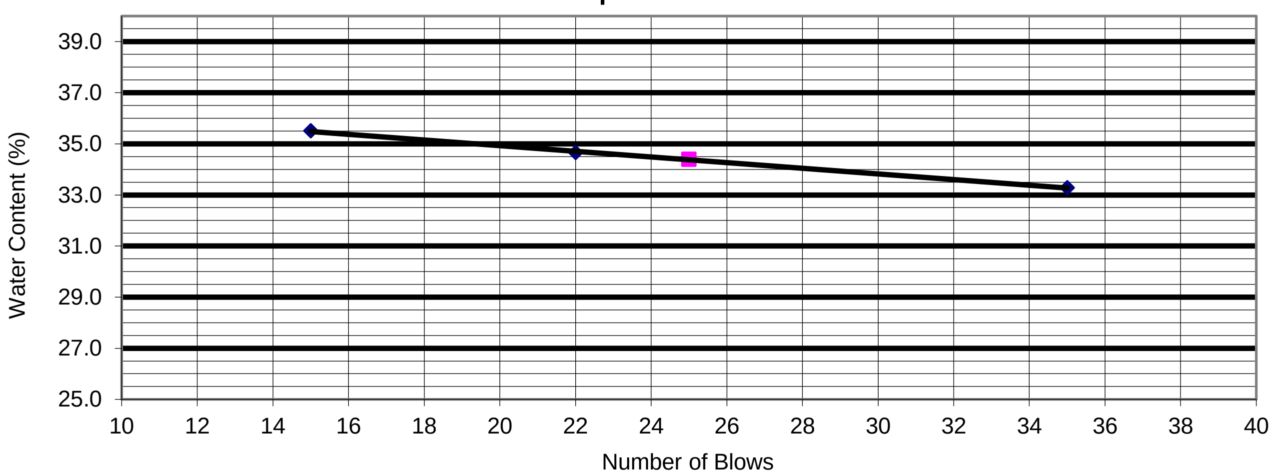
Sample Description: **CL - Low plasticity, inorganic clay**

Performed By:	J.Meehan	**Date:**	November 8, 2017
Verified By:	N.Krebs	**Date:**	November 9, 2017

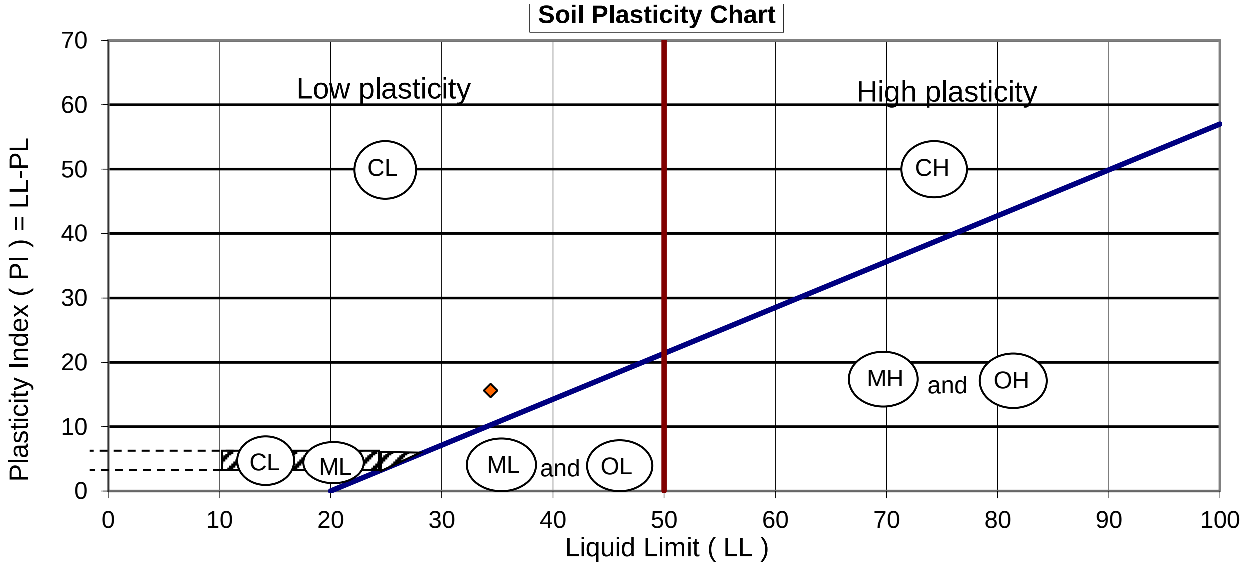
Liquid Limit, Plastic Limit and Plasticity Index of Soils (ASTM D4318)

Client:	Parks Canada	**Lab No.:**	OL 229-5
Project/Site:	Kingston Mills Lockstation Rehabilitation	**Project No.:**	171-02359-00
Borehole No.:	17-6	Sample No.:	SS 9
Sample Depth:	6.1-6.7m		

Liquid Limit Results



Soil Plasticity Chart



Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Natural Water Content %
34	19	16	39.3

Sample Description: **CL - Low plasticity, inorganic clay**

Performed By:	J.Meehan	**Date:**	November 2, 2017
Verified By:	N.Krebs	**Date:**	November 2, 2017

APPENDIX

C-5 *GROUNDWATER DATA*

Figure SWL-1: Static Water Levels and Groundwater Temperature
BH17-1 (2017/2018)

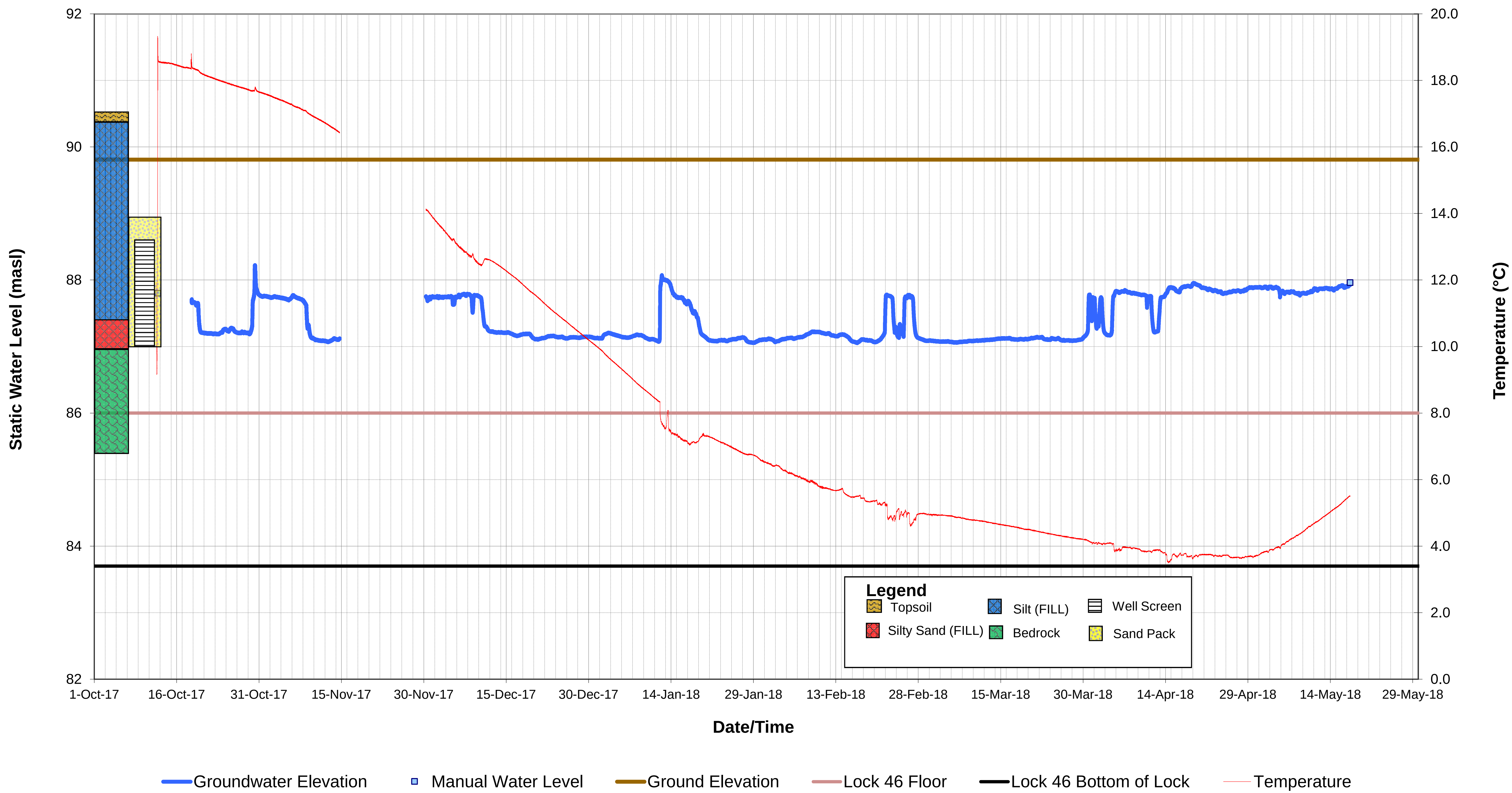


Figure SWL-1: Static Water Levels and Groundwater Temperature
BH17-2 (2017/2018)

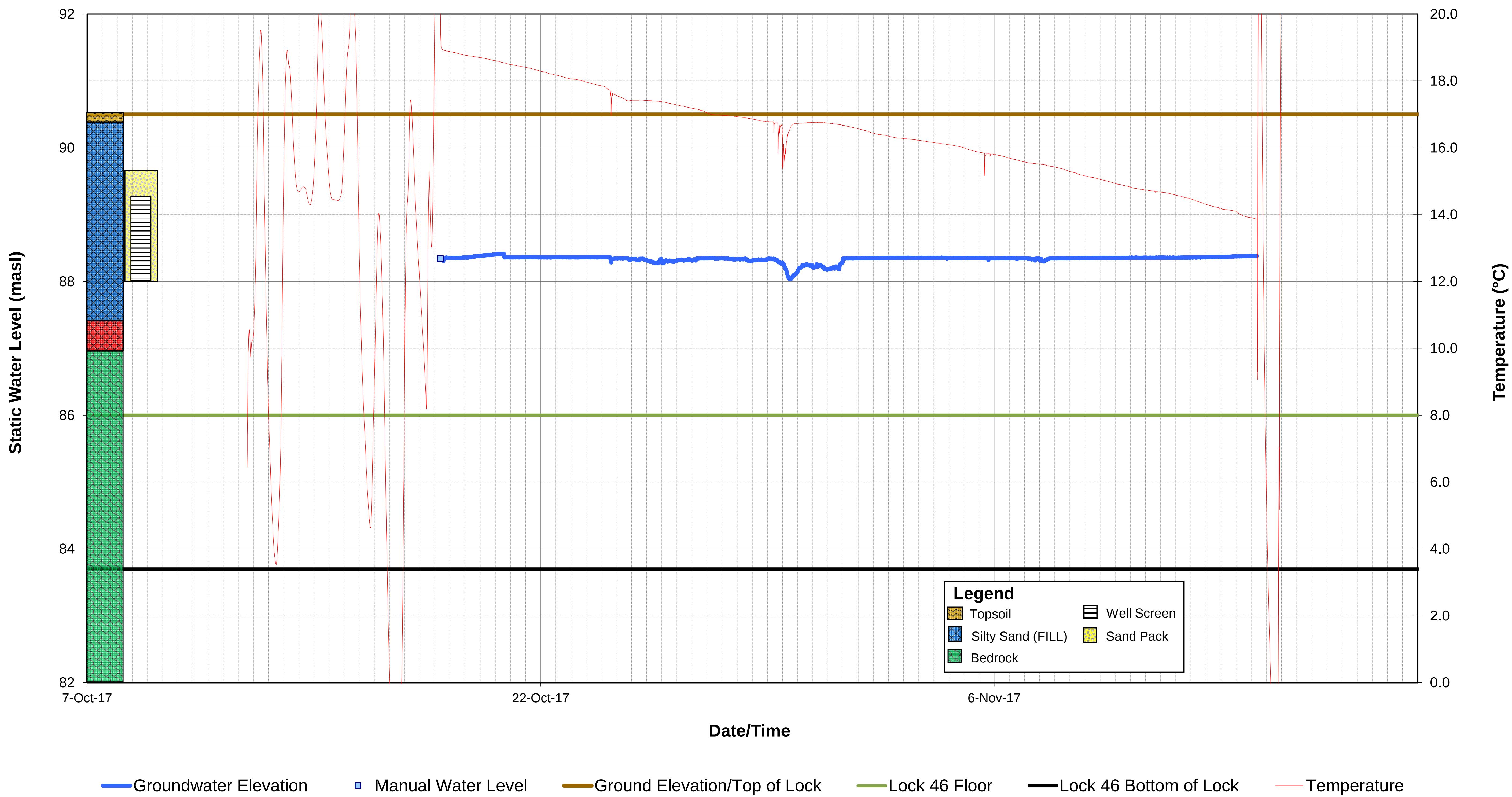
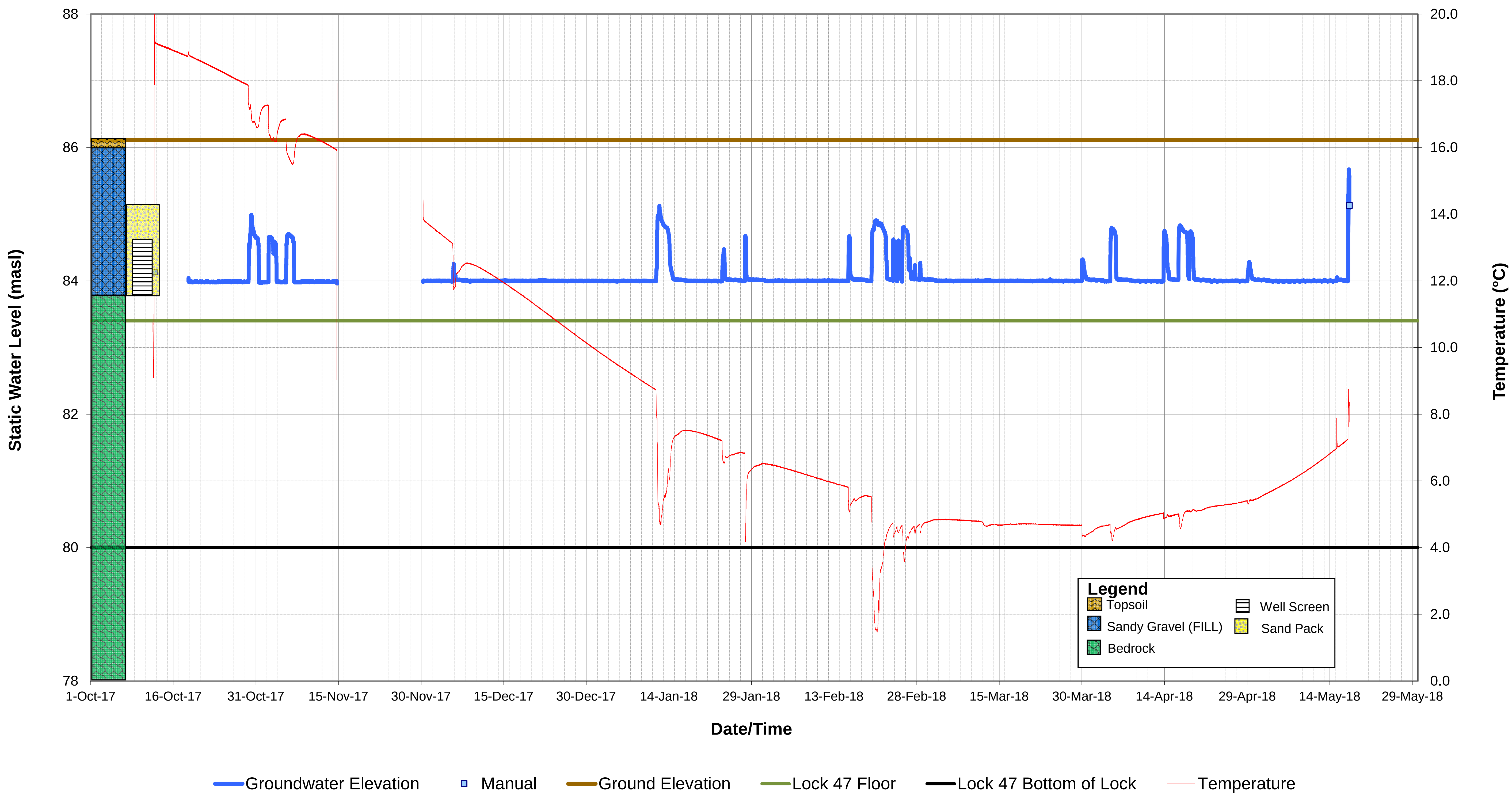


Figure SWL-1: Static Water Levels and Groundwater Temperature
BH17-3 (2017/2018)



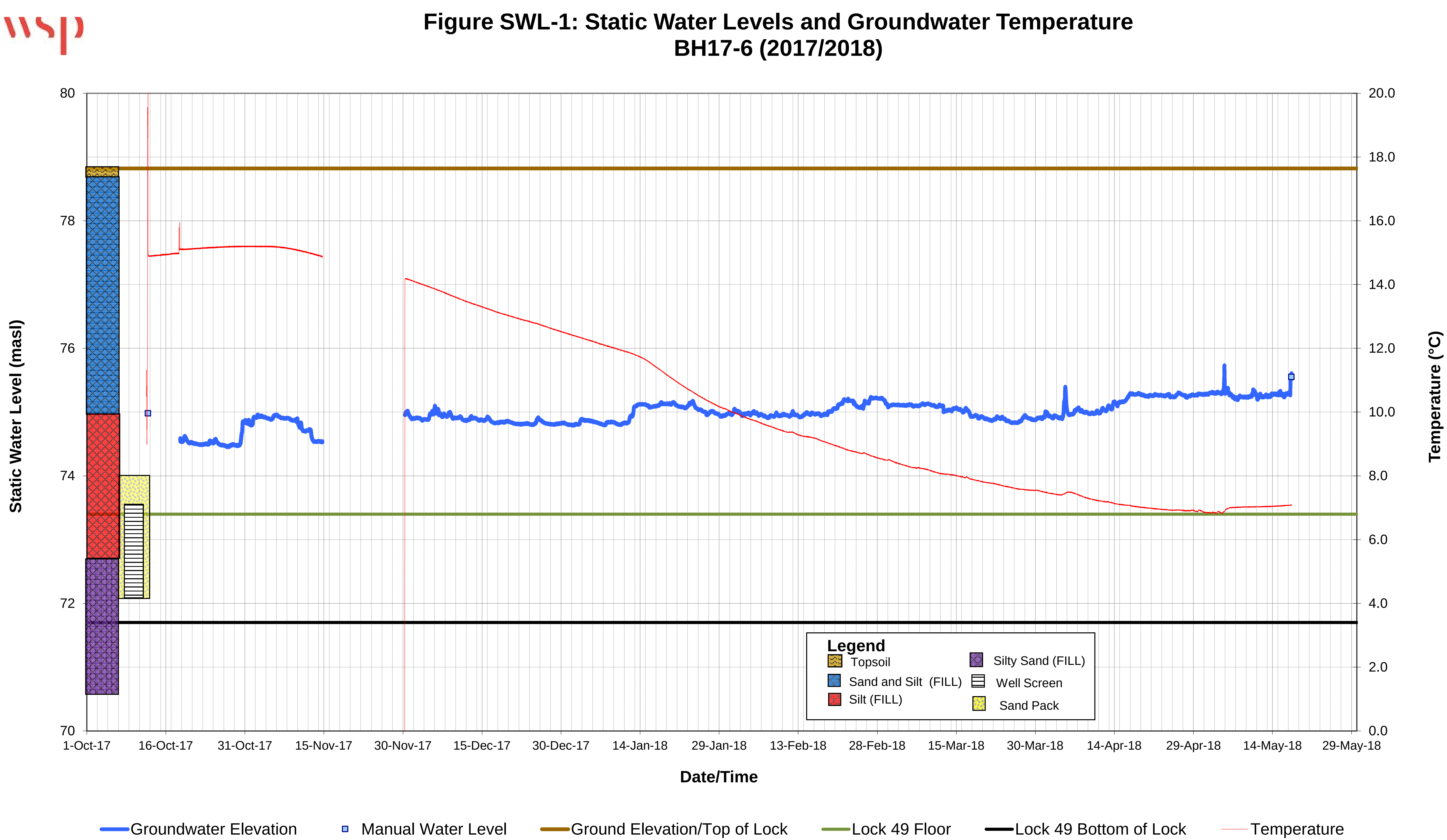


Figure SWL-1: Static Water Levels and Groundwater Temperature
BH17-6 (2017/2018)
Static Water Level (masl)
Temperature (°C)
Date/Time
80
78
76
74
72
70
20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
1-Oct-17
16-Oct-17
31-Oct-17
15-Nov-17
30-Nov-17
15-Dec-17
30-Dec-17
14-Jan-18
29-Jan-18
13-Feb-18
28-Feb-18
15-Mar-18
30-Mar-18
14-Apr-18
29-Apr-18
14-May-18
29-May-18
Legend
Topsoil
Sand and Silt (FILL)
Silt (FILL)
Silty Sand (FILL)
Well Screen
Sand Pack
Groundwater Elevation
Manual Water Level
Ground Elevation/Top of Lock
Lock 49 Floor
Lock 49 Bottom of Lock
Temperature

APPENDIX

D HISTORICAL INVESTIGATION

APPENDIX

D

HISTORICAL INVESTIGATION

APPENDIX

D-1 *1977 SITE INVESTIGATION SERVICES INVESTIGATION*

KM# 7

SWING BRIDGE

AT

KINGSTON MILLS

KM#7

SITE INVESTIGATION SERVICES

LIMITED

J. D. Lee Engineering Limited

1155 Division Street

Kingston, Ontario

KM# 7

SWING BRIDGE

AT

KINGSTON MILLS

KM#7

JOB #1608

JANUARY 1977

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

R. Marttila, P. Eng.

SITE INVESTIGATION SERVICES LIMITED

RM/bd

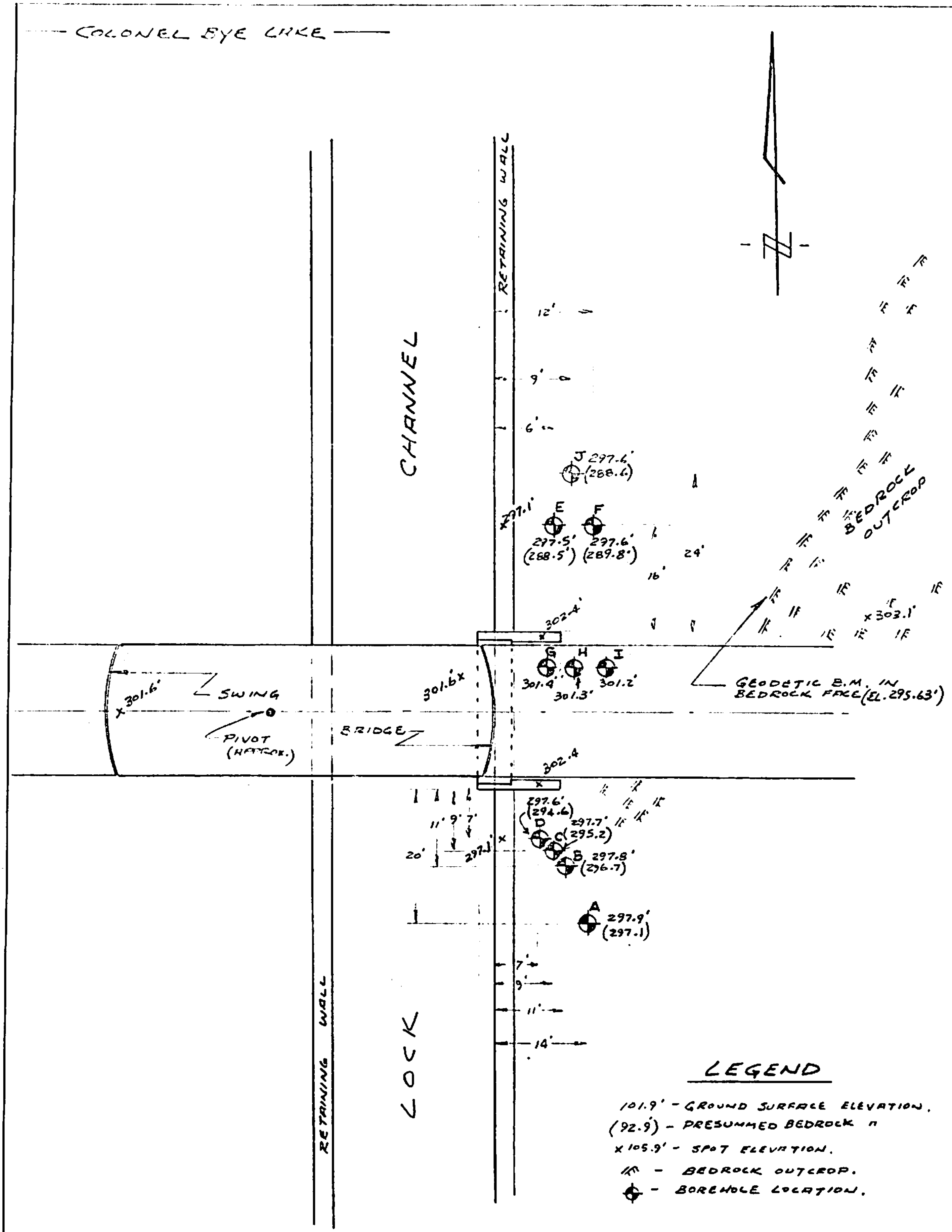
4 c.c.

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		



SITE INVESTIGATION SERVICES LIMITED

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 - 3¼'' H.S.A. (D-8)

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

LEGEND

Gravel | Sand | Clay

(See Appendix "A" for Other Symbols)

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	PENETRATION RESISTANCE (Blows/Ft) 2" O.D. Split Spoon, 2" O.D. Cone (10, 20, 30, 40, 50); SHEAR STRENGTH (Kips/Ft²) Field Vane - X, Unconfined Compression - □
		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							50 blows/9''
		293.7					
REFUSAL OF CONE PROBE							
			10				
			15				
			20				

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)

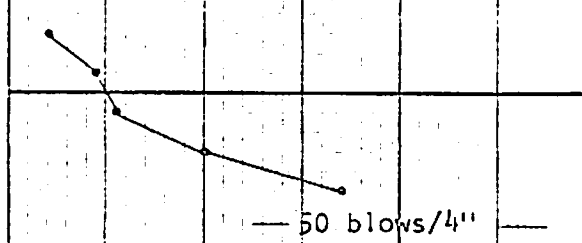
JOB No: 1608 **BOREHOLE No:** H

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

PENETRATION RESISTANCE (Blows/Ft)
2" O.D. Split Spoon —o—o— 2" O.D. Cone —o—o—
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
		301.3				
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			



BOREHOLE DATA and TEST SUMMARY

SITE INVESTIGATION SERVICES Ltd.

JOB No: 1608 | BOREHOLE No: J | FIGURE No: 4

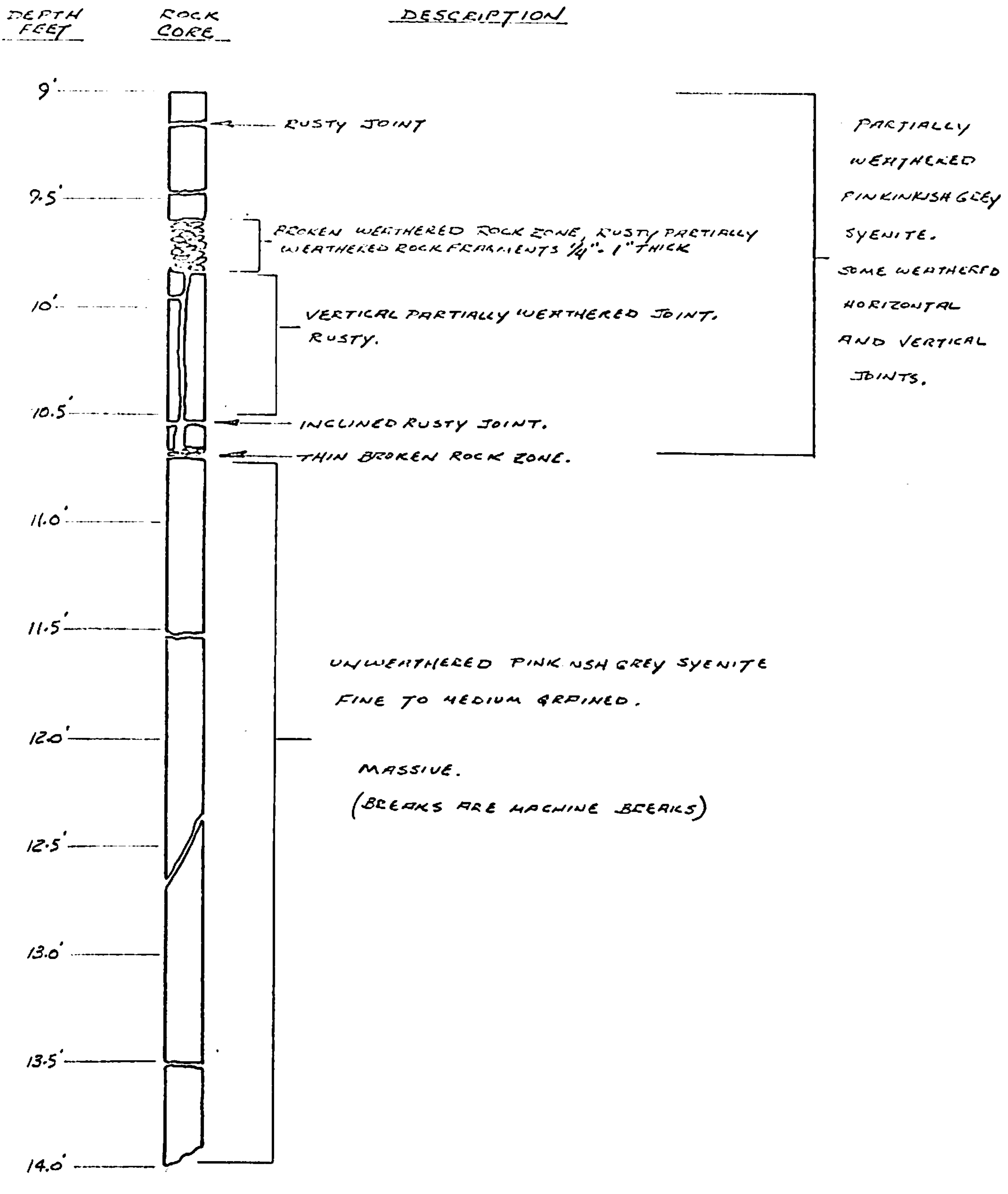
Project - KINGSTON MILLS
Location - SWING BRIDGE ABUTMENT
Hole Location - SEE PLAN

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)

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	PENETRATION RESISTANCE (Blows/Ft) 2" O.D. Split Spoon, 2" O.D. Cone: 10 20 30 40 50 / SHEAR STRENGTH (Kips/Ft²) Field Vane - X, Unconfined Compression - □
		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	98+% Recovery
END OF HOLE (Note: standpipe installed to 14' depth)			15				
			20				



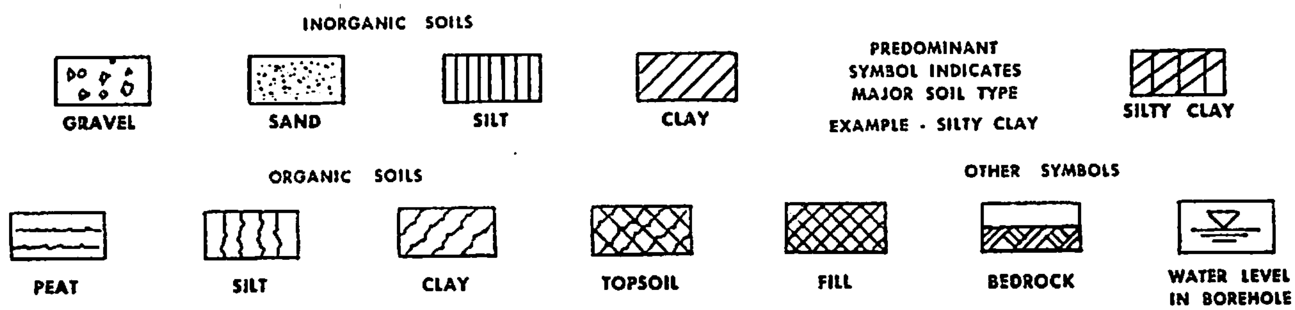
SITE INVESTIGATION SERVICES LIMITED

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

EXPLANATION OF SYMBOLS AND TEST DATA

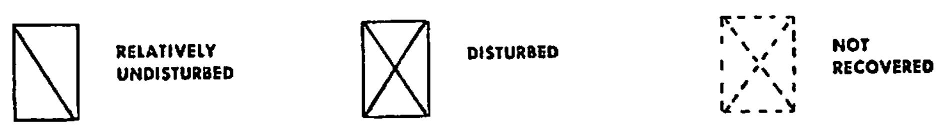
SOIL DESCRIPTION

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:



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_l - 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 drilling. Seasonal ground water fluctuations should be expected at most sites.

APPENDIX

D-2 *1979 GOLDER ASSOCIATES INVESTIGATION*

PARKS CANADA Km. 12

BOREHOLE RESULTS

KINGSTON MILLS LOCKSTATION

RIDEAU CANAL

KINGSTON ONTARIO

Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

REPORT

TO

PARKS CANADA Km. 12

BOREHOLE RESULTS
KINGSTON MILLS LOCKSTATION
RIDEAU CANAL

KINGSTON ONTARIO

Distribution:

4 copies - Parks Canada, Ontario Region
Cornwall, Ontario

2 copies - Golder Associates
Ottawa, Ontario

May, 1979 791-2023

Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

May 28, 1979

Parks Canada,
Ontario Region,
P.O. Box 1359,
Cornwall, Ontario.
N6H 5V4

ATTENTION: Mr. J. Mazhar, P.Eng.

RE: BOREHOLE RESULTS
KINGSTON MILLS LOCKSTATION
RIDEAU CANAL
KINGSTON, ONTARIO

Dear Sirs:

This letter reports the results of two (2) borings put down at specified locations at the Kingston Mills Lockstation, which is on the Rideau Waterway system immediately north of Kingston, Ontario.

The purpose of these borings was to supplement subsurface information already on hand by Parks Canada for the Kingston Mills Lockstation. The borehole locations are shown on the attached Figure 2. The borings were taken to depth and bedrock was proven by recovering some 4 metres of core from each boring. Concrete, masonry, and overburden strata were identified during drilling by core barrel and drive open sampling. The field work was carried out during May 1979 using a raft mounted diamond drill rig supplied and operated by the F.E. Johnston Drilling Co. Ltd. of Ottawa. The field work was supervised and co-ordinated by a member of our engineering staff.

The soil, masonry, and rock core samples were brought to our Ottawa laboratory for detailed examination. No laboratory testing was carried out on any of the samples recovered during this investigation but all samples were examined in detail.

A detailed log of each boring is given on the Record of Borehole sheets following the text of this report. The detailed subsurface profile shown on these logs is based on observation of the drilling operation and on examinations, both in the field and in the laboratory, of the samples recovered from the borings.

As can be seen on the boring logs, the bedrock is indicated to be at a depth of some 27 metres below the lower sill of Lock 49. The sill itself is underlain by a silty clay fill with some boulders on top. The sill between locks 49 and 48 however, appears to be underlain directly by the bedrock. Visual observation also showed that the floor of Lock 48 is also bottomed on bedrock.

The masonry, overburden, and rock core samples will be held in our Ottawa laboratory should you wish to examine them. We trust that this letter along with the attached borehole logs and figures are sufficient for your present purposes. Should you require further information or if we can be of additional service to you on this project, please call us.

Yours very truly,
GOLDER ASSOCIATES

REGISTERED PROFESSIONAL ENGINEER
R. A. J. MONTGOMERY
PROVINCE OF ONTARIO

R.A. Montgomery, P.Eng.

RAM:cn
791-2023

Att. Abbreviations & Symbols
Record of Borehole sheets
Figures 1 and 2

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole", on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
DS Denison type sample
FS foil sample
RC rock core
ST slotted tube
TO thin-walled, open
TP thin-walled, piston
WS wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 63.5 kilogram hammer dropped 0.76 metres required to drive a 5.1 millimetre diameter, 60 degree cone 0.3 metres, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, N: The number of blows by a 63.5 kilogram hammer dropped 0.76 metres required to drive a 5.1 millimetre drive open sampler 0.3 metres.

WH sampler advanced by static weight - weight, hammer

PH sampler advanced by pressure - pressure, hydraulic

PM sampler advanced by pressure - pressure, manual

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Relative Density	N, blows/0.3 metres
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	C_u, kilopascals
Very soft	Less than 12
Soft	12 to 24
Firm	24 to 48
Stiff	48 to 96
Very stiff	96 to 192
Hard	over 192

IV. SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH combined analysis, sieve and hydrometer[1]
Q undrained triaxial[2]
R consolidated undrained triaxial[2]
S drained triaxial
U unconfined compression
V field vane test

Notes:

[1]Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

[2]Undrained triaxial tests in which pore pressures are measured are shown as $\bar{Q}$ or $\bar{R}$.

LIST OF SYMBOLS

I. GENERAL

$\pi = 3.1416$

e = base of natural logarithms 2.7183

$\log_e a$ or $\ln a$, natural logarithm of a

$\log_{10} a$ or $\log a$, logarithm of a to base 10

t time

g acceleration due to gravity

V volume

W weight

M moment

F factor of safety

II. STRESS AND STRAIN

u pore pressure

σ normal stress

σ' normal effective stress ($\bar{\sigma}$ is also used)

τ shear stress

ϵ linear strain

ϵ_{xy} shear strain

ν Poisson's ratio (μ is also used)

E modulus of linear deformation (Young's modulus)

G modulus of shear deformation

K modulus of compressibility

η coefficient of viscosity

III. SOIL PROPERTIES

(a) *Unit weight*

γ unit weight of soil (bulk density)

γ_s unit weight of solid particles

γ_w unit weight of water

γ_d unit dry weight of soil (dry density)

γ' unit weight of submerged soil

G_s specific gravity of solid particles $G_s = \gamma_s/\gamma_w$

e void ratio

n porosity

w water content

S_r degree of saturation

(b) *Consistency*

w_L liquid limit

w_P plastic limit

I_P plasticity index

w_S shrinkage limit

I_L liquidity index $= (w - w_P)/I_P$

I_C consistency index $= (w_L - w)/I_P$

e_{max} void ratio in loosest state

e_{min} void ratio in densest state

D_r relative density $= (e_{max} - e)/(e_{max} - e_{min})$

(c) *Permeability*

h hydraulic head or potential

q rate of discharge

v velocity of flow

i hydraulic gradient

k coefficient of permeability

j seepage force per unit volume

(d) *Consolidation (one-dimensional)*

m_v coefficient of volume change $= -\Delta e/(1+e)\Delta\sigma'$

C_c compression index $= -\Delta e/\Delta \log_{10} \sigma'$

c_v coefficient of consolidation

T_v time factor $= c_v t/d^2$ (d, drainage path)

U degree of consolidation

(e) *Shear strength*

τ_f shear strength

c' effective cohesion intercept; ϕ' effective angle of shearing resistance, or friction — in terms of effective stress $\tau_f = c' + \sigma' \tan \phi'$

c_u apparent cohesion*; ϕ_u apparent angle of shearing resistance, or friction — in terms of total stress $\tau_f = c_u + \sigma \tan \phi_u$

μ coefficient of friction

S_t sensitivity

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

Form G.A.-0-1. Project No. 791-2023

RECORD OF BOREHOLE 1

LOCATION: See Figure 2 BORING DATE: MAY 9-14, 1979 DATUM: GEODETIC

SAMPLER HAMMER WEIGHT: 63.5 Kg DROP 0.76 m PENETRATION TEST HAMMER WEIGHT: 63.5 Kg DROP 0.76 m

Column headings: BORING METHOD | SOIL PROFILE (ELEV'N. DEPTH, DESCRIPTION, STRAT. PLOT) | SAMPLES (NUMBER, TYPE, BLOWS/0.3) | ELEVATION SCALE | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3 (20, 40, 60, 80); SHEAR STRENGTH Cu., K Pa (NAT. V. - + Q. - ●; REM.V. - ⊕ U. - O; 20, 40, 60, 80) | COEFFICIENT OF PERMEABILITY, k, CM./SEC. (1x10, 1x10, 1x10, 1x10); WATER CONTENT, PERCENT (W_P, W, W_L) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

BOREHOLE DRILLED THROUGH LOWER SILL (LOCK 49)

Boring Method	Elev'n. / Depth	Description	Sample No.	Type	Blows/0.3
	75.18	TOP OF WATER			
NX CASING	0.00	WATER			
	72.42 / 2.76	FAIRLY SOUND CONCRETE WITH LEACHED MORTAR	1	BX RC	
			2	"	
	70.84 / 4.34	GRANITE BOULDERS			
	70.53 / 4.65	STIFF GREY SILTY CLAY, SOME SAND & SILT POCKETS SOME GRAVEL AND WOOD (FILL)	3	50 mm DO	8
			4	"	14
	69.10 / 6.08	VERY STIFF GREY SILTY CLAY SOME SAND AND SILT LENSES	5	"	7
	66.27 / 8.9±	LOOSE TO COMPACT GREY SANDY SILT, SOME CLAY AND GRAVEL, NUMEROUS SAND POCKETS	6	"	5
			7	"	29
	62.30 / 12.9±	VERY STIFF GREY SILTY CLAY SOME SAND & SILT LENSES			
	59.26 / 15.9±				

Core recovery: 42 (sample 1), 23 (sample 2) — CORE RECOVERY

Elevation scale: 75, 74, 73, 72, 71, 70, 69, 68, 67, 66, 65, 64, 63, 62, 61, 60

NOTE: W.L. IN CASING FROM SILTY SAND, GRAVEL & BOULDER STRATUM TO ELEV. 76.20 MAY 14, 1979

Method	Elev. / Depth	Description	Sample No.	Type	Blows	Elevation	Core Recovery	RQD
						63		
	62.30 12.9±		7	"	29	62		
		VERY STIFF GREY SILTY CLAY SOME SAND & SILT LENSES				61		
						60		
	59.26 15.9±					59		
		COMPACT GREY FINE SAND, SOME SILT AND CLAY	8	"	11	58		
						57		
ROTARY DRILLING	56.21 19.0±					56		
BX CASING		INTERLAYERED COMPACT GREY SILT WITH SAND AND CLAY LAYERS TO VERY STIFF GREY SILTY CLAY WITH SAND AND SILT LAYERS	9	"		55		
						54		
						53		
						52		
						51		
			10	"	21	50		
	45.81 25.4±		11	BX RC		49	89	
BX CORE		INTERLAYERED SILTY SAND GRAVEL & BOULDERS	12	"		48	18	
						47		
AX CASING						46		
						45		
	43.91 31.30		13	AX RC		44	32	
CORE		HIGHLY FRACTURED GREY TO PINK GRANITE	14	"		43	84	25
			15	"				

Method	Elevation / Depth	Description	Sample No.	Type	Blows	Core Recovery (%)	R.Q.D. (%)
BX CASING		SAND AND SILT LAYERS	10	"	21		
BX CORE	45.81 25.4±	INTERLAYERED SILTY SAND GRAVEL & BOULDERS	11	BX RC		89	
			12	"		18	
AX CASING							
AX CORE			13	AX RC		32	
	43.91 31.30	HIGHLY FRACTURED GREY TO PINK GRANITE BEDROCK	14	"		84	25
			15	"		93	37
			16	"		42	0
			17	"		81	20
	39.91 35.30	END OF HOLE					

Elevation scale: 53, 52, 51, 50, 49, 48, 47, 46, 45, 44, 43, 42, 41, 40, 39, 38

R.Q.D. (%)
CORE RECOVERY (%)

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE
1:50

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Form G.A.-D-I.

Project No. 791-2023

RECORD OF BOREHOLE 2

LOCATION See Figure 2 — BORING DATE MAY 15, 1979 — DATUM GEODETIC

SAMPLER HAMMER WEIGHT 63.5 Kg. DROP 0.76m — PENETRATION TEST HAMMER WEIGHT 63.5 Kg. DROP 0.76m

BOREHOLE DRILLED THROUGH SILL (LOCK 49/48)

BORING METHOD	ELEV'N. DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	CORE RECOVERY (%)	R.Q.D. (%)
	76.78	TOP OF SILL				
ROTARY DRILLING, BX CASING	76.78 / 0.00	GREY LIMESTONE BLOCKS WITH LEACHED MORTAR SEAMS	1	BX RC	88	
			2	"	50	
	74.43 / 2.35	GRANITE & LIMESTONE BLOCKS & BOULDERS WITH LEACHED MORTAR	3	"	74	
			4	"	65	
ROTARY DRILLING, BX CORE	71.99 / 4.79	HIGHLY FRACTURED GREY GRANITE BEDROCK, SOME MORTAR IN FRACTURES NEAR TOP	5	"		
			6	"	100	58
			7	"	97	35
			8	"	100	19
	68.31 / 8.47	END OF HOLE				

Chart headings: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu., K Pa. NAT. V. - + Q. - ● REM.V. - ⊕ U. - O; COEFFICIENT OF PERMEABILITY, k, CM./SEC. (1x10, 1x10, 1x10, 1x10); WATER CONTENT, PERCENT W_p W W_L; ADDITIONAL LAB. TESTING; PIEZOMETER OR STANDPIPE INSTALLATION

Elevation scale: 77, 76, 75, 74, 73, 72, 71, 70, 69, 68, 67

R.Q.D. (%)
CORE RECOVERY (%)

15-0-5-10 Percent axial strain at failure

VERTICAL SCALE 1:50

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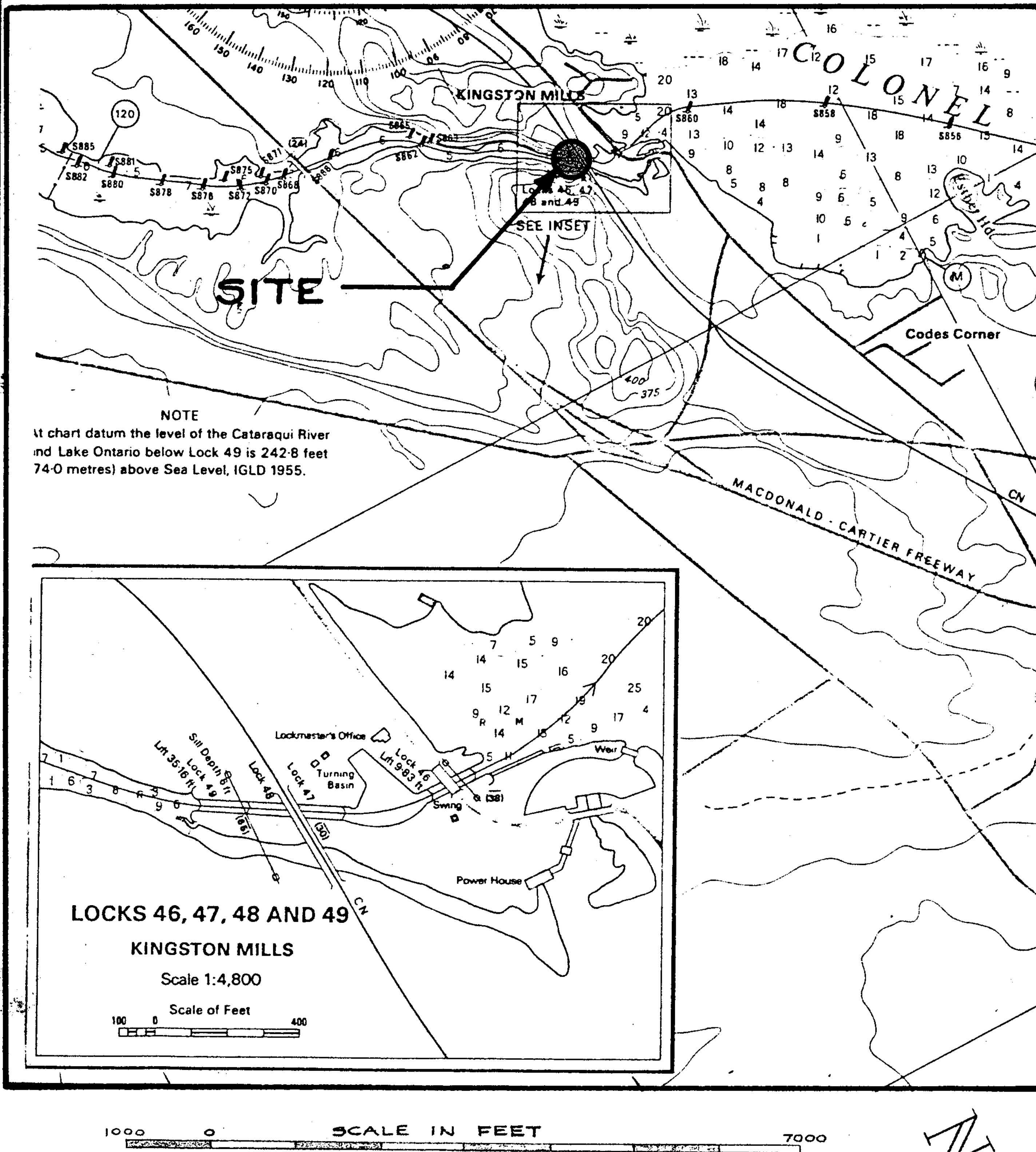
CHECKED [signature]

KEY PLAN

FIGURE 1



SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

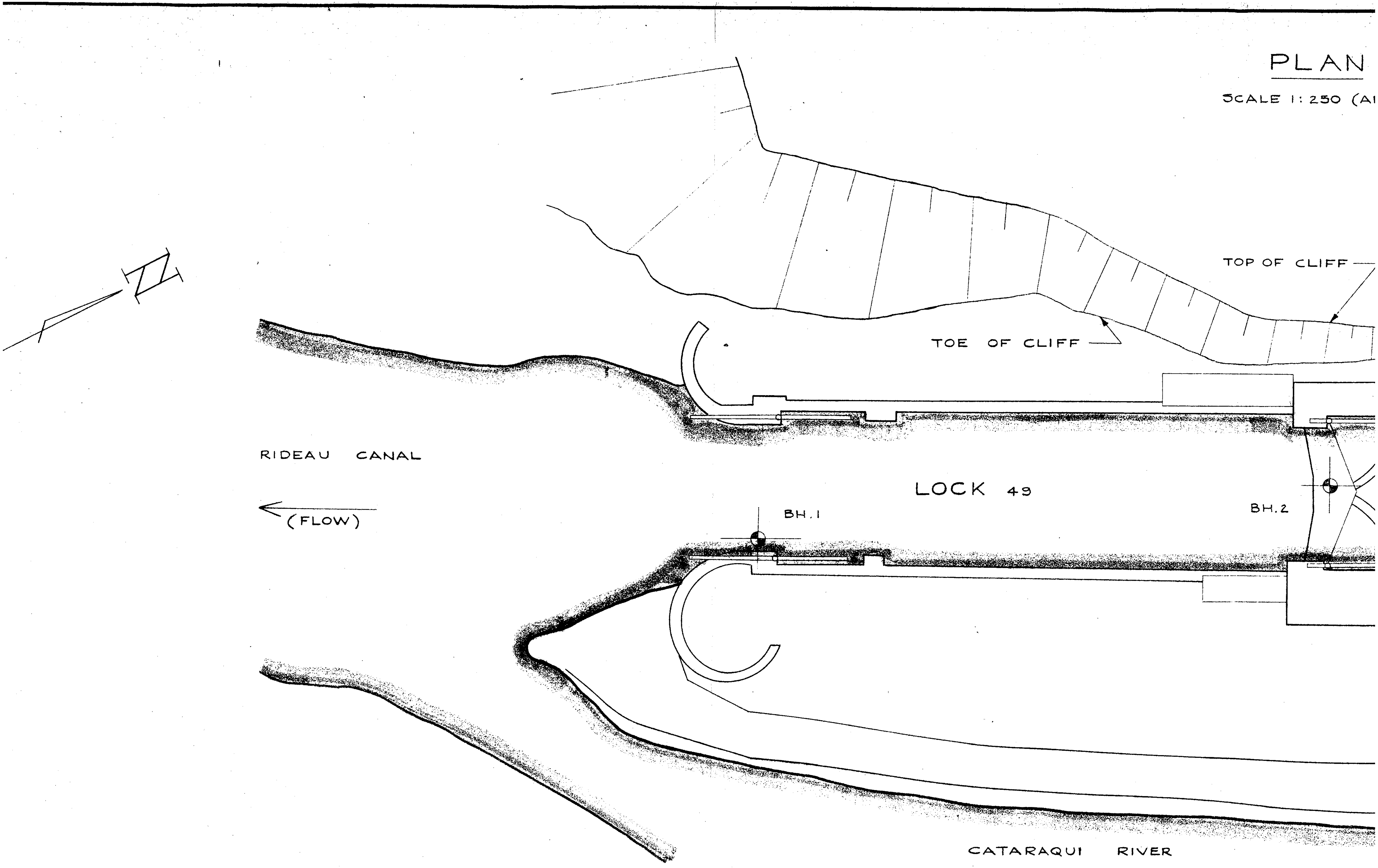
Date MAY 24, 1979

Project 791-2023

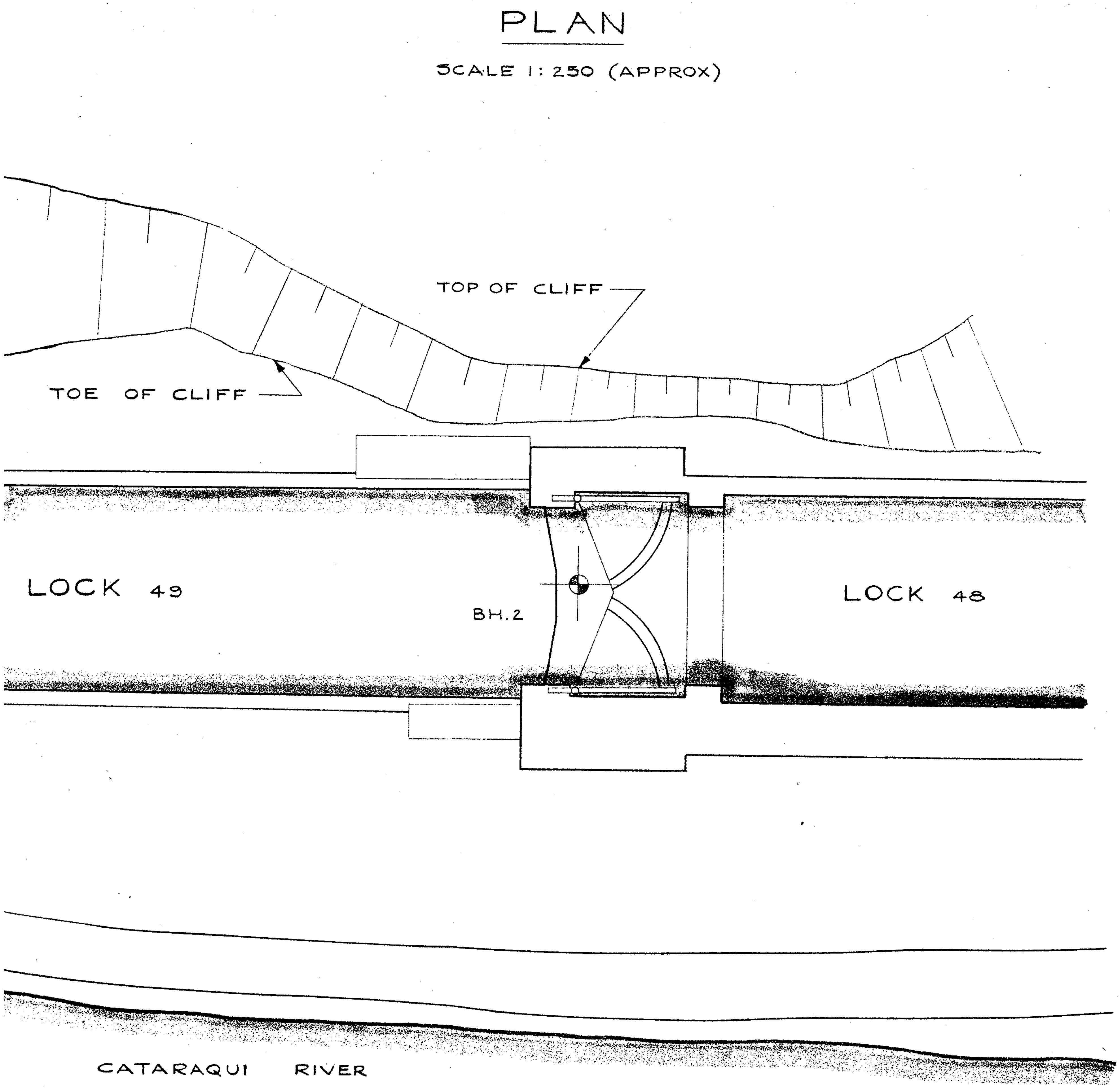
Golder Associates

Drawn RKB

Chkd.



PLAN
SCALE 1:250 (AI
N
TOP OF CLIFF
TOE OF CLIFF
RIDEAU CANAL
(FLOW)
LOCK 49
BH.1
BH.2
CATARAQUI RIVER



BORING PLAN

FIGURE 2

LEGEND

BOREHOLE IN PLAN

SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

REFERENCE

SITE PLAN SUPPLIED BY INDIAN AND NORTHERN AFFAIRS (PARKS CANADA)

NOTE

Data concerning the various strata have been obtained at borehole locations only. The soil stratigraphy between the boreholes has been inferred from geological evidence and so may vary from that shown.
For detailed stratigraphy at each borehole location refer to the record of borehole sheets.

Date MAY 24, 1979

Project 791-2023

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APPENDIX

D-3 *1990 TROW INVESTIGATION*

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Trow

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GEOTECHNICAL INVESTIGATION GROUTING TEST PROGRAM LOCKS 47 AND 48, KINGSTON MILLS LOCKS KINGSTON, ONTARIO

Prepared For:
CANADIAN PARKS SERVICES
ENVIRONMENT CANADA

TROW ONTARIO LTD.

Project No.: R-00404A/GE

Date: January 24, 1990

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6
- spare - given to JM / low bidder (on Oct. 18/94)
- R.C.
- M.W.T.
- Lib.
- files
- spare (1)

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GEOTECHNICAL INVESTIGATION AND GROUTING TEST PROGRAM LOCKS 47 AND 48, KINGSTON MILLS LOCKS KINGSTON, ONTARIO

SUMMARY

A geotechnical investigation and grouting test program was undertaken at the site Locks 47 and 48 of the Kingston Mills Locks located at the southern end of the Rideau Canal, on the Cataraqui River approximately 5 km north of Kingston, Ontario. This work was authorized by Canadian Parks Services, Environment Canada.

The investigation revealed that the west wall of the locks is built against the bedrock face whereas the east wall acts as a retaining wall. Both the locks are founded on bedrock which varies from gneissic syenite to metadiabase.

The grouting test program was undertaken to assess the feasibility and suitability of grouting techniques to rehabilitate the historical natural stone structure, to prevent decay of time due to erosion and freeze thaw action. The grouting test program was successfully completed and achieved the demonstration of various cementitious and chemical grouts as well as joint repair techniques as a function of specific site conditions.

It is concluded that the Kingston Mills lock structures can be economically and adequately repaired by cementitious grouting techniques, provided that:

a) specially designed grouting formulations and selected grouting materials are employed to accommodate the wide range of repair conditions found to exist in the locks.

b) suitable grouting equipment and accessories are employed to achieve design levels of performance and to maintain quality control in placing the various grouting formulations.

c) proper grouting techniques are applied to install a variety of suitable formulations under properly monitored conditions.

d) various repointing techniques and materials are used in conjunction with the actual injection work to hold the grouting materials in place while curing.

It must be emphasized that there are no "miracle" solutions to the repair problems found in the Kingston Mills lock structures. A systematic and organized approach however is required to achieve satisfactory results.

Field conditions must be constantly monitored and combinations of grouting materials and grouting techniques must be modified in the field as required to suit specific conditions.

The general conclusion of our test program is that a properly engineered grouting program is an economical and most suitable approach to rehabilitate the lock structures at Kingston Mills.

INTRODUCTION

A geotechnical investigation was conducted at the Kingston Mills Locks, located at the southern end of the Rideau Canal, on the Cataraqui River, about 5 km north of Kingston, Ontario. Written authorization to proceed with the geotechnical investigation was provided by Public Works Canada, under agreement # 895 328 202 dated July 24, 1989.

The geotechnical investigation was undertaken to:

a) Establish the geotechnical and groundwater conditions encountered including bedrock type, percent Core Recovery for each borehole, Rock Quality Designation, static water level, permeability of the masonry walls and the bedrock, presence of any voids, discontinuities and joints.

b) Provide sketches of typical cross-sections through the walls.

c) Comment on durability of the masonry stones.

d) Recommend remedial measures in terms of the most suitable type of grout and grouting procedures to stop leakage at the walls.

This report is provided on the basis of the terms of reference presented above and on the assumption that the design will be in accordance with applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may be then necessary to carry out additional borings and reporting before the recommendations of the office may be relied upon.

BACKGROUND AND CURRENT PROBLEMS OF THE LOCKS

a) Background

The locks under investigation are located at Kingston Mills and were built of stone masonry in 1826-1832. Extensive repairs have been carried out to these locks through the years. In 1972 Lock Nos. 47 and 48 were pressure grouted. In 1979, pressure grouting of the east chamber walls of Lock Nos. 47 and 48, and all four gate sills in the flight locks was performed. Concrete floor slab were cast over the bedrock floors in Lock Nos. 47 and 48.

In 1960 a portion of the west wall of Lock No. 48 was dismantled, and the adjacent railway bridge pier was stabilized by grouting a void under the footings. The opening in the lock wall was filled with concrete.

b) Current Problems

Some of the problems currently being experienced with the locks are as follows:

1) Lock No. 47: extensive leakage occurs through the east wall when the lock chamber is full, and wet areas appear on the ground surface adjacent to the west wall. The lower monolith walls, upstream of the gate recess, show significant movement of individual stones. The lower monolith walls upstream and downstream of the gate recess show bulging and leaning of up to 165mm.

2) Lock No. 48: leakage occurs through the east wall and through the east sluices. The lower monolith walls are bulging and leaning by up to 160mm.

3) General: extensive re-pointing is needed in all three locks, and the stones on the top of the east lock chamber walls and monoliths have settled unevenly.

FIELD DRILLING PROGRAM

The fieldwork was undertaken from October 16 to November 5, 1989 with a thin wall diamond tipped core barrel mounted on a rotary electric drill. It was supervised on a full time basis by a geotechnical engineer from Trow Ontario Ltd.

The fieldwork was divided into two phases. The first phase consisted of drilling six vertical holes (Borehole Nos. V1 and V6) and twenty horizontal holes at the locations shown on the site plan Drawing No. 1. The vertical holes were advanced through the masonry and rubble fill into the underlying bedrock. A minimum of 1.5m (5 ft) of the bedrock was core drilled in each of the vertical holes. The purpose of the vertical holes was to determine the depth and condition of the masonry, and the type and quality of the underlying bedrock. The horizontal holes were drilled to determine the cross sections of the monoliths and the walls of the locks. For this purpose, a set of two horizontal holes was drilled in each of the lock walls, and a set of three horizontal holes was drilled in each of the monoliths.

The second phase of the fieldwork consisted of performing in-situ permeability tests and trial grouting test program.

All the boreholes were advanced by core drilling and casing. Continuous monitoring of wash water return and drill behavior was recorded during drilling, and particular attention was paid to any sudden drops of drilling rods indicating voids or soft zones. In addition, the inner and outer faces of the lock walls and monoliths were monitored continuously for any seepage of drill water. Photographs were also taken during the course of the fieldwork.

The borehole locations were established in the field by Trow Ontario Ltd. Their elevations were determined relative to a bench mark (# 753-830A) located at the top of the west monolith between Locks Nos. 47 and 48. The elevation of the bench mark was supplied by Public Works Canada as 86.0m geodetic.

All the cores were examined in the field, logged and placed in core boxes and identified by the Trow representative on site. On completion of the fieldwork, all the core boxes were transported to the Trow laboratory in the City of Nepean, Ontario. The samples were visually examined by a geotechnical engineer and laboratory testing assigned. Laboratory testing consisted of performing unit weight, unconfined compressive strength, 24 hours cold water and 5 hours boiling water absorption, and freeze and thaw tests on selected masonry stone samples.

BOREHOLE INSTALLATIONS

a) Standpipes

Monitoring of the water level in the locks was carried out continuously during the course of the fieldwork by means of a standpipe installed in Borehole No. VI. The standpipe consisted of 15mm (PVC pipe) with the bottom 1.5m length slotted and wrapped with a filter fabric. The standpipe was removed from the hole at the end of the fieldwork and the borehole grouted. Water level records are shown on the log of Borehole No. VI.

b) Sleeve Pipes

Trial grouting and hydraulic conductivity, tests were conducted in Borehole Nos. V2, V3, V4 and V6 in which sleeve pipes were installed. The sleeve pipes supplied in 3m length consisted of 25mm diameter, rigid PVC pipes with perforations at a spacing of 333mm along the pipe. The perforations are fitted with external rubber sleeves. Four 7mm diameter grout injection holes are positioned under each rubber sleeve.

The installation of the sleeve pipes in the holes was carried out with great care. By examination of the core of each hole, the engineer on site delineated zones of different permeability. These zones were separated for testing by placing HPSP bags (called barriers) at different locations along the sleeve pipe. The locations of these barriers were logged carefully in the field book. The barriers were screwed to the pipes using punch lock clamps.

On completion of installation of the barriers, the sleeve pipes were lowered into the holes in 3m section and glued together. The sleeve pipes were filled with water to prevent them from floating.

c) **Steel Pipes**

A 25mm diameter by 0.3m long steel pipe was installed in each of the horizontal holes with approximately 100mm of the pipe protruding into the locks. Cement mortar was used to install the pipe and seal the hole. In areas where the horizontal holes day lighted, the back end of the hole was also sealed with cement grout.

TRIAL GROUT TEST PROGRAM

A trial grout test program was undertaken. It consisted of grouting all the horizontal and vertical holes drilled as part of the investigation. A high speed colloidal mixer was used to mix the various grout formations. The results of the trial grout testing are summarized below.

Test Hole H-1

Location: Lock 47, Middle of West Wall; 0.9m from top of wall
Hydraulic Conductivity: + 100 Lugeons
Grout Formulation: 70 litres water
1 kg napthalene sulphonate
4 kg bentonite
70 kg Type F flyash
80 kg Type 10 portland cement
Batch Size: 135 litres
Marsh Value: 42 seconds
Quantity Grouted: 2 batches
270 litres

Results: This holes took large quantities of water during hydraulic conductivity testing. After grouting two batches, however, it was noticed that grout was appearing on the lawn adjacent to the lock and grouting was terminated.

Test Hole H-2

Location: Lock 47, Middle of West Wall; 0.9m from bottom of wall

Hydraulic Conductivity: + 100 Lugeons

Grout Formulation:
70 litres water
1 kg napthalene sulphonate
4 kg bentonite
70 kg Type F flyash
80 kg Type 10 portland cement

Batch Size: 135 litres

Marsh Value: 42 seconds

Quantity Grouted: 3 batches
400 litres

Results: Grouting took place at low pressure with very little evidence of leaking into the lock; grout likely penetrated into the soil. This hole was terminated without reaching refusal pressure.

Test Hole H-3

Location: West monolith between Locks 47 and 48, near top of wall

Hydraulic Conductivity: + 100 Lugeons

Grout Formulation:
92 litres water
0.5 kg napthalene sulphonate
1 kg bentonite
105 kg Type C flyash
40 kg Type 10 portland cement

Batch Size: 125 litres

Marsh Value: 38 seconds

Quantity Grouted: 6 batches
750 litres

Results: This hole encountered open joints connecting to the adjacent flood gate and air shaft. The grout travelled well beyond the repointed area. Due to internal leaks into the air shaft, this hole did not reach refusal pressure.

Test Hole H-4

Location: West monolith between Locks 47 and 48, midway down wall
Hydraulic Conductivity: + 100 Lugeons
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
1 kg bentonite
105 kg Type C flyash
40 kg Type 10 portland cement
Batch Size: 125 litres
Marsh Value: 38 seconds
Quantity Grouted: 3 batches
375 litres

Results: This hole encountered open joints connecting to the adjacent flood gate and air shaft. The grout travelled well beyond the repointed area. Due to internal leaks into the air shaft, this hole did not reach refusal pressure.

Test Hole H-5

Location: West monolith between Locks 47 and 48, bottom of wall
Hydraulic Conductivity: + 100 Lugeons
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
1 kg bentonite
105 kg Type C flyash
40 kg Type 10 portland cement
Batch Size: 125 litres
Marsh Value: 38 seconds
Quantity Grouted: 5 batches
625 litres

Results: The grout travelled well beyond the repointed area, moving through the abutment towards the access ladder, a spreadout distance of 6 metres. This hole did not come to refusal, even though there was no indication of any leakage into the air shaft or floor way.

Test Hole H-6

Location: Lock 48, Middle of West Wall, 0.9m from top of wall
Hydraulic Conductivity: 40 Lugeons
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
4 kg Bentonite
70 kg Type F flyash
40 kg Type 10 portland cement
Batch Size: 135 litres
Marsh Value: 62 seconds
Quantity Grouted: 1.5 batches
200 litres

Results: Grouted at 10 - 15 psi pressure and rate of injection at 10 litres per minute. A major leak occurred at the bottom joint after approximately 200 litres had been injected. Grout travelled approximately 5 metres downhill to the point of escape through wide open joints approximately 7 metres downhill of the grout hole. Upon leaking, the grout was very runny and finally turned to clear water. It appears that the grout had found a pathway at the interface between the rock and the structure, and that such a path exists over a substantial surface.

Test Hole H-7

Location: Lock 48, Middle of West Wall, 0.9m from bottom of wall
Hydraulic Conductivity: 60 Lugeons
Grout Formulation: 85 litres water
8 kg bentonite
70 kg Type F flyash
40 kg Type 10 portland cement

Batch Size: 135 litres
Marsh Value: 82 seconds
Quantity Grouted: 2 batches
270 litres

Results: Prior to grouting H-7, hydrophobic water reactive prepolymer was installed in a number of open joints using absorbent grouting pads and wooden wedges, covering a horizontal distance approximately 15 metres downstream and 10 metres upstream. The bottom joint between the wall and floor was repointed with a fast setting cement mortar until all of the water had been channelled through test hole H-7, as well as a small drain pipe installed approximately 9 metres downstream of H-7 and 0.3 metres above the floor of the lock.

The grout for test hole H-7 was formulated to have the consistency of a very thick toothpaste, almost like a mortar, for the purpose of filling some large voids. All of the water previously escaping through test hole H-7 was channelled out of the drain pipe.

The hole was not brought to refusal. Grout travelled to our newly installed drain pipe. When this drain pipe was closed for a short while, water started to appear about 2 metres above the floor through the joints in the wall over a width of more than 15 metres. The valve of the drain pipe was opened to prevent destruction due to build up of hydrostatic pressures or freezing of water in the joints. The wall dried up.

Test Hole H-8

Location: West monolith between Locks 48 and 49, 0.6m from top of wall
Hydraulic Conductivity: + 100 Lugeons
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
6 kg bentonite
70 kg Type C flyash

	40 kg Type 10 portland cement
Batch Size:	200 kg
	135 litres
Marsh Value:	80 seconds
Quantity Grouted:	1.5 batches
	200 litres

Results: Grouted at low pressure (10 psi); a bit seepage was evident approximately 1 to 2m downstream along the adjacent horizontal joints. Grout broke through the top of the wall when pressure was increased to 15 psi.

Test Hole H-9

Location: West monolith between Locks 48 and 49, at mid-height of monolith
Hydraulic Conductivity:

Grout Formulation:	92 litres water
	0.5 kg napthalene sulphonate
	6 kg bentonite
	70 kg Type C flyash
	40 kg Type 10 portland cement
Batch Size:	202 kg
	135 litres
Marsh Value:	80 seconds
Quantity Grouted:	1.5 batches
	200 litres

Results: While grouting, some early leaks were sealed using dry portland cement held over the leaking areas. Grout eventually travelled 3 metres away from the hole.

Test Hole H-10

Location: West monolith between Locks 48 and 49, close to bottom of monolith
Hydraulic Conductivity: + 100 Lugeons

Grout Formulation: 85 litres water
8 kg bentonite
70 kg Type C flyash
40 kg Type 10 portland cement
Batch Size: 202 kg
135 litres
Marsh Value: 82 seconds
Quantity Grouted: 2 batches
270 litres

Results: Grouted with no leaks and no pressure indicating an open formation. It was not possible to determine whether grout was lost into lock 49, as water level was covering upstream sill.

Test Hole H-11

Location: Lock 47, Middle of East wall, 0.9m from top of wall
Hydraulic Conductivity: + 100 Lugeons
Grout Formulation: 70 litres water
1 kg napthalene sulphonate
4 kg bentonite
70 kg Type F flyash
80 kg Type 10 portland cement
Batch Size: 135 litres
Marsh Value: 42 seconds
Quantity Grouted: 3.5 batches
475 litres

Results: This hole took 3.5 batches of grout when a severe leak occurred approximately 2.5 metres downstream on the outside of the lock wall. The grout was thickened and attempts were made to patch the leak using normal portland cement and fast setting cement. When these attempts were unsuccessful, the grouting operation on this hole was abandoned.

Test Hole H-12

Location: Lock 47, Middle of West wall, 0.9m from bottom of wall
Hydraulic Conductivity: + 100 Lugeons
Grout Formulation: 70 litres water
1 kg napthalene sulphonate
4 kg bentonite
70 kg Type F flyash
80 kg Type 10 portland cement
Batch Size: 135 litres
Marsh Value: 42 seconds
Quantity Grouted: 1 batches
135 litres

Results: This hole took less than one batch before coming to refusal. It is thought that a line blockage caused the hole to come to refusal.

Test Hole H-13

Location: East monolith between Locks 47 and 48, near top of monolith
Hydraulic Conductivity: + 100 Lugeons (Connection with flood gate)
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
1 kg bentonite
105 kg Type C flyash
40 kg Type 10 portland cement
Batch Size: 125 litres
Marsh Value: 38 seconds
Quantity Grouted: 4 batches
500 litres

Results: This hole encountered open joints connecting to the adjacent flood gate and air shaft. The grout travelled well beyond the repointed area and external leaks had to be stopped using dry cement powder. Due to internal leaks into the air shaft, this hole did not reach refusal pressure.

Test Hole H-14

Location: East monolith between Locks 47 and 48, at mid-height
Hydraulic Conductivity: + 100 Lugeons (Connection with flood gate)
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
1 kg bentonite
105 kg Type C flyash
40 kg Type 10 portland cement
Batch Size: 125 litres
Marsh Value: 38 seconds
Quantity Grouted: 4 batches
500 litres

Results: This hole encountered open joints connecting to the adjacent flood gate and air shaft. The grout travelled well beyond the repointed area and external leaks had to be stopped using dry cement powder. Due to internal leaks into the air shaft, this hole did not reach refusal pressure.

Test Hole H-15

Location: East monolith between Locks 47 and 48, close to floor slab
Hydraulic Conductivity: + 100 Lugeons
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
1 kg bentonite
105 kg Type C flyash
40 kg Type 10 portland cement
Batch Size: 125 litres
Marsh Value: 38 seconds
Quantity Grouted: 250 litres

Results: As the hole was drilled through the wall into the soil on the opposite side, the grout was pumped into the soil and had no effect on the abutment structure. The grout boiled up through the soil and the borehole was plugged with fast setting cement. When the operation resumed the plug was blown out and a trough was created to allow the grout boiling up through the soil to cure and seal the drill hole.

Test Hole H-16

Location: Lock 48, Middle of East wall, 0.9m from top of wall
Hydraulic Conductivity: 60 Lugeons
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
4 kg bentonite
35 kg Type C flyash
35 kg. Type F flyash
40 kg Type 10 portland cement
Batch Size: 202 litres
Marsh Value: 60 seconds
Quantity Grouted: 2 batches
270 litres

Results: Grout travelled horizontally only in one joint causing a few small leaks which were easily stopped; did not build up any pressure with this mix. No refusal pressure obtained.

Test Hole H-17

Location: Lock 48, Middle of East wall, 4.9m from top of wall
Hydraulic Conductivity: + 100 Lugeons (hole breaks through wall into soil approximately 1.0m below soil surface)

Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
4 kg bentonite

35 kg Type C flyash
35 kg Type F flyash
40 kg Type 10 portland cement

Batch Size: 202 litres
Marsh Value: 60 seconds
Quantity Grouted: 2 batches
270 litres

Results: Grout travelled through the wall into soil; did not build up any pressure with this mix.

Test Hole H-18

Location: East monolith between Locks 48 and 49, near top of wall
Hydraulic Conductivity: + 100 Lugeons
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
4 kg bentonite
35 kg Type C flyash
35 kg Type F flyash
40 kg Type 10 portland cement

Batch Size: 202 litres
Marsh Value: 60 seconds
Quantity Grouted: 2 batches
270 litres

Results: Grout resulted in major seepages all along upper joints of abutment. One major leak occurred approximately 3m north of test hole H-18, which was stopped by applying handfuls of dry portland cement at the point of leaking. There was no pressure build up. Total estimated grout loss due to seepage was 100 litres.

Test Hole H-19

Location: East monolith between Locks 48 and 49, at mid-height

Hydraulic Conductivity: + 100 Lugeons

Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
4 kg bentonite
35 kg Type C flyash
35 kg Type F flyash
40 kg Type 10 portland cement

Batch Size: 202 kg
135 litres

Marsh Value: 60 seconds

Quantity Grouted: 2 batches
270 litres

Results: Quite a bit of seepage on the outside of the lock; easily stopped by applying dry portland cement. Grout travelled all across the abutment within a horizontal band measuring 1.5m high. Grouting initially displaced in-situ water before grout appeared at the joints.

Test Hole H-20

Location: East monolith between Locks 48 and 49, 0.6m from bottom of wall

Hydraulic Conductivity: + 100 Lugeons

Grout Formulation: 92 litres water
8 kg bentonite
70 kg Type F flyash
40 kg Type 10 portland cement

Batch Size: 135 litres

Marsh Value: 70 seconds

Quantity Grouted: 2 batches
270 litres

Results: Grouted without creating any pressure; possible connection to soil behind wall; no apparent leaks.

Test Hole V-1

Location: Lock 47, Middle of West wall
Hydraulic Conductivity: + 100 Lugeons (indicates a very open formation)
Grout Formulation: 70 litres water
1 kg napthalene sulphonate
4 kg bentonite
70 kg Type F flyash
80 kg Type 10 portland cement
Batch Size: 135 litres
Marsh Value: 42 seconds
Quantity Grouted: 4 batches
540 litres

Results: Test Hole V-1 contained part of a piezometer and was grouted as a single shot, full column test with a single inflatable packer. A significant quantity of repointing was undertaken prior to grouting. Grouting pressure slowly increased to 35 psi with small leaks as far as 4m downstream and 3m upstream from the test hole. Grouting was stopped when 35 psi pressure was reached. After grouting, it was necessary to top off the hole with grout several times as the grout continued to seep into cracks and joints.

Test Hole V-2

Location: West monolith between Locks 47 and 48
Hydraulic Conductivity: 8 Lugeons
Grout Formulation: 100 litres water
0.5 kg napthalene sulphonate
50 kg MC - 500 Microfine Cement
Batch Size: 125 litres
Marsh Value: 30 seconds

Quality Grouted: 4 batches
500 litres

Results: This test hole was divided into four separate grouting zones by selectively pressure grouting 3 - 4 litres of cement grout into predetermined sleeves which had been wrapped in a fine geotextile before installing the sleeve pipe into the hole. The "inflation" of the geotextile bags with grout, served to isolate the respective intervals of the hole for sequential grouting operations.

No grouting was performed on the bottom zone due to the sleeve pipe being blocked by a foreign object at the depth of 19m. The upper zones indicated a hydraulic conductivity value of approximately 8 Lugeon, due to the previous grouting operations through H-3, H-4 and H-5. The grout travelled over 6 metres horizontally and 3 metres vertically from the test hole and was monitored by leaks from the abutment which were sealed using dry cement. The microfine grouting operation eventually increased in pressure until reaching refusal pressure. The success of this operation indicated that it is feasible to use microfine cement to tighten up areas which have been previously grouted using portland cement.

Test Hole V-3

Location: Lock 48, Middle of West wall
Hydraulic Conductivity:
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
1 kg bentonite
70 kg Type F flyash
40 kg Type 10 portland cement
Batch Size: 135 litres
Marsh Value: 38 seconds
Quality Grouted: 3 batches
400 litres

Results: Grout ran out of many joints in the vicinity of test hole V-3, especially in the bottom joint between the wall and floor of the lock. Grouting pads soaked in polyurethane chemical grout and wooden wedges were used to seal the leaking joints. Despite the use of a thixotropic grout, grout quickly poured out of test hole H-7, located in the lock wall approximately 4 metres downstream and approximately 0.9 metres above the floor. When the valve on test hole H-7 was closed, water came out of the wall on the third joint above H-7 and also out of H-6.

Test Hole V-4

Location: Lock 47, Middle of East wall
Hydraulic Conductivity: 3 Lugeons
Grout Formulation: 100 litres water
0.5 kg napthalene sulphonate
50 kg MC - 500 Microfine Cement
Batch Size: 125 litres
Marsh Value: 30 seconds
Quality Grouted: 2.5 batches
300 litres

Results: The formation, with a hydraulic conductivity of only 3 Lugeons, was much tighter than all the other test holes. A few leaks were encountered and the hole reached refusal pressure of 50 psi after injecting 2.5 batches.

Test Hole V-5

Location: East monolith between Locks 47 and 48, approximately 0.6m from air shaft
Hydraulic Conductivity: + 100 Lugeons
Grout Formulation: 92 litres water
0.5 kg napthalene sulphonate
8 kg bentonite
105 kg Type F flyash
40 kg Type 10 portland cement

Batch Size:	145 litres
Marsh Value:	80 seconds
Quantity Grouted:	340 litres

Results: The hole was water tested for hydraulic conductivity using a single packer at the collar of the hole. Water ran through open joints on the outside of the abutment, but most of the water appeared to run into the adjacent air shaft and down into the flood gate feeding Lock 48. A very thick grout was used to keep loss of grout to a minimum. The pressure built up a few times to 30 psi falling to zero. These events coincided with seepage through open joints in the air shaft. The operation was halted when the grout broke through the horizontal surface of the lock, about 1 metre north of the hole. The hole was topped up several times to obtain complete filling.

Test Hole V-6

Location: East monolith between Locks 48 and 49

Hydraulic Conductivity:	+ 100 Lugeons
Grout Formulation:	92 litres water
	0.5 kg napthalene sulphonate
	8 kg bentonite
	70 kg Type C flyash
	40 kg Type 10 portland cement
Batch Size:	135 litres
Marsh Value:	70 seconds
Quantity Grouted:	1.5 batches
	200 litres

Results: Test hole V-6 was connected through open joints with the air shaft of the floodway and a very thick grout was pumped to fill the joints. No refusal pressure was obtained.

STRUCTURE AND GEOTECHNICAL CONDITIONS

a) Cross Sections

The horizontal holes drilled perpendicular to the face of the structure, have revealed that the thickness of the walls and monoliths vary from location to location. The cross sections of the walls and monoliths are shown in details on Drawing Nos. 2 to 4.

At Lock No. 47, the stone masonry thickness varies from 3.0m to 3.8m at the east wall, 1.94m to 3.02m at the west wall, 3.8m to 4m at the east monolith and from 2.3m to 3.9m at the west monolith. Behind the west wall and the monolith, silty clay fill was encountered in the boreholes drilled in the upper to mid height of the locks (Borehole Nos. H1, H3 and H4) and gneissic syenite bedrock in the boreholes drilled close to the floor of the lock (Borehole Nos. H2, H5). Silty clay fill was also encountered in the boreholes drilled through the east wall and monolith close to the floor of the lock (Borehole Nos. H15 and H12).

The vertical holes drilled through the walls of Lock No. 47 and the monolith between Lock Nos. 47 and 48 indicate that the lock walls and monoliths are founded on gneissic syenite or metadiabase bedrock. The height of the masonry wall at the borehole locations varied from 5.7m to 5.9m whereas the monoliths are approximately 7.4m high. The gneissic syenite bedrock consists of alternating ribbons of alkali feldspar and chlorite. It is brick red in colour and is locally jointed and orientated at 60 degrees to the core axis. The metadiabase bedrock is fine to medium grained, moderately to strongly weathered and contains minor vugs. It is moderately soft to moderately hard.

The west wall of Lock No. 48 is approximately 2m wide whereas the east wall of the lock is 3.0m to 3.6m wide. The west monolith between Lock Nos. 48 and 49 is 3.9m wide whereas the east monolith is approximately 3.7m wide. The west wall and monolith are constructed against bedrock face whereas silty clay was encountered behind the east wall and monolith close to the floor of the lock.

Vertical borehole drilled through the west wall of Lock No. 48 indicated that the wall is founded on gneissic syenite or metadiabase bedrock. The height of the masonry varies from 4.4m for the west wall to 7m for the east wall.

Vertical cross sections of the walls and monoliths of Lock Nos. 47 and 48 are shown in Drawing Nos. 5 and 6. Detailed logs of the horizontal and vertical holes are shown on Drawing Nos. 7 to 32.

Photographs taken during the course of drilling of grouting test program are appended in Appendix "A".

b) Stone Masonry

The stone masonry consists of limestone blocks bonded together with mortar. In general, the lime mortar was soft and mostly washed out during the coring process. Coarse to medium grained sand cement mortar was encountered during the coring of most of the horizontal holes. Evidence of voids and soft zones was noticed by sudden drop of the drilling rods and by the loss of the wash water in the stone masonry.

LABORATORY TEST RESULTS

Selected samples of the stone masonry were subjected to laboratory testing consisting of unconfined compressive strength, 24 hours cold water and 5 hours boiling water absorption and 50 cycles of freeze and thawing test. The results of these tests except for 50 cycle freeze and thaw test are given on Table I. The results of the 50 cycle freeze and thaw tests would be forwarded under separate cover on completion of the tests. It is noted that these results represent the properties of the intact stone. A number of cores had cracks or other discontinuities and they were rejected for testing.

A review of Table I indicates that the compressive strength of the masonry stone varies from 56.4 to 176.8 MPa with the average compressive strength being 129.8 MPa. The 24 hour cold water absorption ranged between 0.01 and 0.08 percent whereas the 5 hour Boiling Water Absorption was 0.02 to 0.10 percent. The saturation coefficient of the stone varied from 0.50 to 1.0. The Bulk Specific Gravity of the masonry stone was established to be relatively constant at 2.71 kg/l.

REHABILITATION OBJECTIVES

The working life span of natural stone lock structures, such as the Kingston Mills Locks, is dependent primarily on the hydraulic conductivity of the structures.

High rates of hydraulic conductivity or permeability lead to water flowing through the structure, with associated erosion of mortar particles and freeze/thaw damage resulting in block movement, cracking, deterioration of repointing mortar, etc. The effects of erosion and freeze/thaw damage are cumulative and in extreme situations can require removal of the old structure and complete reconstruction.

Water or moisture is always present in any hydraulic control structure, even in winter. Water migrating from upstream to downstream, from sill to sill within the locks, as well as from ground moisture, assure that a steady supply of water will be available at all times.

The principal objective of any masonry rehabilitation program is to completely fill all existing cracks, joints and voids with materials having low rates of hydraulic conductivity, thereby minimizing the effects of freeze/thaw action and eliminating erosion paths. As there is no permanent cost-effective solution against such natural causes, one has to focus on reducing the rate of deterioration to an acceptable level.

Freeze/thaw action on wet hydraulic structures which are in an advanced state of deterioration, such as at Kingston Mills, causes the stone blocks to move out of place, displaces the jointing materials, and leaves voids and seepage paths behind and throughout the walls.

The simple application of repointing materials at the face of the masonry blocks fails to treat the hidden problems associated with the voids and seepage paths which have been created over time. A proper rehabilitation program deals with the cause and not with the symptoms alone. Several successful grouting operations in Europe in similar structures indicate that filling the voids and accessible pores and crevices in these type of structures, with proper grouting techniques, drastically reduces the rate of deterioration.

The type of grouting materials required to rehabilitate the masonry structures should have the following characteristics:

- low viscosity in liquid state for improved penetrability.
- no segregation of grout particles while curing.
- balanced, stable grout formulation with good resistance against caking.
- chemically compatible with lock structure materials, and flowing water where encountered.
- low cost, commercially available materials.
- suitable for high rates of grouting performance.
- fills voids, joints, fine cracks and fissures.
- cures without shrinkage or bleeding.
- low porosity when cured.
- low hydraulic conductivity when cured.
- durable and long lasting after curing.

A combination of repointing materials and techniques, together with the selection and application of cementitious and chemical grouts is required to achieve cured materials cutting off migration paths through the entire structure.

The application of repointing materials is interdependent with the application of grouting materials. Both aspects of the rehabilitation program must be successfully completed in order for the overall result to achieve the intended level of performance.

REPOINTING OF MASONRY STRUCTURE

In order to assure complete filling of voids, joints, cracks and fissures, the cementitious grout must be very fluid when injected. Since injection pressures must be kept to a minimum, a long cure time is desirable to ensure the penetration of fine cracks is achieved. Unless the repointing work has been properly applied, there is a high probability that the liquid grout will leak out of the joints, thereby causing unnecessary delays to the rehabilitation program, and even jeopardizing the entire operation.

Repointing materials must be selected and applied to accommodate a wide range of joint conditions. (In all cases, it is necessary to clean the joint conditions). In all cases, it is necessary to clean the joints of organic matter to provide for bonding between the repointing material and the masonry blocks. Repointing work should take place sufficiently in advance of grouting in order to have achieved a satisfactory strength.

Three types of repointing mortars are suggested for use under the specified conditions:

a) For open wet joints: Select a ready-to-use, fast hardening, high strength concrete repairing compound with high bond strength, such as CPD Patching Cement (20 Minute Set). This product is well suited for typical repointing work but is formulated from coarse materials and is not suitable for fine cracks.

b) For open dry joints: Select a ready-to-use, cement-based grout containing non-ferrous fluidifiers and antishrinkage compounds, such as CPD Non-Shrink Construction Grout (Hi-Flo). This product is slower curing than CPD 20 Minute Set and provides a longer working time, but is still formulated from coarse materials and is not suitable for fine cracks.

c) For fine cracks: Prepare a blend of 2 parts by weight Type C Flyash and 1 part of Portland Cement; add Type F Flyash to blend the colour to match the limestone masonry. Add water until a mud-like consistency is achieved; the resulting product will be very sticky. Smear into fine cracks by hand and allow 24 hours to cure.

In large open joints specifically on lock sills, the use of the following "pre-grouting" technique is recommended to minimize the potential for excessive force to be exerted on a large repointing patch.

In conjunction with repointing as described above, install 1/4" threaded grout pipes in the open joint areas (preferably PVC pipes). After allowing the repointing material to cure, "pre-grout" the bulk of the openings behind the repointing using small quantities of cementitious grout under very low pressure (<5 psi). Allow the "pre-grout" material to cure before commencing normal injection work.

During the actual grouting operation, it is to be expected that grout leaks will occur through the repointing at any time and at any place. This is a normal aspect of masonry rehabilitation work. It is necessary to anticipate such events, and to be prepared to take immediate corrective action.

A plug of dry portland cement, applied over the leaking area will cause the leaking grout stream to dehydrate through capillary action, thereby thickening and plugging the seepage path inside the masonry joint. Ladders and scaffolding should be erected in the vicinity of grouting operations, with grouting staff and repair materials readily available. Quick setting cement curing in less than 2 minutes, should be used to stop persistent leaks. In these zones, where water is draining from the formation through the lock wall, one component, water reactive hydrophobic polyurethanes should be used. Grouting pads, drenched in this chemical grout should be introduced in open leaking joints, effectively blocking badly leaking openings.

DRILLING PATTERN FOR GROUT HOLES

The test grouting program indicated the superiority of vertical grout holes as compared with the use of horizontal holes. Fewer holes are required, the set-up and operation of drilling and grouting equipment is much easier, and the interconnection of seepage paths and voids and as a result, the spread out radius is superior in the vertical holes.

It is recommended to use a primary row of vertical grout holes located at the mid-point of the masonry walls, with a tentative hole spacing at 6 metres. A secondary row of grout holes will be required between the primary grouting. The drilling of this secondary row should only take place after the first row of holes has been grouted. The interspacing of the holes has to be adjusted to accommodate the theoretical spread out radius.

Special measures will be required to seal all air shafts, flood gates and drains in the vicinity of the lock doors prior to commencing cementitious grouting work. The test grouting program demonstrated that grout poured in an unconfined manner into these openings when grouting on adjacent holes. It is suggested that the installation of selected geotextile lining materials and the temporary use of inflatable air bags inside the drains and shafts may be the most cost-effective means of sealing off these areas, to prevent loss of grout during the rehabilitation program.

The sills between each lock should be drilled using three rows of vertical grout holes installed into competent rock. A staggered drilling pattern with a 3 metre spacing between holes is recommended. Grouting in the sills is essential if water "boils" are to be cut-off, thereby eliminating the accompanying loss of soil and mortar particles and resulting subsidence.

After completion of the grouting program, it will be required to drill drain holes to monitor and remove any water building up behind the rehabilitated wall. This is the case in the west wall of Lock 48. It is required to grout the west wall of Lock 48, following the normal procedures, but to install a number of drain holes extending well into the fractured rock behind the wall. These holes should be near the floor of the lock to prevent damage to boats. A drain pipe should be sealed inside the hole in the lock structure equipped with a simple one way valve system to allow the water to drain from the rock formation into the lock during the winter. This drainage system is required to prevent further deformation of this wall due to freezing.

The vertical drill holes should be 75 - 100mm in diameter, in order to accommodate the insertion of 1-1/2" PVC sleeve pipes (50mm outside diameter) for grouting purposes. It is preferable to use diamond drills for this purpose, as percussion drilling tends to clog joints and fissures with drill cuttings and impede the flow of grout from the drill hole.

Sleeve pipes should be used for grouting. Sleeve pipes are rigid perforated PVC pipes, fitted with exterior rubber sleeves over the perforations at a spacing of 333mm along the pipe. Four 7mm diameter grout injection holes are positioned under each rubber sleeve.

Lengths of sleeve pipe are jointed together, with a rubber plug fitted into the bottom of the pipe stem. When properly installed, the rubber sleeves open only under pressure and close upon release of grouting pressure.

Sleeve pipes are sealed into the vertical drill holes by installing barriers, filled with casing grout in the annulus between the sleeve pipe and the drill hole. These barriers are geotextile bags, strapped on the sleeve pipes at two carefully selected locations. This way the drill hole is divided into 3 zones which are tested. They are grouted with the most suitable formulation corresponding with the measured in-situ hydraulic conductivity of each particular zone. Normal casing grout consists of the following ingredients, in order of mixing:

- 75 litres water
- 5 kg bentonite
- 30 kg portland cement

Sleeve pipes are initially used to test hydraulic conductivity at various intervals along the hole, so that specific grout formulations may be injected according to the conditions at different depths. By flushing the sleeve pipe after each stage of grouting, the sleeve pipe can be accessed for repeated stages at a later date. It is however not absolutely required to use sleeve pipes to grout the vertical drill holes.

It is also appropriate to grout vertical holes using straddle packers. The grouting test program also demonstrated the practicality of using a single inflatable packer. With a single inflatable packer, grout injection is initially carried out from the bottom of the hole, with the packer being subsequently relocated to higher elevations within the drill hole.

Fewer problems are expected in lowering and retrieving a single inflatable packer, than using a straddle packer.

GROUT FORMULATIONS

The use of two basis grout formulations is recommended for various stages of the grouting work, with variations in viscosity being used as a function of in-situ hydraulic conductivity.

The use of a balanced portland cement grout is recommended for primary injection to seal against repointing work and to fill voids and seepage paths. The use of a microfine cement grout formulation is recommended for secondary grouting, with the objective of sealing fine cracks and crevices not accessible with the primary grouting formulation.

A typical primary grout formulation will include:

- water - as the medium and reaction partner in the hydraulic reaction with the cement
- Napthalene Sulphonate - as a deflocculator and superplasticizer
- Type C Flyash - as a thixotropic agent and artificial pozzolan
- Type F Flyash - as a retarder and antificial pozzolan
- Bentonite or silica fume - as a stabilizing and water repellant agent and to enhance the penetrability
- Portland Cement - as the binding agent.

The relative proportioning ratios will be determined on site depending on the grout viscosity dictated by the in-situ hydraulic conductivity at various stages of each drill hole.

A typical secondary grout formulation will include:

- water - as the medium
- Napthalene Sulphonate - as a deflocculator and superplasticizer
- MC 500 Microfine Cement - as the binding agent at the rate of 400 kg per cubic metre

Properly balanced stable cementitious grout will penetrate cracks as fine as 100 micron. A colloidally mixed neat cement grout, even with a high W/C factor will only penetrate cracks over 160 micron.

Microfine cement, on the other hand, prepared in a colloidal mixer and with the proper deflocculator, will penetrate cracks as fine as 30 micron.

Hence the selection of a stable, balanced cementitious grout for the primary row of grout holes and microfine cement in the secondary row.

In the west wall, in Lock 48, it might be required to inject one component hydrophobic water reactive prepolymers at the interface between the rock and the wall. A short set time is to be selected. The test program indicated easy travel of stable cementitious grout along the interface and through the lock wall. These travel paths have to be cut-off with fast setting prepolymer grout to prevent movement of the structure during the regular grouting program with cementitious grout. The cement grout has to be water repellant and buffered against dilution to prevent the formation of bleed pockets or channels.

GROUT PREPARATION AND MIXING

In order to achieve optimum performance from all the grouting materials and formulations, it is imperative that the grouting materials be mixed in the correct sequence and blended in a colloidal (high speed) mixer for the appropriate time periods. The use of old-style slow-speed paddle mixers is not an acceptable grouting practice.

The principle difference between colloidal and paddle mixers is the speed of rotation and the nature of the mixing element:

- colloidal mixers operate at 1500 - 2000 rpm and use an externally-mounted, high speed, vortex type mixing pump without requiring an internal mixing element.

- paddle mixers rotate an internal mixing element at 100 rpm or less directly within the mixing tank.

The colloidal mixing pump creates high turbulence and a high shearing action within the pump casing. This violent action repeatedly separates and subsequently impinges adjacent grout and water particles, thus achieving full hydration in a short mix time.

The high capacity of most mixing pumps allow the contents of the mixer to be recirculated 4 - 5 times per minute. Large clearances between the impellers and pump casing allows slugs of grout material to pass through the mixer without clogging the pump.

The trajectory of the recirculated grout mix sets up a cyclone vortex within the mixing tank which functions as a centrifugal separator. Thicker portions of the mix are spun to the outside of the tank by centrifugal force.

Colloidal mix tanks features an eccentric cone shaped bottom to direct the thicker material by gravity to the inlet of the mixing pump. A homogeneous and fully hydrated grout is normally achieved after recirculating for 4 - 6 minutes in a colloidal mixer.

A colloidally mixed grout when cured will be light in colour and density, and should be free of colour banding normally associated with the inferior mixes from a paddle mixer, and caused by the segregation of coarse cement particles. Grout prepared in a colloidal mixer will be capable of penetrating finer cracks than grout from a paddle mixer.

GROUTING PLANT AND EQUIPMENT

The following components of grouting equipment are recommended in order to maintain a consistently high level of quality control from batch to batch:

- colloidal mixer as previously described
- agitator tank for surge control between mixer and pump
- Moyno-type screw pump or Hany-type piston pump
- delivery and return lines between pump, grout hole and agitator
- single inflatable packer system
- pressure regulating devices
- grouting pressure and flow recording equipment (x-y recorder)
- Marsh Funnel to measure grout viscosity
- Baroid Mud Balance to measure grout density

The individual components of the grouting plant should be appropriately sized for smooth and efficient operation. Particular attention must be given to the capability of the plant to respond to operating controls at very low injection pressures.

Grouting pressures will start as low as 5 psi and be gradually increased as dictated by grouting performance in the field. Under no circumstances should effective injection pressures be allowed to exceed 35 psi.

Multiple grouting procedures should be considered "feeding" a number of "slow" holes simultaneously. The headlosses through the grout lines and header have to be established prior to the grouting operation for various flows (graph), to determine the effective grouting pressure.

It is required to have an automated monitoring system, to record the flow and the pressure and the accumulative flow. This computerized device prevents cheating or miscalculation of consumed grouts and provides the engineer with accurate data. The recording device prints out the data every minute, and the operator only enters the formulation used at a particular time. This way, all objective and relevant information for payment of quantities injected are available at any time.

RECOMMENDATION FOR THE TENDER

In order to obtain honest and realistic bids, it appears that the rehabilitation project would have to be split into a number of well defined activities:

- Mobilization and demobilization : L.S.
- Hoarding and heating of situ: L.S.
- Specialized grouting and monitoring equipment for the duration of the project: L.S.
- Repointing: per m^2 of lock wall
  - ordinary
  - in conjunction with "backwall" grouting
- Installation of drill holes: per lineal metre of hole
  - with sleeve pipes and barriers
  - with single inflatable packer
- Injection of grout: payment per cubic metre of grout
  - specify 5 types of stable cement formulations and develop an "equation" based on the solids per cubic metre of grout to translate unit prices from one formulation to another, and anything in between
- Per cubic metre of microfine cement installed
- Per litre of prepolymer installed
- Special provisions to prevent loss of grout in air shaft with inflated bags: L.S.
- Installation and sealing of drain pipes in west wall of Lock 48 per lineal metre.

Respectfully Submitted,
TROW ONTARIO LTD.

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AAN/SKA/jlc
Encls.

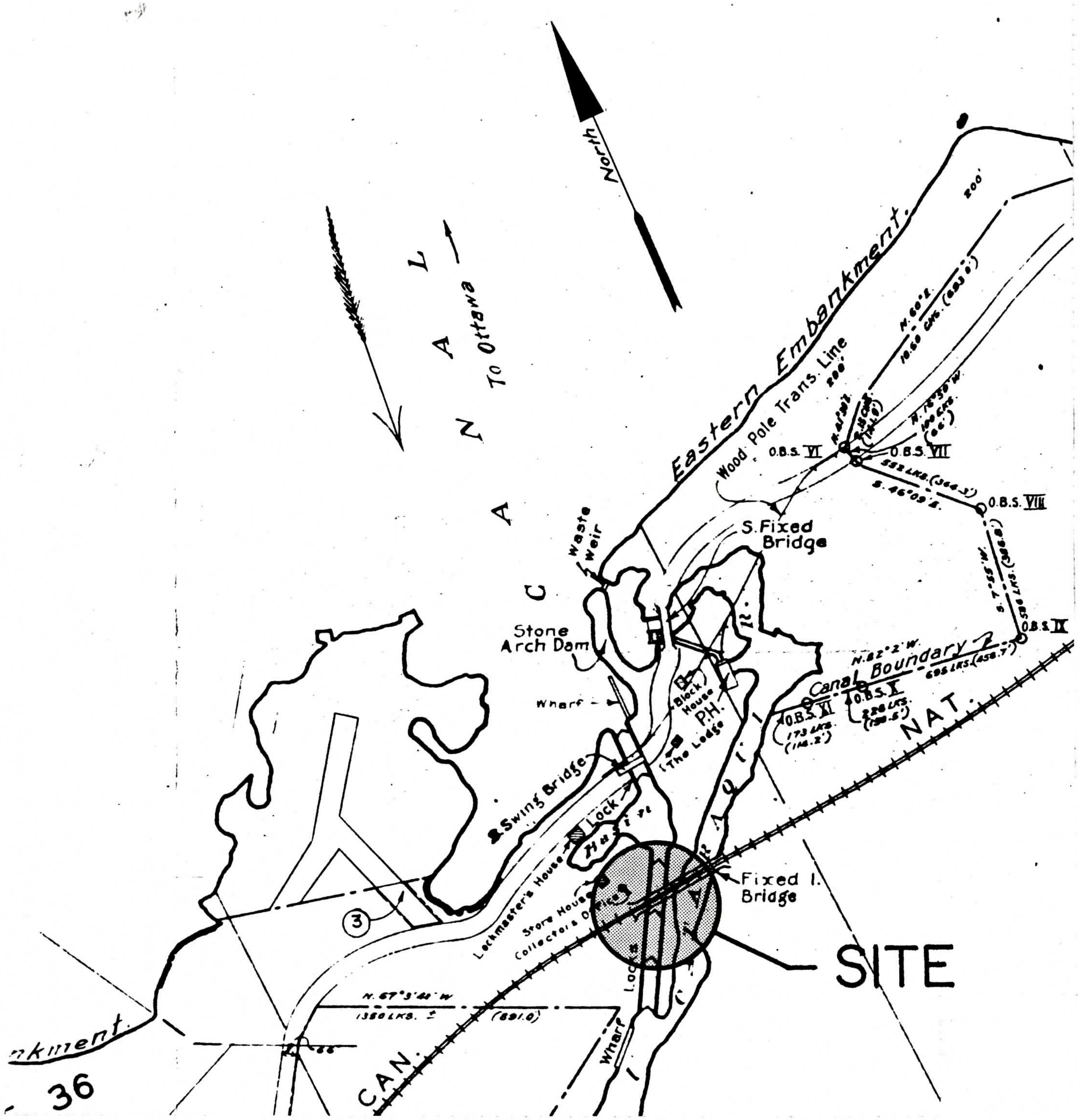
TABLE 1

RESULTS OF COMPRESSIVE STRENGTH 24 HOUR COLD WATER AND 5 HOUR BOILING WATER ABSORPTION TESTS ON STONE MASONRY

HOLE NO.	H-1	H-3	H-6	H-7	H-10	H-20	
Hole location	West wall Lock 47 1.07m below coping	West monolith between Locks 47 and 48 0.9m below coping	West wall Lock 47 0.9m below coping	West wall Lock 48 4.98m below coping	West monolith between Locks 48 and 49 6.3m below coping	East monolith between Locks 48 and 49 0.6m above lock bottom	
TEST PERFORMED	**TEST RESULTS**						**Average**
Compressive Strength (MPa)	176.8	56.4	172.1	91.7	152.0	-	129.8
Absorption Test Cold Water Absorption Percent	0.06	0.01	-	0.02	0.07	0.08	0.05
Absorption 5 Hour Boiling Absorption Percent	0.08	0.03	-	0.02	0.08	0.10	0.06
Saturation Co-efficient	0.60	0.50	-	1.00	0.82	0.79	0.74
Bulk Specific Gravity	2.72	2.71	-	2.71	2.71	2.71	2.71

NOTE:

1. All tests were conducted in accordance with methods of CSA A82.2-M78.



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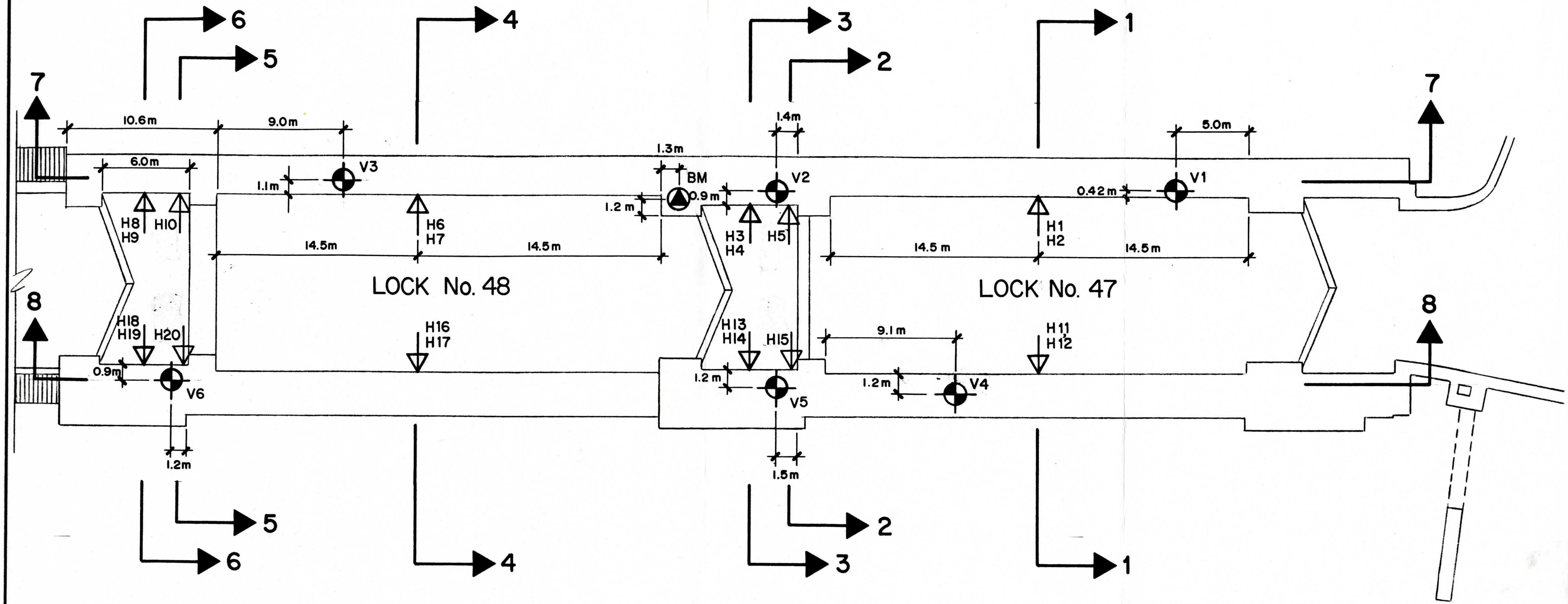
Trow Ontario Ltd.

GEOTECHNICAL INVESTIGATION	Proj. No. R-00404A/GE
	Scale 1" = 400'
	Drwn. by T.J.S.
KINGSTON MILLS LOCKS KINGSTON, ONTARIO SITE LOCATION PLAN	Appr. by S.K.A.
	Revised
	Date JANUARY, 1990
	DWG. 1



PLAN

SCALE : N.T.S.

LEGEND:

H2 HORIZONTAL HOLE

V2 VERTICAL HOLE

BM BENCHMARK GEODETIC No. 753 803A

LICENSED PROFESSIONAL ENGINEER
S. K. AGGARWAL
PROVINCE OF ONTARIO

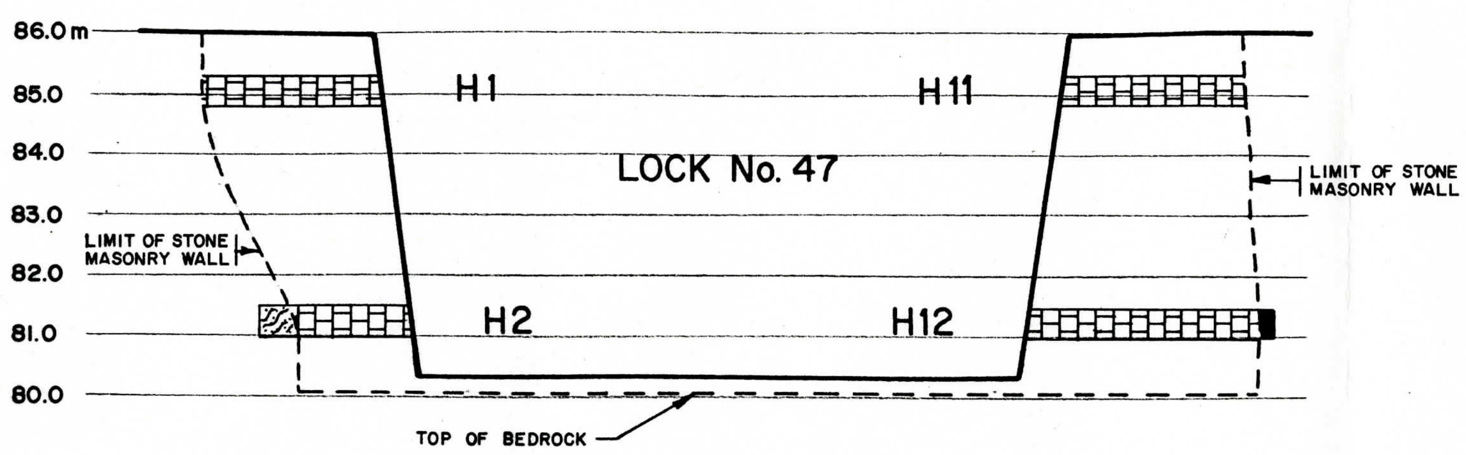
Trow Ontario Ltd.

GEOTECHNICAL INVESTIGATION

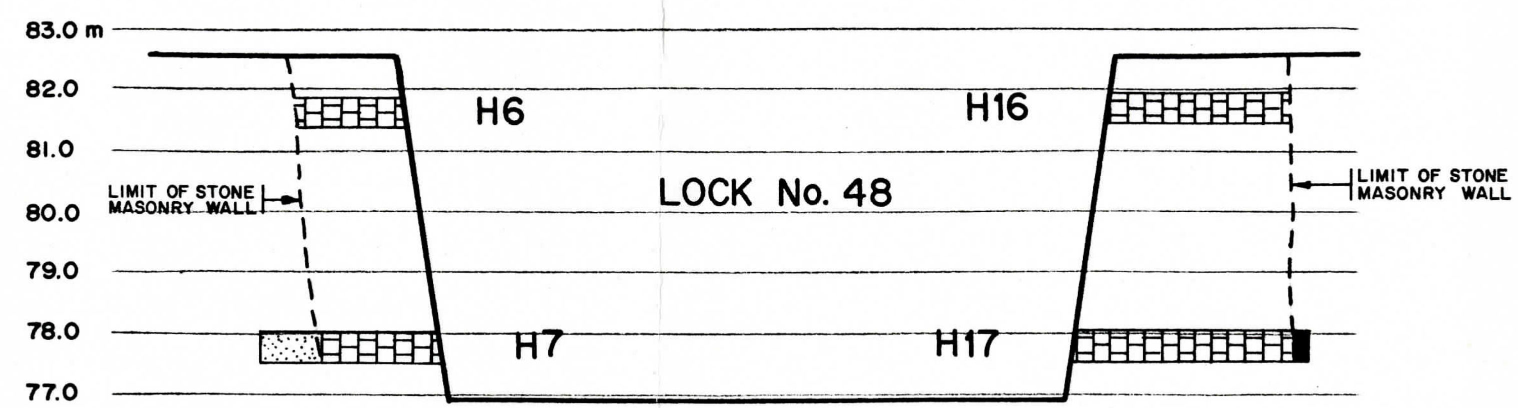
KINGSTON MILLS LOCKS
KINGSTON, ONTARIO
LOCKS 47 & 48

Proj. No.	R-00404A/GE
Scale	AS SHOWN
Drwn. by	T.J.S.
Appr. by	S.K.A.
Revised	
Date	JANUARY, 1990

DWG. 1A



SECTION 1-1
SCALE : N.T.S.



SECTION 4-4
SCALE : N.T.S.

LEGEND

- STONE MASONRY
- GNEISSIC SYENITE BEDROCK
- METADIABASE BEDROCK
- SOIL BACKFILL

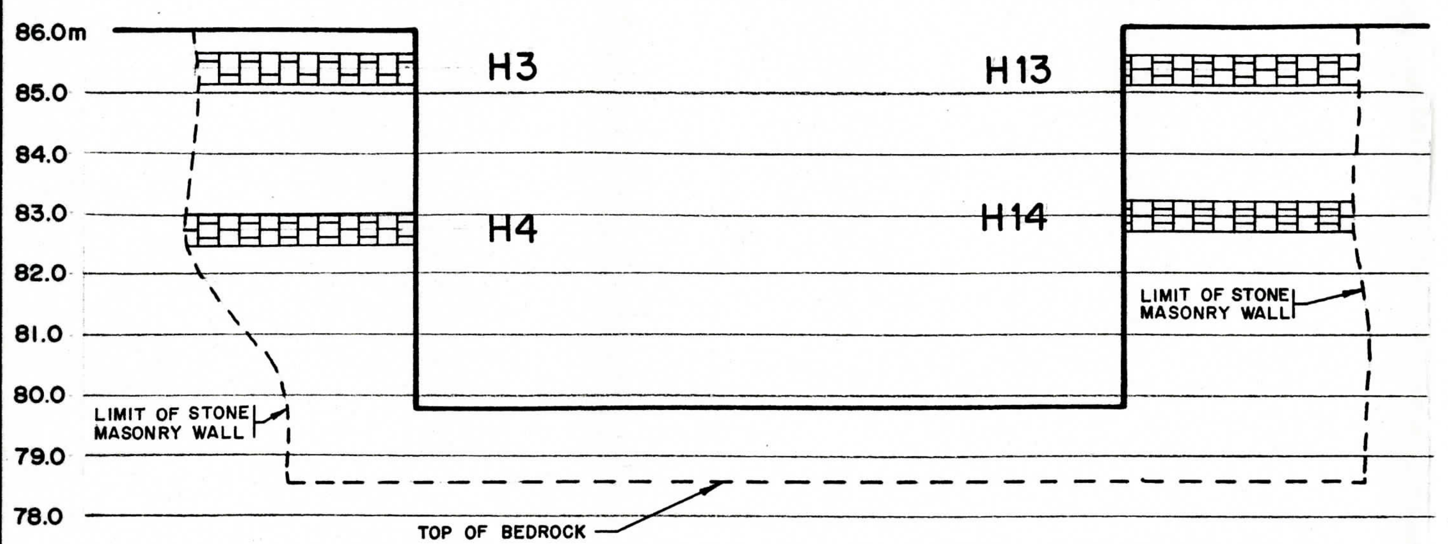
LICENSED PROFESSIONAL ENGINEER
S. K. AGGARWAL
PROVINCE OF ONTARIO

Trow Ontario Ltd.

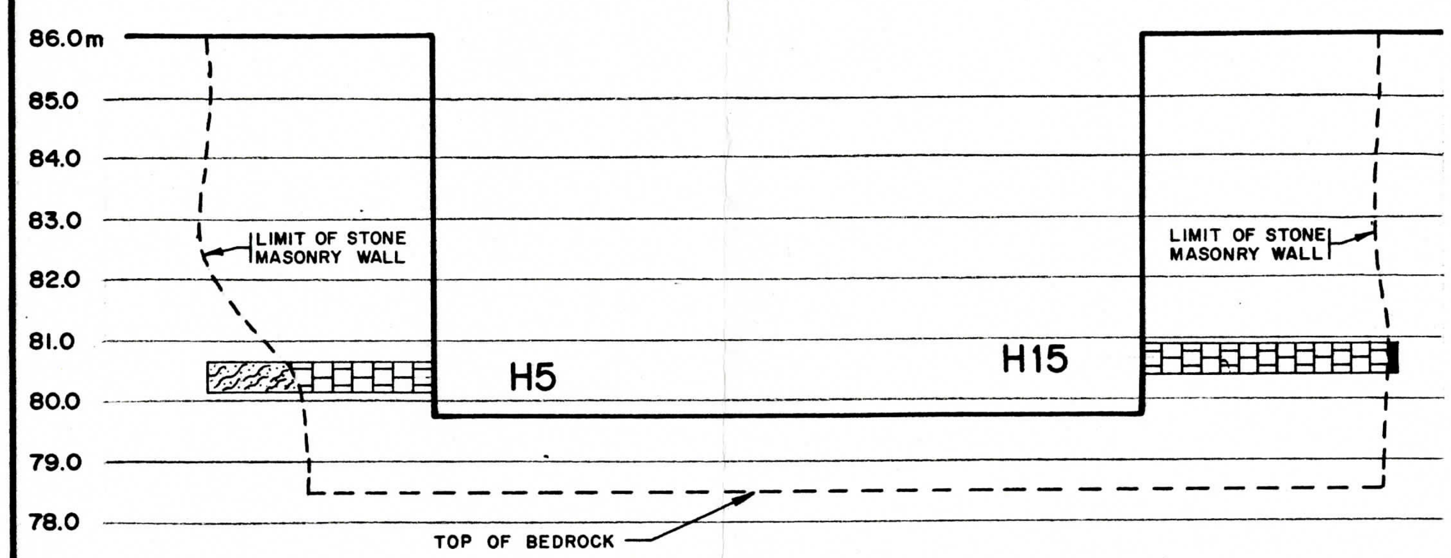
GEOTECHNICAL INVESTIGATION

KINGSTON MILLS LOCKS
KINGSTON, ONTARIO
LOCKS 47 & 48

Proj. No.	R-00404A/GE
Scale	AS SHOWN
Drwn. by	T.J.S.
Appr. by	S.K.A.
Revised	
Date	JANUARY, 1990
DWG. 2	



SECTION 3-3
SCALE : N.T.S.



SECTION 2-2
SCALE : N.T.S.

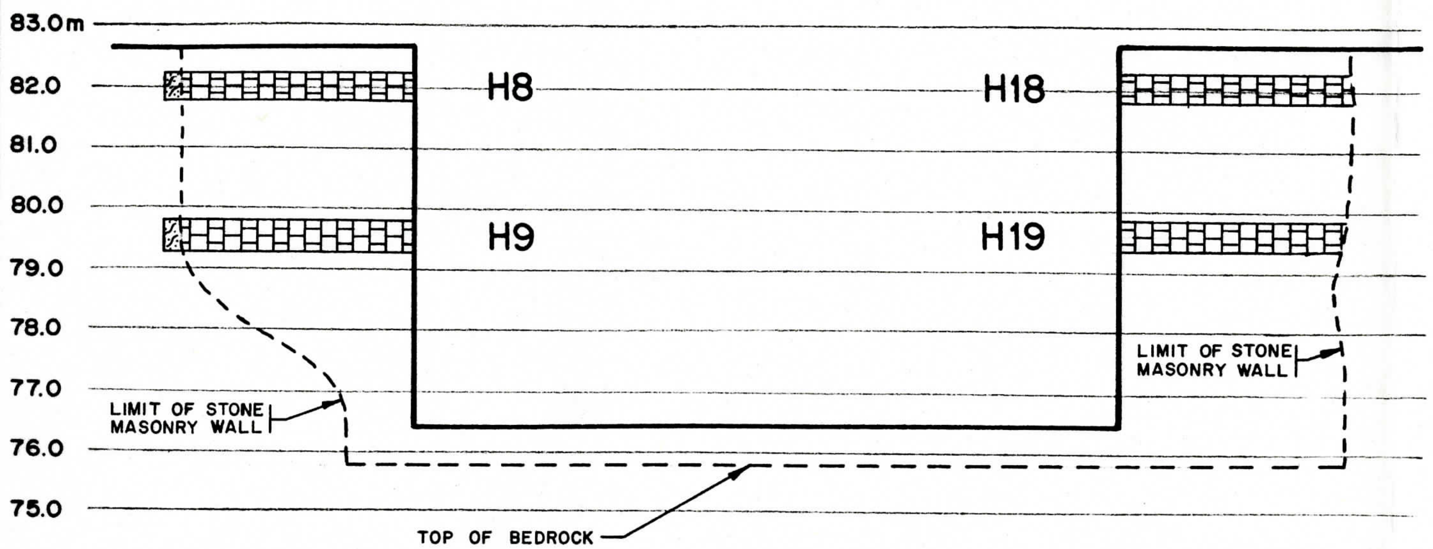
LEGEND

- STONE MASONRY
- GNEISSIC SYENITE BEDROCK
- METADIABASE BEDROCK
- SOIL BACKFILL

LICENSED PROFESSIONAL ENGINEER
S. K. AGGARWAL
PROVINCE OF ONTARIO

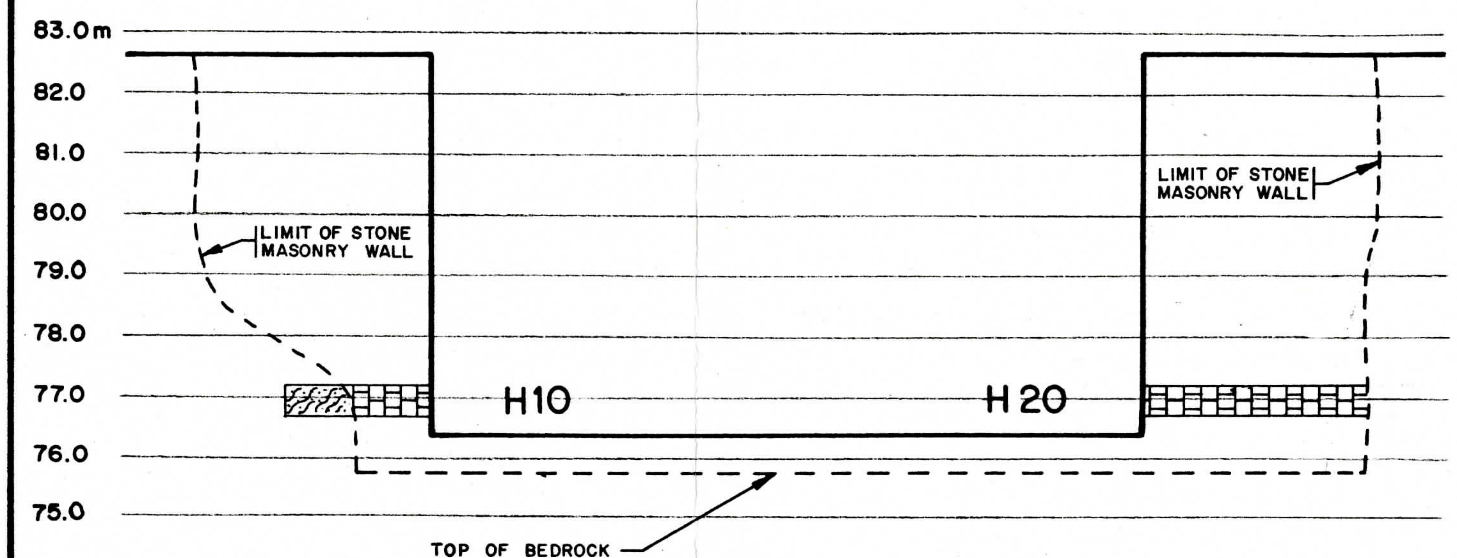
Trow Ontario Ltd.

GEOTECHNICAL INVESTIGATION	Proj. No.	R-00404A/GE
	Scale	AS SHOWN
	Drwn. by	T.J.S.
KINGSTON MILLS LOCKS KINGSTON, ONTARIO MONOLITH BETWEEN LOCK 47 & 48	Appr. by	S.K.A.
	Revised	
	Date	JANUARY, 1990
	DWG. 3	



SECTION 6-6

SCALE : N.T.S.



SECTION 5-5

SCALE : N.T.S.

LEGEND

- STONE MASONRY
- GNEISSIC SYENITE BEDROCK
- METADIABASE BEDROCK
- SOIL BACKFILL

LICENSED PROFESSIONAL ENGINEER
S. K. AGGARWAL
PROVINCE OF ONTARIO

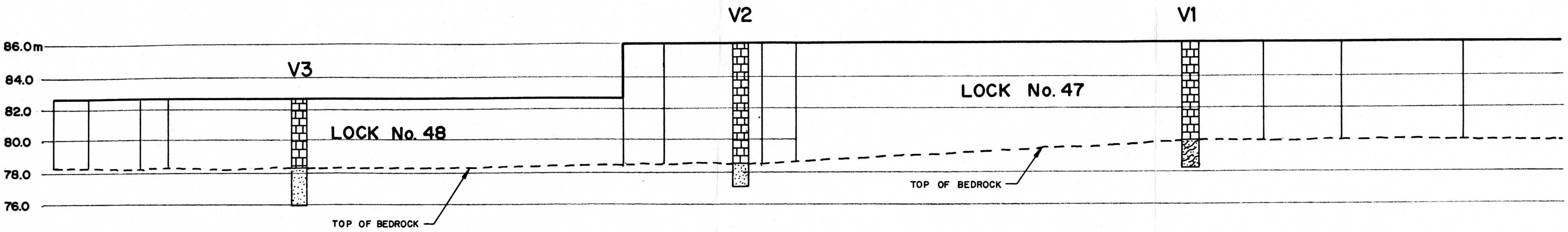
Trow Ontario Ltd.

GEOTECHNICAL INVESTIGATION

KINGSTON MILLS LOCKS
KINGSTON, ONTARIO
MONOLITH BETWEEN LOCK 48 & 49

Proj. No.	R-00404A/GE
Scale	AS SHOWN
Drwn. by	T.J.S.
Appr. by	S.K.A.
Revised	
Date	JANUARY, 1990

DWG. 4



SECTION 7-7

SCALE : N.T.S.

LICENSED PROFESSIONAL ENGINEER
S. K. AGGARWAL
PROVINCE OF ONTARIO

LEGEND

- STONE MASONRY
- GNEISSIC SYENITE BEDROCK
- METADIABASE BEDROCK
- SOIL BACKFILL

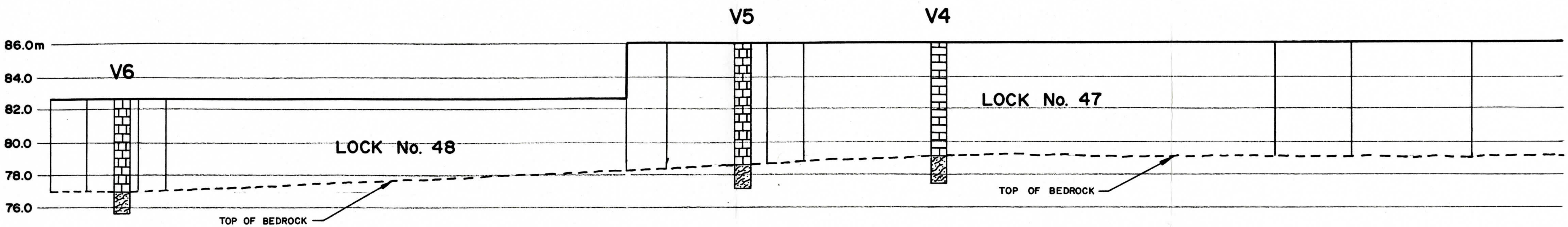
Trow Ontario Ltd.

GEOTECHNICAL INVESTIGATION

KINGSTON MILLS LOCKS
KINGSTON, ONTARIO
WEST WALL SECTION

Proj. No.	R-00404A/GE
Scale	AS SHOWN
Drwn. by	T.J.S.
Appr. by	S.K.A.
Revised	
Date	JANUARY, 1990

DWG. 5



SECTION 8-8

SCALE : N.T.S.

LICENSED PROFESSIONAL ENGINEER
S. K. AGGARWAL
PROVINCE OF ONTARIO

LEGEND

- STONE MASONRY
- GNEISSIC SYENITE BEDROCK
- METADIABASE BEDROCK
- SOIL BACKFILL

Trow Ontario Ltd.

GEOTECHNICAL INVESTIGATION	Proj. No. R-00404A/GE
	Scale AS SHOWN
	Drwn. by T.J.S.
KINGSTON MILLS LOCKS KINGSTON, ONTARIO EAST WALL SECTION	Appr. by S.K.A.
	Revised
	Date JANUARY, 1990
	DWG. 6

Log of Borehole H1

Trow

Legend	Symbol	Legend	Symbol
Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▨	Plastic and Liquid Limit	├──○
Dynamic Cone Test	———	Undrained Triaxial at Overburden Pressure % Strain at Failure	0, 5, 10, 15
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks — Dwg. No: 7

West wall, Lock 47, 1.07m below coping at centre mid length of walls.

Kingston, Ontario — Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Dist. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - grey, limestone masonry, weathered with lime mortar joints, mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 10)	3.0	0–3				
		FILL - silty clay, brown, traces gravel.	3.2	3				
		END OF HOLE						

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	84.9	Geotetic	H1
Location Kingston , Ontario	**Date Started**		**Logged By** I. Taki	**Drawing: 7A**
Client Public Works Canada	**Drilling Agency** Geotechnical Services	**Drill** Electric	**Core Barrel & Bit Design** AW	**Project Number** R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall Description
1	0	0.33	100		90	- Weathered and fractured grey limestone masonry.
2	0.33	0.68	82		100	- Limestone masonry, mortar washed out from 0.46m to 0.50m. Soft zone from 0.56m to 0.68m.
3	0.68	0.78	94		90	- Limestone masonry with mortar.
4	0.78	1.15	95		90	- Limestone masonry with mortar.
5	1.15	1.45	91			- Fractured limestone masonry with mortar.
6	1.45	1.52	94		90	- Limestone masonry - lost wash water from 1.49m to 1.52m.
7	1.52	1.65	100		40	- Limestone masonry - lost wash water from 1.52m to 1.55m and from 1.60m to 1.65m.
8	1.65	1.98	98		65	- Limestone masonry with mortar washing out.
9	1.98	2.24	60		30	- Fractured limestone masonry with coarse grained mortar joints.
10	2.24	3.02	69		50	- Generally sound limestone masonry with grey cement grout. End of Wall.
11	3.02	3.04			-	- Silty clay, fill.

NOTES:

1. Hole core drilled perpendicular to the wall face of Lock 47, uncased with a 100mm size core barrel from 0m to 0.33m and with AW size core barrel from 0.33m to termination at 3.04m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H2

Trow

Auger Sample	⊠	Natural Moisture	X	Project	Kingston Mills Locks	Dwg. No 8
SPT (N) Value	▧	Plastic and Liquid Limit	├──○		West Wall, Lock 47 - 4.87m below coping at mid length of wall.	
Dynamic Cone Test	────	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10		Kingston, Ontario	Project No. R-00404A/GE
Shelby Tube	● ● ■					
Field Vane Test	+ s	Penetrometer	▲		Hole location and datum see drawing No. 1	

GWL	symbol	Description	Dist. (m)	depth	N Value 20 40 60 80 / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight 10 20 30		Natural Unit Weight kN/cu m
		STONE MASONRY - grey, weathered limestone masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run No. 1 to Run No. 5)		0				
				1				
		GNEISSIC SYENITE BEDROCK- alternating ribbons of alkali feldspar and chlorite, brick red, locally jointed and oriented at 60° to core axis, poor quality (see Core Log Run Nos. 6 and 7)	1.9	2				
		END OF HOLE	3.0	3				
				4				
				5				
				6				
				7				
				8				
				9				
				10				

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 81.3	Datum Geodetic	Borehole No. H2
Location Kingston, Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 8A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall & Bedrock Description
1	0	0.33	100		80	Limestone Masonry
2	0.33	117	69		50	Limestone Masonry- very soft zone from 0.66m to 0.71m lost wash water from 0.66m to 0.71m
3	1.17	1.27	100		40	Limestone Masonry
4	1.24	1.84	59		40	Limestone Masonry with mortar joints. Lost wash water from 1.27m to 1.52m. Seepage of water at the inner face of wall at the joint located 0.3m above Lock Bottom at the location of drill hole.
5	1.84	1.94	100		30	Fracture limestone with granite fragments. Seepage same location as in Run No. 4. End of Wall
6	1.94	2.49	100	32	30	Gneissic Syenite Bedrock, poor quality, seepage same location as in Run No. 4.
7	2.49	2.99	100	32	0	Gneissic Syenite Bedrock, poor quality, seepage same location as Run No. 4.

NOTES:

1. Hole core drilled perpendicular to the wall face of Lock 47, uncased with a 100mm size core barrel from 0 to 0.33m, and with AW size core barrel from 0.33m to termination at 2.99m.

2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.

3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H3

Trow

Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▨	Plastic and Liquid Limit	├──○
Dynamic Cone Test	——	Undrained Triaxial at Overburden Pressure % Strain at Failure	0, 5, 10, 15
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project Kingston Mills Locks **Dwg. No** 9

West monolith between Locks 47 & 48 - 0.9m below coping

Kingston, Ontario **Project No.** R-00404A/GE

Hole location and datum see drawing No. 1

GWL	Symbol	Description	Dist. (m)	depth	N Value 20 40 60 80 / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight 10 20 30		Natural Unit Weight kN/cu m
			85.20	0				
		STONE MASONRY - grey, limestone masonry with lime mortar in joint. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 13)		1				
				2				
				3				
		FILL - silty clay, trace of gravel	3.6					
		END OF HOLE	3.65	4				
				5				
				6				
				7				
				8				
				9				
				10				

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

G 009R

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	85.2m	Geotetic	H3
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 9A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW	Project Number R-00404A/GE

Run No.	Depth (m)		Total Core Recovery	Rock Quality Designation	Wash Water Recovery (%)	Wall Description
	From	To	TCR %	RQD %		
1	0	0.32	100		80	- Limestone Masonry
2	0.32	0.44	77		80	- Limestone Masonry
3	0.44	0.56	100		90	- Limestone Masonry
4	0.56	0.76	94		0	- Limestone Masonry, fractured. Lost wash water. Seepage at the inner face of wall at the joint located 1.5m north of drill hole, 2.13m below wall coping.
5	0.76	1.02	40		0	- Limestone masonry fractured. Seepage same as in Run No. 4
6	1.02	1.13	94		0	- Fractured limestone masonry Seepage same as in Run No. 4.
7	1.13	1.60	81		80	- Limestone Masonry - mortar washed out from 1.30m to 1.34m.
8	1.60	1.70	100		80	- Limestone Masonry - mortar washed out from 1.60m to 1.63m.
9	1.70	1.84	61		80	- Limestone with mortar Mortar joints from 1.80m to 1.85m.
10	1.84	2.0	61		80	- Limestone with mortar Mortar from 1.89m to 1.96m.
11	2.0	3.0	67		80	- Limestone with mortar. Mortar from 2.03m to 2.13m.
12	3.0	3.51	78		90	- Limestone with mortar Mortar from 3.0m to 3.07m and from 3.27m to 3.30m.
13	3.51	3.68	57		90	- Limestone Masonry End of Wall

NOTES:

1. Hole core drilled perpendicular to the west monolith wall, between Locks 47 and 48 uncased, with 100mm size core barrel from 0m to 0.32m and with AW size core barrel from 0.32m to termination at 3.68m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H4

Trow

Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	├──○
Dynamic Cone Test	────	Undrained Triaxial at Overburden Pressure % Strain at Failure	(0, 5, 10, 15)
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks Dwg. No: 10

West monolith between Locks 47 & 48 - 3.25m below coping, at centre mid-length of monolith

Kingston, Ontario Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	Symbol	Description	Dist. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - grey, limestone masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 13)		0–2				
		changes at 2.89m to granite masonry with mortar	3.8	3				
		FILL - silty clay, trace gravel, brown.	3.9	4				
		END OF HOLE		5–10				

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	**Borehole Location:** (see Site Plan)	**Collar Elevation** 82.8m	**Datum** Geotetic	**Borehole No.** H4
Location Kingston, Ontario	**Date Started**	**Completed**	**Logged By** I. Taki	**Drawing: 10 A**
Client Public Works Canada	**Drilling Agency** Geotechnical Services	**Drill** Electric	**Core Barrel & Bit Design** AW	**Project Number** R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.33	100		90	- Limestone Masonry
2	0.33	0.44	88		90	- Limestone Masonry
3	0.44	0.59	100		80	- Fractured limestone masonry.
4	0.59	1.22	16		80	- Fractured limestone. Mortar washed out from 0.80m to 1.02m.
5	1.22	1.47	80		80	- Limestone Masonry. Mortar washed out from 1.43m to 1.46m.
6	1.47	2.69	66		100	- Limestone Masonry. Mortar washed out from 1.57m to 1.61m and from 1.98m to 2.0m.
7	2.69	2.89	88		100	- Limestone Masonry with grout.
8	2.89	2.96	100		90	- Granitic Masonry with Mortar.
9	2.96	3.12	77		90	- Granitic Masonry with Mortar.
10	3.12	3.28	89		70	- Granitic Masonry with Mortar.
11	3.28	3.45	100		100	- Granitic Masonry with Mortar.
12	3.45	3.88	100		100	- Granitic Masonry with Mortar.
13	3.68	3.88	75		80	- Granitic Masonry with Mortar. End of Wall

NOTES:

1. Hole core drilled perpendicular to the west monolith, at the centreline, uncased with a 100mm size core barrel from 0m to 0.33m, and with AW size core barrel from 0.33m to termination of 3.88m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H5

Trow

Auger Sample	☒	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	├──o
Dynamic Cone Test	───	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project Kingston Mills Locks **Dwg. No** 11

West monolith between Locks 47 & 48, 0.8m from lock bottom 2.5m North of centreline

Kingston, Ontario **Project No.** R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Dist. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - grey, limestone masonry with lime mortar in joints. Mortar soft and mostly washed out.		0–2				
		GNEISSIC SYENITE BEDROCK - alternating ribbons of alkali feldspar and chlorite, brick red, locally jointed and oriented at 60° degrees to core axis, poor to very poor quality	2.2	2–3				
		END OF HOLE	3.8	4–10				

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	80.5m	Geotetic	H5
Location	Date Started	Completed	Logged By	Drawing: 11A
Kingston , Ontario			I. Taki	
Client	Drilling Agency	Drill	Core Barrel & Bit Design	Project Number
Public Works Canada	Geotechnical Services	Electric	AW	R-00404A/GE

Run No.	Depth (m)		Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
	From	To				
1	0	0.25	100		90	- Limestone Masonry, fractured.
2	0.25	0.34	86		90	- Limestone Masonry, fractured.
3	0.34	0.71	83		80	- Fractured limestone masonry. Mortor from 0.53m to 0.56m.
4	0.71	0.89	100		80	- Fractured limestone masonry.
5	0.89	1.68	30		80	- Limestone with mortar. Mortar from 0.96m to 1.22m and from 1.39m to 1.68m.
6	1.68	1.90	78		80	- Limestone with mortar.
7	1.90	2.08	100		80	- Limestone and granite with mortar. Mortar from 1.90m to 1.93m.
8	2.08	2.21	100		80	- Fractured granite with mortar. End of Wall
9	2.21	2.31	50		80	- Granitic with Mortar
10	2.31	2.51	100		80	- Gneissic Syenite Bedrock, fractured, very poor quality
11	2.51	2.59	100	0	80	- Gneissic Syenite Bedrock, fractured, very poor quality
12	2.59	2.97	87	0	80	- Gneissic Syenite Bedrock, fractured, very poor quality
13	3.68	3.88	100	30	80	- Gneissic Syenite Bedrock, fractured, very poor quality
14	3.18	3.43	61	0	80	- Gneissic Syenite Bedrock, fractured, very poor quality
15	3.43	3.76	58	0	80	- Gneissic Syenite Bedrock, fractured, very poor quality

NOTES:

1. Hole core drilled perpendicular to the face of the west monolith between Locks 47 and 48, uncased with a 100mm size core barrel from 0m to 0.25m and with AW size core barrel from 0.25m to termination at 3.76m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H6

Trow

Auger Sample	☒	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	├──○
Dynamic Cone Test	———	Undrained Triaxial at Overburden Pressure % Strain at Failure	0, 15, 5, 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks — Dwg. No 12

West wall of Lock 48, 0.9m below coping at mid length of wall.

Kingston, Ontario — Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	Symbol	Description	Dist. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - limestone and granitic masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 9).		0–2				
		END OF HOLE	2.0	2				

Depth scale: 0 1 2 3 4 5 6 7 8 9 10

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 81.7m	Datum Geodetic	Borehole No. H6
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 12A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.33	100		90	- Fractured limestone masonry
2	0.33	0.61	82		90	- Limestone masonry with grout.
3	0.61	0.94	46		60	- Limestone with mortar. Lost wash water from 0.68m to 0.94m.
4	0.94	0.99	100		60	- Limestone with granite fragments.
5	0.99	1.27	96		90	- Limestone masonry, fractured.
6	1.27	1.32	100		90	- Limestone masonry with grout.
7	1.32	1.47	83		80	- Limestone masonry with grout.
8	1.47	1.80	69		80	- Limestone masonry.
9	1.80	2.0	50		80	- End of wall.

NOTES:

1. Hole core drilled, perpendicular to the west wall of Lock 48, 0.9m below coping, uncased with a100mm core barrel from 0 to 0.33m and with AW size core barrel from 0.33m to termination at 2.0m.

2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.

3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length of the total core run expressed as a percentage.

Log of Borehole H7

Trow

Legend	Symbol	Legend	Symbol
Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	├──○
Dynamic Cone Test	──	Undrained Triaxial at Overburden Pressure % Strain at Failure	0, 15, 5, 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks — Dwg. No: 13

West wall of Lock 48, 4.98m below coping at mid point of wall

Kingston, Ontario — Project No.: R-00404A/GE

Hole location and datum see drawing No. 1

GWL	Symbol	Description	Dist. (m)	depth	N Value (20, 40, 60, 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10, 20, 30)	Natural Unit Weight kN/cu m
		STONE MASONRY - limestone and granite masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 11).		0			
		METASCDIMEUT TO DIORITE- fractured, dark grey, probable boulder. (see Core Log Run Nos. 11 to 16).	2.0	2			
		END OF HOLE	3.0	3			

O 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	77.6m	Geodetic	H7
Location Kingston , Ontario	**Date Started**	**Completed**	**Logged By** I. Taki	**Drawing:** 13A
Client Public Works Canada	**Drilling Agency** Geotechnical Services	**Drill** Electric	**Core Barrel & Bit Design** AW and EW	**Project Number** R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.29	100		90	- Limestone masonry.
2	0.29	0.42	95		80	- Limestone masonry
3	0.42	0.48	100		80	- Limestone masonry.
4	0.48	0.64	25		80	- Limestone masonry with mortar. Mortar washed out from 0.53m to 0.56m and from 0.61m to 0.64m.
5	0.64	0.72	100		80	- Granite masonry with mortar.
6	0.72	0.80	100		80	- Granite masonry with mortar. Mortar from 0.76m to 0.8m.
7	0.80	1.13	100		80	- Granite masonry with mortar. Mortar from 1.07m to 0.8m. Seepage at the inner face of wall, at the joint located 0.30m above the bottom of lock.
8	1.13	1.4	75		80	- Limestone and granite masonry with mortar.
9	1.4	1.60	87		80	- Limestone masonry with grout.
10	1.60	1.98	38		80	- Limestone fragments and mortar.
11	1.98	2.18	50	0	70	- Dark grey metascdimeut, fractured, very poor quality (probable boulder) (see Core Log Run Nos. 11 to 16)
12	2.18	2.28	75	0	70	- Dark grey metascdimeut, fractured, very poor quality (probable boulder)
13	2.28	2.44	83	0	70	- Dark grey metascdimeut, fractured, very poor quality (probable boulder)
14	2.44	2.59	80	10	70	- Dark grey metascdimeut, probable boulder.
15	2.59	2.74	80	20	70	- Dark grey metascdimeut, probably boulder.
16	2.74	2.99	90	20	80	- Dark grey metascdimeut, probable boulder.

NOTES CONTINUED ON NEXT PAGE:

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation	Datum Geodetic	Borehole No. H7
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 13A (cont.)
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
						NOTES: 1. Hole core drilled, perpendicular to the west wall of Lock 48, 4.98m below coping, uncased using a 100mm core barrel from 0 to 0.29m and with AW size core barrel from 0.29m to termination at 2.0m. 2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage. 3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length of the total core run expressed as a percentage.

Log of Borehole

H8

Trow

Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	⊢—○
Dynamic Cone Test	——	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project Kingston Mills Locks Dwg. No 14

West Monolith between Locks 48 & 49, 0.6m below coping at centreline

Kingston, Ontario Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Dist. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - limestone and granite masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 11).		0–3				
		GNEISSIC SYENITE BEDROCK - alternating ribbons of alkali feldspar and chlorite, brick red, locally jointed and oriented at 60° degrees to core axis (good quality)	3.9	4				
		END OF HOLE	4.2					

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 82.01m	Datum Geodetic	Borehole No. H8
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 14A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m)		Total Core Recovery	Rock Quality Designation	Wash Water Recovery (%)	Wall and Bedrock Description
	From	To	TCR %	RQD %		
1	0	0.33	100		100	- Limestone masonry.
2	0.33	1.07	69		90	- Limestone and granitic masonry with mortar. 30% wash water return from 0.61m to 0.76m.
3	1.07	1.45	60		80	- Limestone and granitic masonry with grout
4	1.45	1.57	100		60	- Granitic masonry with mortar.
5	1.57	1.78	44		70	- Granitic masonry with mortar.
6	1.78	1.91	70		70	- Granitic masonry with mortar.
7	1.91	2.06	83		70	- Granitic masonry with mortar.
8	2.06	2.11	0		70	- No recovery.
9	2.11	2.07	76		70	- Granitic masonry with mortar.
10	2.07	3.12	78		70	- Granitic masonry with mortar.
11	3.12	3.94	45		70	- End of wall - sand washing out
	3.94	4.19	100		80	- Gneissic Syenite Bedrock, good quality.

NOTES:

1. Hole core drilled, perpendicular to the face of the west monolith wall between Locks 48 and 49, 0.60m below coping, uncased with a 100mm size core barrel from 0 to 0.33m and with AW size core barrel from 0.33m to termination at 4.19m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H9

Trow

Legend		Legend	
Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	├──○
Dynamic Cone Test	──	Undrained Triaxial at Overburden Pressure % Strain at Failure	0, 5, 10, 15
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks — Dwg. No: 15

West Monolith between Locks 48 & 49, 3.05m below coping at centreline

Kingston, Ontario — Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	Symbol	Description	Dist. (m)	depth	N Value (20, 40, 60, 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10, 20, 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - limestone and granite masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 9).		0–3				
			3.6					
		GNEISSIC SYENITE BEDROCK - alternating ribbons of alkali feldspar and chlorite, brick red, locally jointed and oriented at 60° degrees to core axis (fair quality)	4.2	4–5				
		END OF HOLE						

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 79.6m	Datum Geodetic	Borehole No. H9
Location Kingston, Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 15A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.28	100		90	- Limestone masonry.
2	0.28	0.53	80		80	- Limestone masonry with mortar.
3	0.53	1.14	50		80	- Limestone with mortar. Mortar from 0.61m to 0.71m
4	1.14	1.32	100		70	- Limestone with mortar.
5	1.32	1.60	68		70	- Limestone with cement grout.
6	1.60	2.29	74		70	- Granitic masonry with mortar.
7	2.29	2.74	55		70	- Granitic masonry with cement grout.
8	2.74	3.25	100		70	- Granitic masonry with grout. Lost wash water from 3.07m to 3.25m
9	3.25	3.60	78		70	- Granitic masonry with grout. End of wall at 3.6m.
	3.60	3.94			70	- Gneissic Syenite Bedrock, fair quality.
10	3.94	4.19	100	75	70	- Gneissic Syenite Bedrock, fair quality.

NOTES:

1. Hole core drilled, perpendicular to the face of the west monolith wall between Locks 48 and 49, 3.05m below coping, uncased with a 100mm size core barrel from 0 to 0.28m and with AW size core barrel from 0.28m to termination at 4.19m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H10

Trow

Auger Sample	☒	Natural Moisture	×
SPT (N) Value	▧	Plastic and Liquid Limit	├──○
Dynamic Cone Test	────	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project Kingston Mills Locks Dwg. No 16

West Monolith between Locks 48 & 49, 6.3m below coping

Kingston, Ontario Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Dist. (m)	depth	N Value 20 40 60 80 / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight 10 20 30		Natural Unit Weight kN/cu m
		STONE MASONRY - limestone and granite masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 8).	1.3	0				
		GNEISSIC SYNITE BEDROCK -alternating ribbons of alkali feldspar and chlorite, brick red, locally jointed and oriented at 60° degrees to core axis (excellent quality).	2.4	1				
		END OF HOLE		2				
				3				
				4				
				5				
				6				
				7				
				8				
				9				
				10				

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 76.3m	Datum Geodetic	Borehole No. H10
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 16A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m)		Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
	From	To				
1	0	0.30	100		90	- Limestone masonry.
2	0.33	0.59	100		90	- Limestone masonry.
3	0.59	0.81	100		90	- Limestone with grout.
4	0.81	1.09	77		90	- Limestone with mortar.
5	1.09	1.19	100		80	- Granitic masonry with mortar.
6	1.19	1.32	100		80	- Granitic masonry with mortar. End of wall.
7	1.32	1.47	100		100	- Gneissic Syenite Bedrock, excellent quality.
8	1.47	1.74	70	100	70	- Gneissic Syenite Bedrock, excellent quality.
9	1.74	2.40	100	100	100	- Gneissic Syenite Bedrock, excellent quality.

NOTES:

1. Hole core drilled, perpendicular to the face of the west monolith wall between Locks 48 and 49, 6.29m below coping, uncased with a 100mm size core barrel from 0 to 0.33m and with AW size core barrel from 0.33m to termination at 2.40m.

2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.

3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length of the total core run expressed as a percentage.

Log of Borehole

H11

Trow

Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	├──○
Dynamic Cone Test	────	Undrained Triaxial at Overburden Pressure % Strain at Failure	0, 5, 10, 15
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks — Dwg. No 17

East wall - Lock 47, 1.1m below coping at centreline

Kingston, Ontario — Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Dist. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		<u>STONE MASONRY</u> - grey, fractured and weathered limestone masonry with lime mortar in joints. Mortar soft and mostly washed out.		0				
				1				
				2				
		END OF HOLE	3.0	3				
				4				
				5				
				6				
				7				
				8				
				9				
				10				

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	**Borehole Location:** (see Site Plan)	**Collar Elevation** 84.9m	**Datum** Geodetic	**Borehole No.** H11
Location Kingston , Ontario	**Date Started**	**Completed**	**Logged By** I. Taki	**Drawing: 17A**
Client Public Works Canada	**Drilling Agency** Geotechnical Services	**Drill** Electric	**Core Barrel & Bit Design** AW	**Project Number** R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.22	90		90	- Limestone Masonry, weathered.
2	0.22	0.46	100		70	- Limestone Masonry, weathered.
3	0.46	0.81	59		70	- Limestone Masonry with mortar. Fine sand washed out from 0.81m to 1.17m.
4	0.81	1.17	100		30	- Fractured limestone with mortar, very soft from 0.81m to 1.07m.
5	1.17	1.35	100		70	- Fractured limestone masonry.
6	1.35	1.42	100		80	- Fractured limestone masonry.
7	1.42	1.75	100		80	- Fractured and oxidized limestone masonry.
8	1.75	1.85	100		50	- Fractured limestone masonry.
9	1.85	2.36	100		0	- Generally soft drilling. Fractured limestone masonry with grey cement grout.
10	2.36	2.53	91		0	- Limestone masonry.
11	2.53	3.05	91		0	- Limestone masonry. End of wall at 3.05m.

NOTES:
1. Hole core drilled prepeadicular to the face of the east wall of Lock 47, uncased with 100mm size core barrel from 0m to 0.22m and with AW size core barrel from 0.22m to termination at 3.05m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H12

Trow

Auger Sample	☒	Natural Moisture	X
SPT (N) Value	▨	Plastic and Liquid Limit	⊢—○
Dynamic Cone Test	———	Undrained Triaxial at Overburden Pressure % Strain at Failure	0, 15, 5, 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks — Dwg. No: 18

East wall - Lock 47, 0.9m above lock bottom at centreline

Kingston, Ontario — Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Dist. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		<u>STONE MASONRY</u> - grey, fractured and weathered limestone masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 18)		0				
		END OF HOLE	3.8					

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	81.0m	Geodetic	H12
Location Kingston , Ontario	**Date Started**	**Completed**	**Logged By** I. Taki	**Drawing: 18A**
Client Public Works Canada	**Drilling Agency** Geotechnical Services	**Drill** Electric	**Core Barrel & Bit Design** AW	**Project Number** R-00404A/GE

Run No.	Depth (m)		Total Core Recovery	Rock Quality Designation	Wash Water Recovery (%)	Wall and Bedrock Description
	From	To	TCR %	RQD %		
1	0	0.33	100		90	- Limestone masonry, fractured.
2	0.33	0.51	100		65	- Limestone masonry.
3	0.51	0.68	75		65	- Limestone masonry with mortar, generally easy drilling.
4	0.68	0.84	50		70	- Limestone masonry with mortar.
5	0.84	1.07	83		70	- Limestone and granite masonry with mortar.
6	1.07	1.19	100		80	- Granitic masonry.
7	1.19	1.29	100		70	- Granitic and limestone masonry.
8	1.29	1.52	89		70	- Granitic masonry with mortar.
9	1.52	1.66	100		70	- Granitic masonry with mortar.
10	1.66	1.73	83		80	- Granitic masonry with mortar. Mortar washed out from 1.72m to 1.75m.
11	1.73	2.06	38		80	- Granitic masonry with mortar. Mortar from 1.98m to 2.01m.
12	2.06	2.41	100		80	- Limestone with cement grout.
13	2.41	2.78	100		80	- Limestone masonry with mortar.
14	2.78	2.84	86		80	- Limestone masonry with mortar.
15	2.84	2.95	100		80	- Limestone masonry.
16	2.95	3.37	100		90	- Limestone masonry with mortar.
17	3.37	3.56	71		90	- Limestone masonry with mortar.
18	3.56	4.03	53		0	- Limestone masonry. End of wall at 3.78m.

NOTES:

1. Hole core drilled prependicular to the face of the wall, uncased with a 100mm size core barrel from 0m to 0.33m, and with AW size core barrel from 0.33m to termination at 4.03m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H13

Trow

Auger Sample	☒	Natural Moisture	X
SPT (N) Value	▨	Plastic and Liquid Limit	├—○
Dynamic Cone Test	——	Undrained Triaxial at Overburden Pressure % Strain at Failure	⊕ (0, 5, 10, 15)
Shelby Tube	● ● ■		
Field Vane Test	+ 5	Penetrometer	▲

Project: Kingston Mills Locks   Dwg. No 19

East Monolith wall between Locks 47 & 48, 0.6m below coping at centreline

Kingston, Ontario   Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	Symbol	Description	Dist. (m)	depth	N Value (20, 40, 60, 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10, 20, 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - weathered and fractured limestone masonry, with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 12)		0–3				
		END OF HOLE	3.9	4				
				5–10				

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

G 009R

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	85.3m	Geodetic	H13
Location	**Date Started**	**Completed**	**Logged By**	**Drawing: 19A**
Kingston , Ontario			I. Taki	
Client	**Drilling Agency**	**Drill**	**Core Barrel & Bit Design**	**Project Number**
Public Works Canada	Geotechnical Services	Electric	AW	R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall Description
1	0	0.30	100		90	- Fractured limestone masonry.
2	0.3	0.4	100		70	- Fractured limestone masonry.
3	0.4	0.58	90		60	- Limestone with mortar. Mortar washed out from 0.53m to 0.57m.
4	0.58	0.91	100		40	- Limestone masonry, with mortar, weathered. Mortar washed out from 0.74m to 0.76m.
5	0.91	1.60	59		40	- Fractured limestone masonry with mortar joints. Suspected voids from 1.19m to 1.22m and from 1.24m to 1.27m.
6	1.60	1.68	100		20	- Fractured limestone and granite masonry with mortar.
7	1.68	2.23	95		70	- Granitic and limestone masonry with mortar in between soft zone from 2.06m to 2.18m.
8	2.23	2.36	100		80	- Fractured limestone with mortar.
9	2.36	2.64	82		70	- Limestone masonry with mortar.
10	2.64	2.82	75		60	- Granitic masonry with mortar. Soft zone from 2.69m to 2.74m.
11	2.82	3.20	73		80	- Granitic masonry with mortar. Soft zone from 3.12m to 3.20mm
12	3.20	3.86	100		50	- Limestone masonry with mortar. End of wall.

NOTES:

1. Hole core drilled perpendicular to the east monolith wall between Locks 47& 48, uncased with a 100mm core barrel from 0 to 0.3m and with AW size core barrel from 0.3m to termination at 3.86m.

2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.

3. Rock Quality Designation (R.Q.D.) is expressed as the ratio ofthe length of sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H14

Trow

Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	├──○
Dynamic Cone Test	────	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project Kingston Mills Locks **Dwg. No** 20

East Monolith wall between Locks 47 & 48, 3.07m below coping at centreline.

Kingston, Ontario **Project No.** R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Soil Description	Dist. (m)	depth	N Value 20 40 60 80 / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight 10 20 30		Natural Unit Weight kN/cu m
		STONE MASONRY - weathered and fractured limestone masonry with occasional granite fragments, lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 8)		0				
				1				
				2				
				3				
		END OF HOLE	3.8	4				
				5				
				6				
				7				
				8				
				9				
				10				

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	82.9m	Geodetic	H14
Location Kingston , Ontario	**Date Started**	**Completed**	**Logged By** I. Taki	**Drawing: 20A**
Client Public Works Canada	**Drilling Agency** Geotechnical Services	**Drill** Electric	**Core Barrel & Bit Design** AW	**Project Number** R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.33	100		90	- Fractured limestone masonry.
2	0.33	0.42	90		70	- Fractured limestone masonry.
3	0.42	0.79	79		80	- Limestone with mortar. Mortar washed out from 0.71m to 0.79m.
4	0.79	1.37	80		60	- Fractured granitic masonry with mortar. Mortar washed out from 0.93m to 0.96m and from 1.09m to 1.12m.
5	1.37	1.75	100		70	- Granite masonry with mortar.
6	1.75	2.23	94		80	- Granite masonry with mortar, fractured.
7	2.23	3.15	55		80	- Granite masonry with mortar. Voids up to 0.07m between recovered core.
8	3.15	3.81	100		80	- Limestone masonry, with mortar. End of wall.

NOTES:

1. Hole core drilled, 3.07m below coping, prependicular to the east monolith wall between Locks 47& 48 uncased with a 100mm core barrel from 0m to 0.33m and with AW size core barrel from 0.33m to termination at 3.81m.

2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.

3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H15

Trow

Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▨	Plastic and Liquid Limit	├──○
Dynamic Cone Test	──	Undrained Triaxial at Overburden Pressure % Strain at Failure	⊕ (0, 5, 10, 15)
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project Kingston Mills Locks Dwg. No 21

East Monolith wall between Locks 47 & 48

Kingston, Ontario Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Dist. (m)	depth	N Value 20 40 60 80 / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight 10 20 30		Natural Unit Weight kN/cu m
		STONE MASONRY - weathered and fractured limestone masonry with lime mortar in joints and occasional granitic fragments. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 12)		0				
			4.0	4				
		FILL - clayey soil,brown, some gravel.	4.2					
		END OF HOLE						

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	79.9m	Geodetic	H15
Location: Kingston, Ontario	Date Started	Completed	Logged By: I. Taki	Drawing: 21A
Client: Public Works Canada	Drilling Agency: Geotechnical Services	Drill: Electric	Core Barrel & Bit Design: AW	Project Number: R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.33	100		80	- Fractured limestone masonry.
2	0.33	1.07	69		80	- Fractured limestone and granitic masonry with mortar. Mortar washed out from 0.53m to 0.56m and from 0.91m to 0.95m.
3	1.07	1.26	93		80	- Fractured limestone and granitic masonry with mortar.
4	1.26	1.65	58		60	- Granitic masonry with lime mortar from 1.46m to 1.65m.
5	1.65	2.18	26		60	- Limestone masonry with cement grout.
6	2.18	2.44	30		60	- Limestone and granite with mortar. Mortar washed out from 2.21m to 2.44m. Voids up to 0.07m.
7	2.44	2.89	11		40	- Granitic fragments.
8	2.89	3.68	22		60	- Granitic fragments.
9	3.68	3.77	100		60	- Granitic fragments.
10	3.77	3.86	100		60	- Granitic fragments with cement grout.
11	3.86	4.01	100		60	- Cement grout. End of wall.
12	4.01	4.19	0		60	- Washed out clayey soil.

NOTES:

1. Hole core drilled 0.60m above lock bottom, perpendicular to the east monolith wall between Locks 47& 48 uncased with a 100mm core barrel from 0m to 0.33m and with AW size core barrel from 0.33m to termination at 4.19m.

2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.

3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H16

Trow

Auger Sample	⊠	Natural Moisture	✕
SPT (N) Value	▨	Plastic and Liquid Limit	├──○
Dynamic Cone Test	──	Undrained Triaxial at Overburden Pressure % Strain at Failure	0, 5, 10, 15 ⊕
Shelby Tube	● ● ■		
Field Vane Test	+ S	Penetrometer	▲

Project Kingston Mills Locks **Dwg. No** 22

East Wall, Lock 48, 0.84m below coping at centreline.

Kingston, Ontario **Project No.** R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Dist. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)	Natural Unit Weight kN/cu m
		STONE MASONRY - weathered and fractured limestone masonry, with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 8)		0			
		END OF HOLE	3.0	3			

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 81.7m	Datum Geodetic	Borehole No. H16
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 22A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall Description
1	0	0.17	100		100	- Fractured limestone masonry.
2	0.17	0.66	95		90	- Weathered and fractured limestone masonry. Lost wash water from 0.51m to 0.66m.
3	0.66	0.89	40		90	- Weathered limestone masonry. Suspected void from 0.86m to 0.89m.
4	0.89	1.11	94		90	- Limestone masonry.
5	1.11	1.78	78		90	- Limestone masonry with mortar. Mortar from 1.14m to to 1.22m.
6	1.78	1.96	85		80	- Limestone masonry with grout. Lost wash water from 1.62m to 1.96m.
7	1.96	2.67	87		70	- Limestone masonry with mortar. Lost wash water from 2.0m to 2.67m. Void from 2.31m to 2.36m.
8	2.67	2.95	73		0	- Limestone masonry. End of wall.

NOTES:

1. Hole core drilled, 0.84m below coping, prependicular to the east wall face of Lock 48, uncased with a 100mm core barrel from 0m to 0.17m and with AW size core barrel from 0.17m to termination at 2.95m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H17

Trow

Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▨	Plastic and Liquid Limit	⊢—○
Dynamic Cone Test	——	Undrained Triaxial at Overburden Pressure % Strain at Failure	0, 15, 5, 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks — Dwg. No 23

East Wall, Lock 48, 0.91m above lock bottom at centreline.

Kingston, Ontario — Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	Symbol	Description	Dist. (m)	depth	N Value 20 40 60 80 / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight 10 20 30		Natural Unit Weight kN/cu m
		STONE MASONRY - weathered limestone and granitic masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 16)		0				
		FILL - silty clay, trace gravel, brown	3.6					
		END OF HOLE	3.8					

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	22.6m	Geodetic	H17
Location: Kingston , Ontario	Date Started	Completed	Logged By: I. Taki	Drawing: 23A
Client: Public Works Canada	Drilling Agency: Geotechnical Services	Drill: Electric	Core Barrel & Bit Design: AW	Project Number: R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall Description
1	0	0.33	100		90	- Limestone masonry.
2	0.33	0.59	100		90	- Limestone masonry.
3	0.59	0.76	46		90	- Limestone masonry. Mortar from 0.71m to 0.76m.
4	0.76	1.04	86		90	- Limestone and granitic masonry.
5	1.04	1.22	64		90	- Granitic masonry with mortar.
6	1.22	1.33	100		90	- Granitic masonry with mortar. Mortar from 1.22m to 1.25m.
7	1.33	1.65	69		90	- Granitic masonry with mortar.
8	1.65	1.93	45		90	- Granitic masonry with mortar.
9	1.93	1.98	100		80	- Granite fragments. Lost wash water from 1.93m to 1.98m.
10	1.98	2.29	60		50	- Granitic masonry with mortar. Mortar from 2.21m to 2.24m. Lost wash water from 2.23m to 2.29m.
11	2.29	2.70	50		70	- Granitic masonry with mortar.
12	2.70	2.90	89		20	- Granitic masonry with mortar.Mortar from 2.70m to 2.90m.
13	2.90	3.0	100		70	- Granitic masonry with mortar.
14	3.0	3.12	100		70	- Granitic masonry.
15	3.12	3.20	100		70	- Granitic masonry.
16	3.20	3.83	50		90	- Granite with masonry. Void from 3.50m to 3.55m. End of wall at 3.58m.

NOTES:

1. Hole core drilled 0.91m above Lock bottom, perpendicular to the face of the uncased with a 100mm core barrel from 0m to 0.33m and with AW size core barrel from 0.33m to termination at 3.83m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H18

Trow

Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	├──○
Dynamic Cone Test	────	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks   Dwg. No 24

East Monolith between Locks 48 & 49, 0.60m below coping at centreline

Kingston, Ontario   Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Soil Description	Dist. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - grey limestone masonry with lime mortar in joints. Mortar soft and mostly washed out. (Run Nos. 1 to 5)		0				
		END OF HOLE	3.8					

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 82.0m	Datum Geodetic	Borehole No. H18
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 24A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.33	100		100	- Limestone masonry.
2	0.33	1.52	83		90	- Limestone masonry with mortar.
3	1.52	2.38	80		90	- Limestone masonry.
4	2.38	2.84	89		80	- Limestone masonry with mortar.
5	2.84	3.75	89		80	- Limestone masonry with mortar. Void from 3.28m to 3.33m. Lost wash water from 3.33m to 3.76m. End of wall at 3.75m.

NOTES:

1. Hole core drilled 0.60m below coping, perpendicular to the face of the east monolith wall between Locks 48 and 49, with a 100mm core barrel from 0m to 0.33m and with AW size core barrel from 0.33m to termination at 3.75m.

2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.

3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H19

Trow

Auger Sample	☒	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	⊢—○
Dynamic Cone Test	——	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project Kingston Mills Locks **Dwg. No** 25

East Monolith wall between Locks 48 & 49, 3.1m below coping at centreline

Kingston, Ontario **Project No.** R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Dist. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - grey limestone masonry with lime mortar in joints. Mortar soft and mostly washed out. (Run Nos. 1 to 9)		0				
		END OF HOLE	3.7					

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	79.4m	Geodetic	H19
Location	**Date Started**	**Completed**	**Logged By**	**Drawing: 25A**
Kingston , Ontario			I. Taki	
Client	**Drilling Agency**	**Drill**	**Core Barrel & Bit Design**	**Project Number**
Public Works Canada	Geotechnical Services	Electric	AW and EW	R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.25	67		70	- Fractured and weathered limestone masonry.
2	0.25	0.49	84		80	- Fractured and weathered limestone masonry.
3	0.49	0.81	78		80	- Limestone masonry with grout. Mortar from 0.61m to 0.69m
4	0.81	1.37	91		80	- Limestone masonry.
5	1.37	1.65	91		80	- Limestone masonry.
6	1.65	2.06	100		60	- Limestone masonry with mortar and granite fragments.
7	2.06	2.54	74		60	- Granitic masonry with mortar and limestone fragments.
8	2.54	2.92	73		60	- Limestone masonry with mortar.
9	2.92	3.72	32		60	- Limestone masonry with mortar. Mortar washed out from 3.15m to 3.45m. - Lost wash water from 3.45m to 3.72m. - Extensive seepage at the outer face of the wall, at the joint located 3.53m below coping. End of wall.

NOTES:

1. Hole core drilled, 3.1mbelow coping, perpendicular to the face of the east monolith between Locks 48 and 49, with a 100mm core barrel from 0m to 0.25m and with AW size core barrel from 0.25m to termination at 3.72m.

2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.

3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole H20

Trow

Auger Sample	⊠	Natural Moisture	X	Project	Kingston Mills Locks
SPT (N) Value	▧	Plastic and Liquid Limit	├──○	Dwg. No	26
Dynamic Cone Test	——	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10		East Monolith wall between Locks 48 & 49
Shelby Tube	● ● ■			Project No.	R-00404A/GE
Field Vane Test	+ s	Penetrometer	▲		Kingston, Ontario

Hole location and datum see drawing No. 1

GWL	symbol	Description	Dist. (m)	depth	N Value (20, 40, 60, 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10, 20, 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - grey limestone masonry with lime mortar in joints. Mortar soft and mostly washed out. (Run Nos. 1 to 12)		0				
		END OF HOLE	3.7					

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation	Datum Geodetic	Borehole No. H20
Location Kingston, Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 26A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall Description
1	0	0.32	100		100	- Limestone masonry.
2	0.32	0.58	78		80	- Limestone masonry with mortar.
3	0.58	0.93	78		80	- Limestone with mortar.
4	0.93	1.14	91		80	- Limestone with mortar.
5	1.14	1.39	60		80	- Limestone and granite with mortar.
6	1.39	1.51	89		80	- Limestone with mortar.
7	1.51	2.21	65		80	- Limestone with mortar. - Soft zone from 2.13m to 2.21m.
8	2.21	2.49	92		80	- Limestone with grout. - Soft zone from 2.21m to 2.34m.
9	2.49	2.64	100		80	- Limestone with grout.
10	2.64	2.87	67		80	- Limestone masonry.
11	2.87	3.09	89		80	- Limestone masonry.
12	3.09	3.66	100		60	- Limestone masonry with grout. End of wall

NOTES:

1. Hole core drilled 0.6m above bottom of Lock 48, perpendicular to the face of the east monolith wall between Locks 48 and 49, with a 100mm core barrel from 0m to 0.32m, and with AW size core barrel from 0.32m to termination at 3.66m.

2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.

3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole V1

Trow

Auger Sample	⊠	Natural Moisture	X	Project: Kingston Mills Locks — West wall - Locks 47 — Kingston, Ontario
SPT (N) Value	▧	Plastic and Liquid Limit	├──○	Dwg. No 27
Dynamic Cone Test	───	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10	Project No. R-00404A/GE
Shelby Tube	● ● ■			
Field Vane Test	+ s	Penetrometer	▲	Hole location and datum see drawing No. 1

GWL	symbol	Description	Elev. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - grey, limestone masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 11)	86.0	0				
				1				
				2				
				3				
				4				
▽				5				
		GNEISSIC SYENITE BEDROCK - alternating ribbons of alkali feldspar and chlorite, brick, red, locally jointed and oriented at 60° degrees to core axis (poor to good quality).	80.1	6				
		END OF HOLE	78.8	7				
				8				
				9				
				10				

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 86.0m	Datum Geodetic	Borehole No. V1
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 27A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.4	97		90	- Grey limestone masonry with mortar joints, unless otherwise specified.
2	0.4	0.7	58		0	- Lost wash water - seepage of water at the inner face of wall at the joint located 2.2m below coping. - suspected void from 0.48m depth to 0.53m depth.
3	0.7	1.1	69		0	- Generally soft coring. - suspected void from 0.96m depth to 1.04m depth. - Mortar washing out.
4	1.1	2.3	29		0	- Very soft zones from 1.60m to 1.62m and from 1.70m to 1.80m.
5	2.3	2.6	100		0	- Borehole caved in at 2.0m depth.
6	2.6	3.2	48		0	- Suspected voids from 2.61m to 2.64m and from 2.77m to 2.82m and from 3.15m to 3.18m depth.
7	3.2	3.9	100		0	- Soft zone from 3.36m to 3.88m depth - With granite and mortar fragments.
8	3.9	4.4	79		0	- Suspected void from 3.98m to 4.01m depth.
9	4.4	5.0	100		0	- Limestone and granitic masonry with mortar.
10	5.0	5.6	80		0	- Granitic masonry with mortar.
11	5.6	5.9	50		0	- Seepage of water at the inner face of wall, at the joint located 1.7m North to the drill hole, 1.8m below coping.
12	5.9	6.5	50		0	- Gneissic Syenite Bedrock, poor quality.
13	6.5	6.9	100	29	0	- Gneissic Syenite Bedrock, poor quality.
14	6.9	7.2	75	75	0	- Gneissic Syenite Bedrock, fair quality.
15	7.2	7.5	94	94	0	- Gneissic Syenite Bedrock, good quality.
						NOTES CONTINUED ON NEXT PAGE:

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation	Datum Geodetic	Borehole No. V1
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 27A (cont.)
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall Description
						NOTES:

NOTES:

1. Borehole core drilled uncased with AW size core barrel from 0m depth to 5.6m depth and cased with EW size core barrel from 5.6m to termination at 7.2m depth.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.
4. Standpipe installed at 7.2m depth, water level records as follows:

DATE	WATER LEVEL (m)
Oct. 20, 1989	-
Oct. 23, 1989	5.4
Oct. 24, 1989	5.4
Oct. 25, 1989	5.2
Oct. 26, 1989	5.1
Oct. 27, 1989	5.2
Oct. 30, 1989	5.2
Oct. 31, 1989	5.2
Nov. 1, 1989	5.35
Nov. 2, 1989	5.40
Nov. 3, 1989	

Log of Borehole V2

Trow

Auger Sample	⊠	Natural Moisture	×	**Project**	Kingston Mills Locks — **Dwg. No** 28
SPT (N) Value	▨	Plastic and Liquid Limit	├──○		East Monolith between Locks 47 & 48
Dynamic Cone Test	────	Undrained Triaxial at Overburden Pressure % Strain at Failure	⊕ (0, 5, 10, 15)		Kingston, Ontario — **Project No.** R-00404A/GE
Shelby Tube	● ● ■				
Field Vane Test	+ s	Penetrometer	▲		Hole location and datum see drawing No. 1

GWL	Symbol	Description	Elev. (m)	depth	N Value (20, 40, 60, 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10, 20, 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - grey, limestone masonry, weathered and fractured, lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 15)	86.0	0				
		METADIABASE BEDROCK - dark green, fine to medium weathered, minor vugs, moderately soft to hard (very poor to good quality).	78.6					
		END OF HOLE	77.1					

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 86.0m	Datum Geodetic	Borehole No. V2
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing 28A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m)		Total Core Recovery	Rock Quality Designation	Wash Water Recovery (%)	Wall and Bedrock Description
	From	To	TCR %	RQD %		
1	0	0.25	85		30	- Generally limestone masonry unless otherwise specified. - Suspected void from 0.14m to 0.15m depth. - Lost wash water at 0.15m depth.
2	0.25	0.43	93		50	- Generally soft coring.
3	0.43	0.91	31		0	- Extensive grey water seepage at the inner face of wall at the joints located 0.13m and 1.75m below coping (Run Nos. 3 to 9) (Picture #2).
4	0.91	1.24	61		0	
5	1.24	1.45	87		0	
6	1.45	1.65	81		0	- Very soft from 1.54m to 1.60m depth.
7	1.65	1.88	83		0	- Very soft from 1.72m to 1.75m depth.
8	1.88	2.08	75		0	- Changes to limestone and granitic masonry with mortar.
9	2.08	2.29	75		0	- Limestone and granitic masonry with cement grout.
10	2.29	2.41	100		0	- Extensive brown water seepage at the inner face of wall at the joints located 2.13m and 4.5m below coping (Run Nos. 10 to 15).
11	2.41	2.56	83		0	- Limestone masonry with mortar.
12	2.56	2.70	54		0	- Limestone masonry with mortar.
13	2.70	2.84	45		0	- Limestone masonry with mortar.
14	2.84	3.08	84		0	- Granitic masonry with mortar.
15	3.08	3.40	96		0	- Limestone and granitic masonry with mortar.
16	3.40	3.71	100		0	- Limestone and granitic masonry with mortar.
17	3.71	4.04	100		0	- Granitic masonry with mortar. - Seepage at the inner face of wall at the joint located 4.6m below coping. (Run Nos. 17 to 22) (Picture #3).
18	4.04	4.30	52		0	- Granitic masonry with cement grout.
19	4.30	4.52	100		0	- Limestone and granitic masonry with grout. - Suspected void from 4.28m to 4.30m depth.
20	4.52	5.28	100		0	- Limestone and granite with grout. - Brownish grey water seepage at the inner face of wall at the joint located 0.60m South of drill hole, 2.13m below coping.

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	86.0m	Geodetic	V2
Location: Kingston, Ontario	Date Started	Completed	Logged By: I. Taki	Drawing: 28A (cont.)
Client: Public Works Canada	Drilling Agency: Geotechnical Services	Drill: Electric	Core Barrel & Bit Design: AW and EW	Project Number: R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall Description
21	5.28	5.48	75		0	- Limestone and granitic masonry with mortar.
22	5.48	5.89	56		0	- Same as Run No. 21 with suspected voids from 5.64m to 5.71m and from 5.84m to 5.89m depth.
23	5.89	6.52	76		0	- Recovery same as Run No. 21.
24	6.52	6.73	50		0	- Gneissic Synetic Bedrock with mortar.
25	6.73	7.39	80		0	- Gneissic Synetic Bedrock with mortar.
26	7.39	7.67	91	0	0	- Metadiabase Bedrock, very poor quality.
27	7.67	7.97	100	96		- Metadiabase Bedrock, very good quality.
28	7.97	8.07	100	0		- Metadiabase Bedrock, very good quality.
29	8.07	8.55	99	66		- Metadiabase Bedrock, fair quality,
30	8.55	8.91	100	0		- Metadiabse Bedrock, very poor quality.

NOTES:

1. Hole core drilled uncased with AW size core barrel from 0m depth to termination at 8.9m depth.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.
4. Sleeve pipe installed in the borehole for trial grouting tests with barriers installed at the following locations.

BARRIER NO.	DEPTH FROM TOP
1	7.39m
2	4.03m
3	2.51m

Log of Borehole V3

Trow

Legend	Symbol	Legend	Symbol
Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	├──○
Dynamic Cone Test	──	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks — West wall Lock 48 — Kingston, Ontario

Dwg. No: 29

Project No.: R-00404A/GE

Hole location and datum see drawing No. 1

GWL	Symbol	Soil Description	Elev. (m)	Depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - grey limestone masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 14).	82.6	0				
		METADIABASE BEDROCK - fine to medium grained, moderately to strongly weathered, minor vugs, moderately soft to hard (very poor to excellent quality).	78.2					
		END OF HOLE	75.9					

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	82.6m	Geodetic	V3
Location	**Date Started**	**Completed**	**Logged By**	
Kingston , Ontario			I. Taki	Drawing: 29A
Client	**Drilling Agency**	**Drill**	**Core Barrel & Bit Design**	**Project Number**
Public Works Canada	Geotechnical Services	Electric	AW and EW	R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.23	100		90	- Limestone masonry - fractured. Void from 0.14m to 0.15m.
2	0.23	0.33	87		70	- Limestone masonry - fractured. Seepage at the inner face of wall at the joint located 2.74m below coping. (Picture #4 and 5).
3	0.33	0.47	100		0	- Limestone masonry Seepage same as in Run No. 2
4	0.47	0.86	71		0	- Limestone masonry Seepage same as in Run No. 2 Soft zone from 0.66m to 0.68m.
5	0.86	1.68	81		0	- Limestone masonry with mortar. Void from 1.19m to 1.21m Soft from 1.35m to 1.38m Seepage same as in Run No. 2.
6	1.68	2.03	82		0	- Limestone masonry with mortar. Little seepage at the location noted in Run No.5.
7	2.03	2.72	65		0	- Limestone masonry with cement grout. Void from 2.13m to 2.16m. Soft zone from 2.51m to 2.54m.
8	2.72	2.99	64		0	- Limestone masonry with mortar Void from 2.79m to 2.82m.
9	2.99	3.28	54		0	- Granitic masonry with mortar Void from 2.99m to 3.09m.
10	3.28	3.48	69		0	- Granitic masonry with mortar. Generally easy drilling.
11	3.48	3.63	50		0	- Granitic masonry with mortar.
12	3.63	3.86	87		0	- Granitic masonry with mortar.
13	3.86	4.16	75		0	- Granitic masonry with mortar.
14	4.16	4.39	55		0	- Granitic masonry with lime mortar and grey cement grout.
15	4.39	4.86	81	60	0	- Metadiabase Bedrock, fair quality,

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 82.6m	Datum Geodetic	Borehole No. V3
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 29A (cont.)
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall Description
16	4.86	4.98	100	100	0	- Metadiabase Bedrock - excellent quality
17	4.98	5.08	100	100	0	- Metadiabase Bedrock - excellent quality
18	5.08	5.61	84	43	0	- Metadiabase Bedrock - poor quality.
19	5.61	5.90	96	56	0	- Metadiabase Bedrock - fair quality.
20	5.90	6.42	97	56	0	- Metadiabase Bedrock - fair quality.
21	6.42	6.68	80	0	0	- Metadiabase Bedrock - very poor quality.

NOTES:

1. Borehole core drilled uncased with AW size core barrel from 0m depth to 4.98m depth, and cased with EW size core barrel from 4.98m to termination at 6.68m.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.
4. Sleeve pipe installed in the borehole for trial grouting tests with barriers installed at the following locations:

BARRIER NO.	DEPTH FROM TOP
1	4.85m
2	2.43m

Log of Borehole

V4

Trow

Auger Sample	⊠	Natural Moisture	×
SPT (N) Value	▧	Plastic and Liquid Limit	├──○
Dynamic Cone Test	———	Undrained Triaxial at Overburden Pressure % Strain at Failure	0 / 15 ⊕ 5 / 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project: Kingston Mills Locks — Dwg. No: 30

East Wall Lock 47

Kingston, Ontario — Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Elev. (m)	depth	N Value 20 40 60 80 / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight 10 20 30		Natural Unit Weight kN/cu m
		STONE MASONRY - grey, limestone masonry, weathered and fractured with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 10)	86.0	0				
				1				
				2				
				3				
				4				
				5				
		GNEISSIC SYENITE BEDROCK - alternating ribbons of alkali feldspar and chlorite, brick red, locally jointed and oriented at 60° degrees to core axis (poor to good quality).	80.3	6				
		END OF HOLE	78.9	7				
				8				
				9				
				10				

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	**Borehole Location:** (see Site Plan)	**Collar Elevation** 86.0m	**Datum** Geotetic	**Borehole No.** V4
Location Kingston , Ontario	**Date Started**	**Completed**	**Logged By** I. Taki	**Drawing: 30A**
Client Public Works Canada	**Drilling Agency** Geotechnical Services	**Drill** Electrical	**Core Barrel & Bit Design** AW and EW	**Project Number** R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.48	68		90	Grey limestone masonry with mortar unless otherwise is specified. - Lost wash water from 0.18m to 0.3m depth.
2	0.48	0.76	91		90	- Seepage of brownish grey water at the outer face of wall at the joint located 0.3m below coping, 1.72m North of drill hole. Lost wash water from 0.66m to 0.76m depth. (Picture #6).
3	0.76	1.19	94		0	- Limestone and granitic masonry with mortar. Very soft from 0.81m to 0.85m depth. No seepage at the outer face wall
4	1.19	1.60	75		80	
5	1.60	2.08	71		80	- Limestone masonry with granitic fragments.
6	2.08	3.20	81			- Suspected void from 2.45m to 2.49m depth. Soft zones from 2.79m to 2.82m and from 3.14m to 3.20m depth. Wet areas appeared at the inner face of wall at 0.91m north and 1.82m south of drill hole. Extensive seepage same location as in Run 2.
7	3.20	3.94	94		0	- Changes to limestone and granitic masonry with mortar starting from 3.20m depth. Soft zone from 3.33m to 3.40m depth.
8	3.94	4.19	75		0	- Soft zone from 4.04m to 4.14m depth. Suspected voids from 4.04m to 4.06m and from 4.19m to 4.21m depth.
9	4.19	5.28	67		0	- Suspected void from 4.64m to 4.70m depth. - Very soft zone from 4.70m to 5.18m depth.
10	5.28	5.66	53		0	- Suspected void from 5.46m to 5.49m depth.
11	5.66	6.08	100	26	0	- Gneissic Syenite Bedrock, poor quality.
12	6.08	6.29	82	27	0	- Gneissic Syenite Bedrock, poor quality. Extensive seepage at the joint located. - 0.3m above bottom of lock, 6.7m south of drill hole and 1.01m above bottom of lock, 3.3m south of drill hole.

TROW

CORE LOG

Project	Borehole Location: (see Site Plan)	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks		86.0m	Geotetic	V4
Location	**Date Started**	**Completed**	**Logged By**	**Drawing: 30A (cont.)**
Kingston, Ontario			I. Taki	
Client	**Drilling Agency**	**Drill**	**Core Barrel & Bit Design**	**Project Number**
Public Works Canada	Geotechnical Services	Electric	Aw and Ew	R-00404A/GE

Run No.	Depth (m)		Total Core Recovery	Rock Quality Designation	Wash Water Recovery (%)	Wall and Bedrock Description
	From	To	TCR %	RQD %		
13	6.29	6.55	100	40	0	- Gneissic Syenite Bedrock, poor quality.
14	6.55	6.78	89	78	0	- Gneissic Syenite Bedrock, good quality.
15	6.78	7.03	95	47	0	- Gneissic Syenite Bedrock, poor quality.
16	7.03	7.14	100	100	0	- Gneissic Syenite Bedrock, good quality.

NOTES:

1. Borehole core drilled uncased with A.W. size core barrel from 0m depth to termination at 7.14m depth.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.
4. Sleeve pipe installed in the Borehole for trial grouting tests with Barriers placed at the following locations.

Barrier No.	Depth From Top
# 1	5.58m
# 2	2.44m

Log of Borehole V5

Trow

Auger Sample	☒	Natural Moisture	×
SPT (N) Value	▨	Plastic and Liquid Limit	├──○
Dynamic Cone Test	───	Undrained Triaxial at Overburden Pressure % Strain at Failure	⊕ (0, 5, 10, 15)
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project Kingston Mills Locks Dwg. No 31

East monolith between Locks 47 & 48

Kingston, Ontario Project No. R-00404A/GE

Hole location and datum see drawing No. 1

GWL	Symbol	Description	Elev. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
			86.0	0				
		STONE MASONRY - grey, limestone masonry, weathered and fractured with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 26)		1 2 3 4 5 6 7				
7.4			78.6					
		GNEISSIC SYNITE BEDROCK - alternating ribbons of alkali feldspar and chlorite, brick red, locally jointed and oriented at 60° degrees to core axis (poor to good quality).		8				
8.9		END OF HOLE	77.1	9 10				

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)	86.0m	Geodetic	V5
Location	**Date Started**	**Completed**	**Logged By**	
Kingston , Ontario			I. Taki	Drawing: 31A
Client	**Drilling Agency**	**Drill**	**Core Barrel & Bit Design**	**Project Number**
Public Works Canada	Geotechnical Services	Electric	AW and EW	R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
1	0	0.18	86		0	- Generally grey limestone masonry with mortar, unless otherwise specified. Masonry is weathered and fractured.
2	0.18	0.56	87		0	- Extensive grey water seepage at the inner face of wall at the joint located 0.30m below coping at the wall mid height during Run Nos. 2 to 10.
3	0.56	0.76	100		0	- Soft zone from 0.62m to 0.68m depth
4	0.76	0.93	80		0	
5	0.93	1.12	27		0	
6	1.12	1.39	64		0	- Suspected void from 1.19m to 1.39m depth with granite fragments.
7	1.39	1.47	100		0	
8	1.47	1.56	100		0	- Very soft zone from 1.5m to 1.56m depth.
9	1.56	1.60	67		0	
10	1.60	1.78	86		0	- Soft zone from 1.72m to 1.75m depth.
11	1.78	2.10	77		0	
12	2.10	2.43	77		0	
13	2.43	2.61	100		0	- Seepage at the outer face of wall at the joint located 1.82m south of drill hole, 3.2m below coping. (Picture #8).
14	2.61	3.09	58		0	- Granitic masonry with mortar.
15	3.09	3.30	88		0	- Limestone and granitic masonry with mortar starting from 3.09m depth.
16	3.30	3.67	83		0	- Same as in Run No. 16.
17	3.67	4.29	57		0	- Soft zone from 4.11m to 4.19m depth.
18	4.29	4.88	46		0	- Generally easy drilling.
19	4.88	5.04	93		0	
20	5.04	5.28	42		0	- Little seepage to none at the outer face of wall at the location indicated in Run No. 13.
21	5.28	5.64	93		0	
22	5.64	5.84	75		0	

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 86.0m	Datum Geodetic	Borehole No. V5
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 31A (cont.)
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
23	5.84	6.02	100		0	
24	6.02	6.19	71		0	
25	6.19	6.70	75		0	- With grout pieces up to 75mm in diameter.
26	6.70	7.42	39		0	- Suspected void from 6.95m to 6.98m depth. - Soft zones from 7.0m to 7.05m and from 7.24m to 7.26m.
27	7.42	7.68	100	81	0	- Gneissic Syenite Bedrock, good quality.
28	7.68	7.79	89	67	0	- Gneissic Syenite Bedrock, fair quality,
29	7.79	8.33	95	65	0	- Gneissic Syenite Bedrock, fair quality.
30	8.33	8.89	89	50	0	- Gneissic Syenite Bedrock, poor quality.

<u>NOTES:</u>

1. Borehole core drilled uncased with AW size core barrel from 0m to termination at 8.89m depth.
2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.
3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

Log of Borehole V6

Trow

Legend		Legend	
Auger Sample	⊠	Natural Moisture	X
SPT (N) Value	▧	Plastic and Liquid Limit	⊢—o
Dynamic Cone Test	——	Undrained Triaxial at Overburden Pressure % Strain at Failure	0, 15, 5, 10
Shelby Tube	● ● ■		
Field Vane Test	+ s	Penetrometer	▲

Project Kingston Mills Locks **Dwg. No** 32

East Monolith between Locks 47 & 48

Kingston, Ontario **Project No.** R-00404A/GE

Hole location and datum see drawing No. 1

GWL	symbol	Description	Elev. (m)	depth	N Value (20 40 60 80) / Shear Strength MPa	Natural Moisture Content and Atterberg Limits % Dry Weight (10 20 30)		Natural Unit Weight kN/cu m
		STONE MASONRY - fractured and weathered limestone masonry with lime mortar in joints. Mortar soft and mostly washed out. (see Core Log Run Nos. 1 to 16)	82.6	0–7				
		GNEISSIC SYENITE BEDROCK - alternating ribbons of alkali feldspan and chlorite, brick red, locally jointed and oriented at 60° degrees to core axis (good to excellent quality). (see Core Log Run Nos. 17 to 19)	7.0	7–8.6				
		END OF HOLE	8.6					

G 009R

Note: BOREHOLE DATA REQUIRES INTERPRETATION ASSISTANCE FROM TROW BEFORE USE BY OTHERS

TROW

CORE LOG

Project Kingston Mills Locks	Borehole Location: (see Site Plan)	Collar Elevation 82.58m	Datum Geodetic	Borehole No. V6
Location Kingston , Ontario	Date Started	Completed	Logged By I. Taki	Drawing: 32A
Client Public Works Canada	Drilling Agency Geotechnical Services	Drill Electric	Core Barrel & Bit Design AW and EW	Project Number R-00404A/GE

Run No.	Depth (m)		Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
	From	To				
1	0	0.58	46		80	- Limestone masonry, fractured with mortar seams.
2	0.58	0.83	73		80	- Limestone masonry, fractured with mortar seams.
3	0.83	1.62	67		80	- Limestone and granitic masonry with mortar. Mortar washed out from 0.91m to 1.29m. - Suspected voids from 1.29m to 1.32m and from 1.57m to 1.60m. - Hole caved in at 0.94m depth.
4	1.62	2.03	43		80	- Fractured granitic masonry. Mortar washed out from 1.62m to 2.03m.
5	2.03	2.67	62		80	- Granite fragments with mortar. Suspected void from 2.3m to 2.4m depth. - Lost wash water from 2.46m to 2.49m.
6	2.67	3.45	84		80	- Limestone and granite masonry with mortar. - Mortar washing out from 2.67m to 3.45m.
7	3.45	3.71	90		80	- Fractured limestone with mortar.
8	3.71	4.11	81		70	- Fractured limestone with mortar. - Soft zone from 4.03m to 4.08m. - Lost wash water from 3.89m to 4.11m.
9	4.11	4.21	100		80	- Fractured limestone masonry.
10	4.21	4.57	96		60	- Fractured limestone masonry with mortar. - Lost wash water from 4.21m to 4.37m.
11	4.57	4.89	64		60	- Fractured limestone with mortar.
12	4.89	5.08	47		80	- Fractured limestone with mortar.
13	5.08	5.28	100		80	- Fractured limestone with mortar.
14	5.28	5.93	78		80	- Limestone and granitic masonry with mortar. - Lost wash water from 5.84m to 5.93m.
15	5.93	6.69	43		50	- Limestone with grout pieces up to 0.076m in length. - Soft zone from 6.5m to 6.70m.
16	6.69	6.97	87		60	- Gneissic masonry with mortar.
17	6.97	7.69	98	95	0	- Gneissic Syenite Bedrock, excellent quality Suspected void from 6.98m to 7.02m.

TROW

CORE LOG

Project	Borehole Location:	Collar Elevation	Datum	Borehole No.
Kingston Mills Locks	(see Site Plan)		Geodetic	V6
Location Kingston , Ontario	**Date Started**	**Completed**	**Logged By** I. Taki	**Drawing: 32A (cont.)**
Client Public Works Canada	**Drilling Agency** Geotechnical Services	**Drill** Electric	**Core Barrel & Bit Design** AW and EW	**Project Number** R-00404A/GE

Run No.	Depth (m) From	Depth (m) To	Total Core Recovery TCR %	Rock Quality Designation RQD %	Wash Water Recovery (%)	Wall and Bedrock Description
18	7.51	7.69	71	71	0	- Gneissic Syenite Bedrock, good quality.
19	7.69	8.63	82	99	0	- Gneissic Syenite Bedrock, excellent quality.

NOTES:

1. Hole core drilled at the east monolith between Locks 48 and 49, uncased with an AW size core barrel from 0m to 6.97m and cased with EW size core barrel from 6.97m to termination at 8.63m depth.

2. Total Core Recovery (T.C.R.) is expressed as the ratio of total core length to the total core run expressed as a percentage.

3. Rock Quality Designation (R.Q.D.) is expressed as the ratio of the length of hard sound pieces of rock core 100mm or greater in length to the total core run expressed as a percentage.

4. Sleeve pipe installed at the bottom of the Borehole for trial grouting test, with barriers installed at the following locations.

BARRIER NO.	DEPTH FROM TOP
1	7.1m
2	4.7m
3	2.2m

APPENDIX 'A'

PHOTOGRAPHS

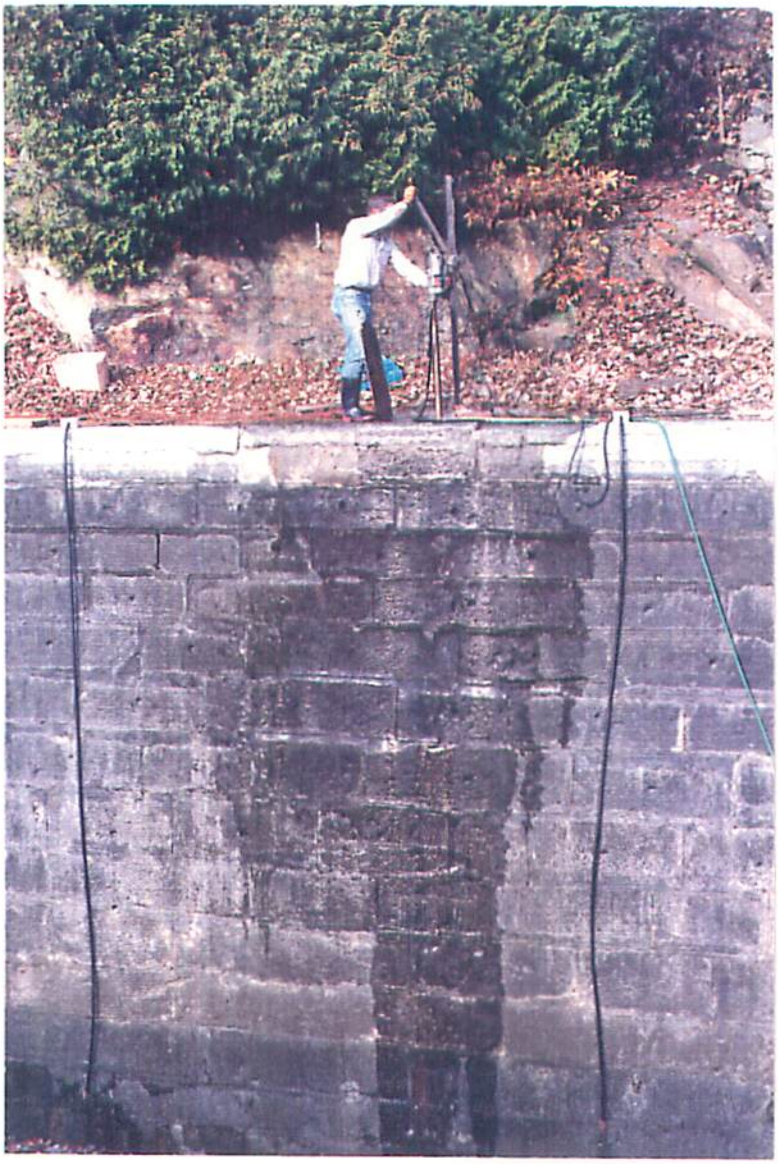
Trow

DRILLING OPERATIONS

Photograph No. 1 Coring of Hole Nos. V3 and H6

Photograph No. 2 Seepage noted in Run No. 2 - Hole V2

Photograph No. 3 Seepage noted in Run No. 17 - Hole V2



Photograph No. 4 Seepage noted in Run No. 2 - Hole V3

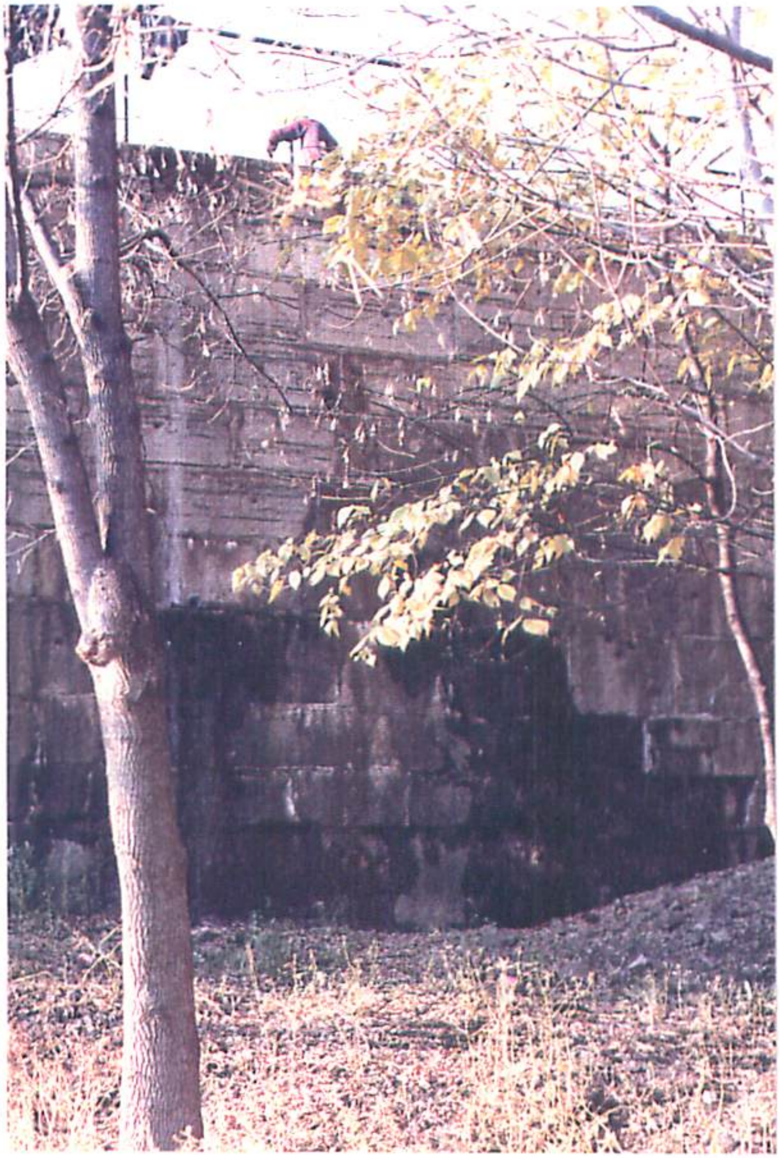
Trow

Photograph No. 5 Seepage noted in Run No. 2 - Hole V3

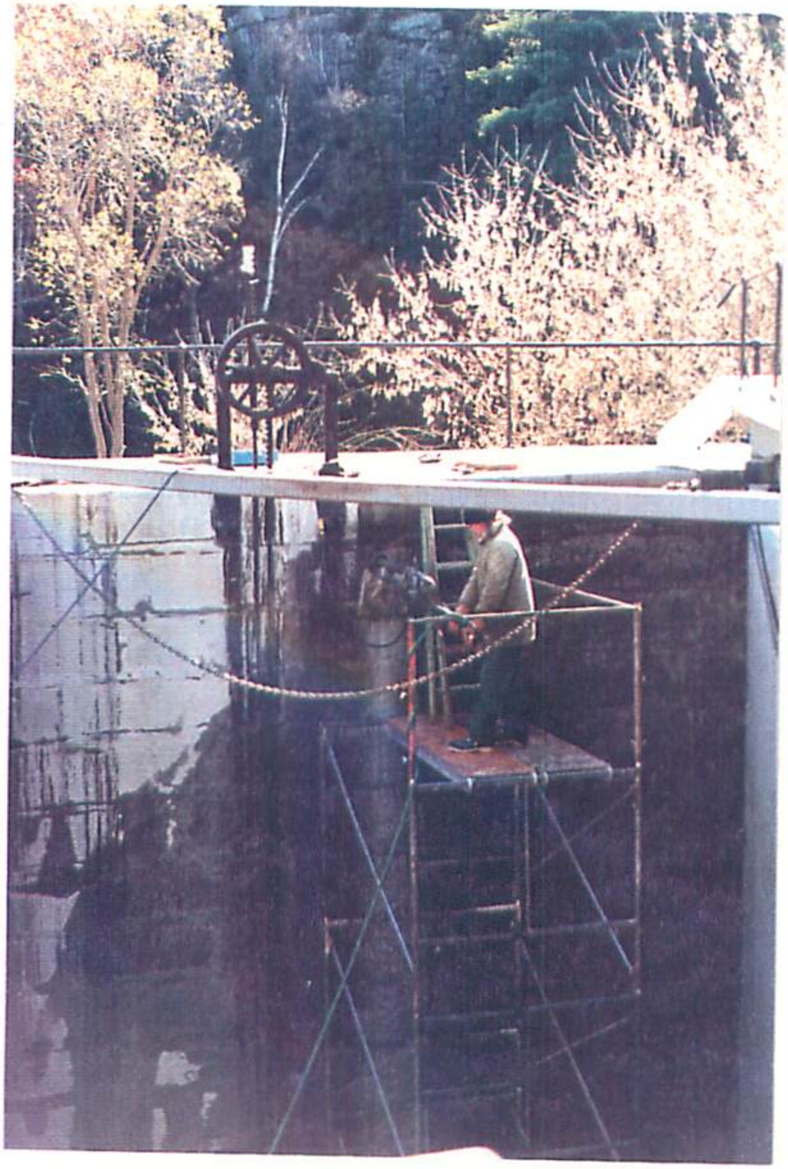
Photograph No. 6 Seepage noted in Run No. 2 - Hole V4

Trow

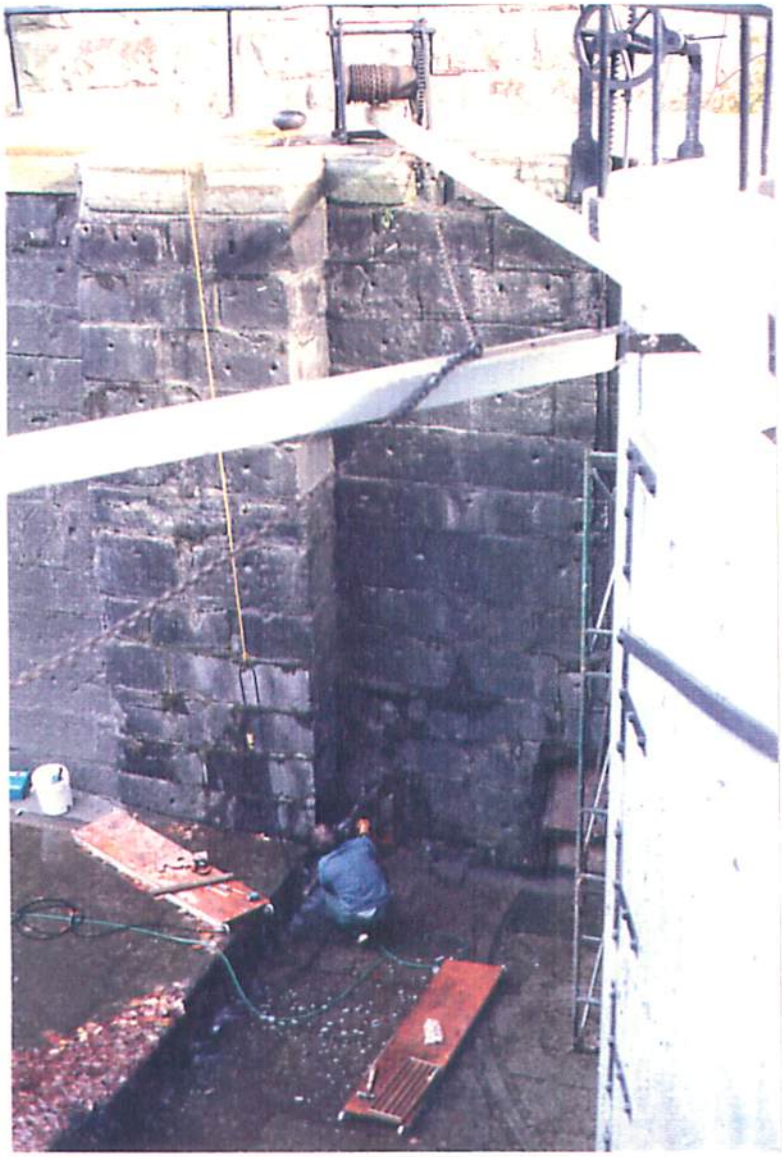
Photograph No. 7 Coring of Hole No. V5



Photograph No. 8 Seepage noted in Run No. 13 - Hole V5



Photograph No. 9 Coring of Hole No. H 18

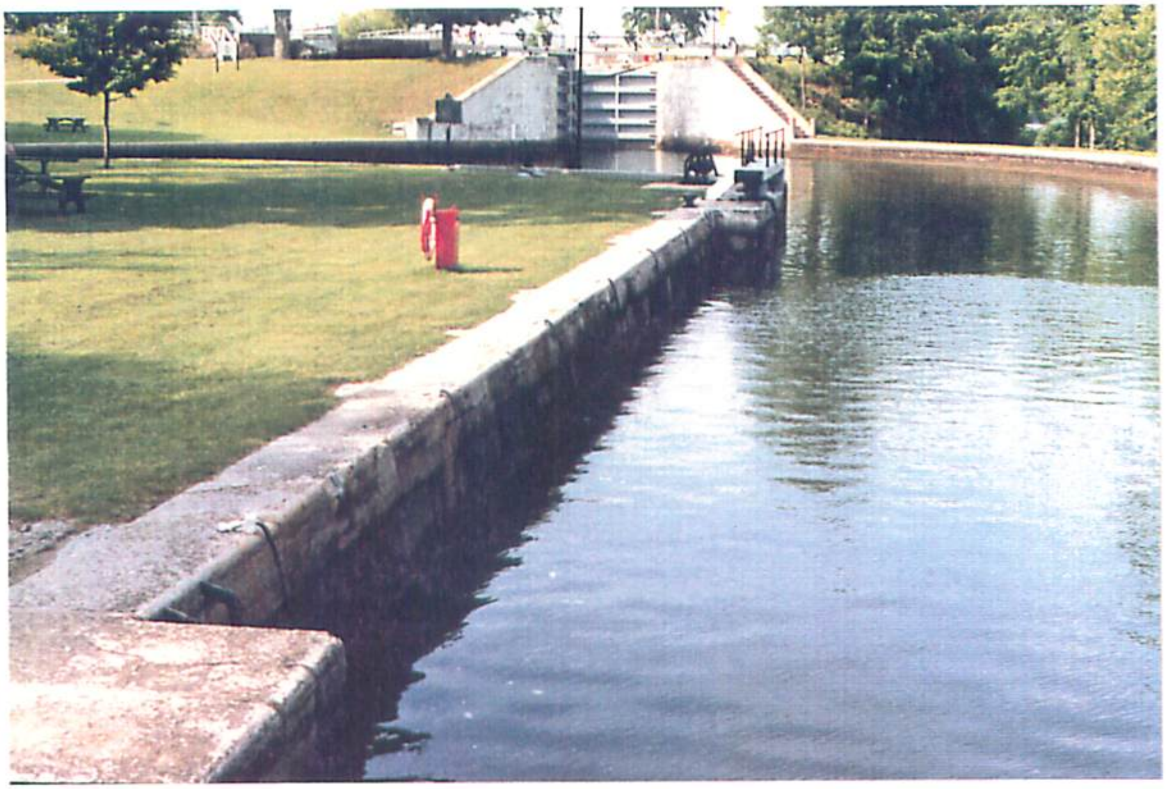


Photograph No. 10 Coring of Hole No. H15

Photograph No. 11 Coring of Hole No. H17

LOCK 47

SHOWING CONDITIONS



Lock 47 West Wall - Looking North

Lock 47 West Wall - Close Up View
Note Running Leak Betwen Boats On West Wall

Lock 47 East Wall - External Seepage
Note Gushing Leak

Lock 47 East Wall - External Seepage
Note Flowing Water

Lock 47 East Wall - External Seepage
Close Up Of Flowing Water Leak

Lock 47 East Wall - External Seepage

Lock 47 East Wall - External Seepage
Note Water Bubbling Up From Below Surface

Lock 47 East Wall - External Seepage

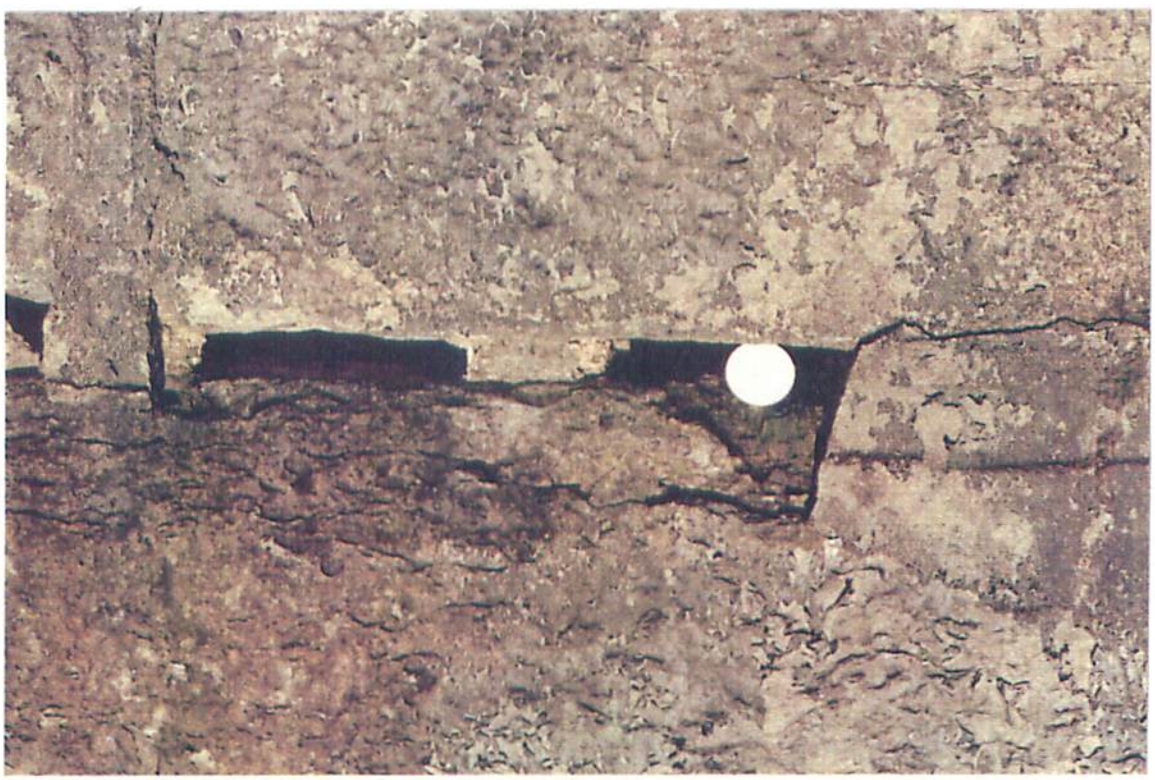
Lock 47 East Wall - External Seepage

Lock 47 East Wall - External Seepage

Lock 47 - Upstream Sill

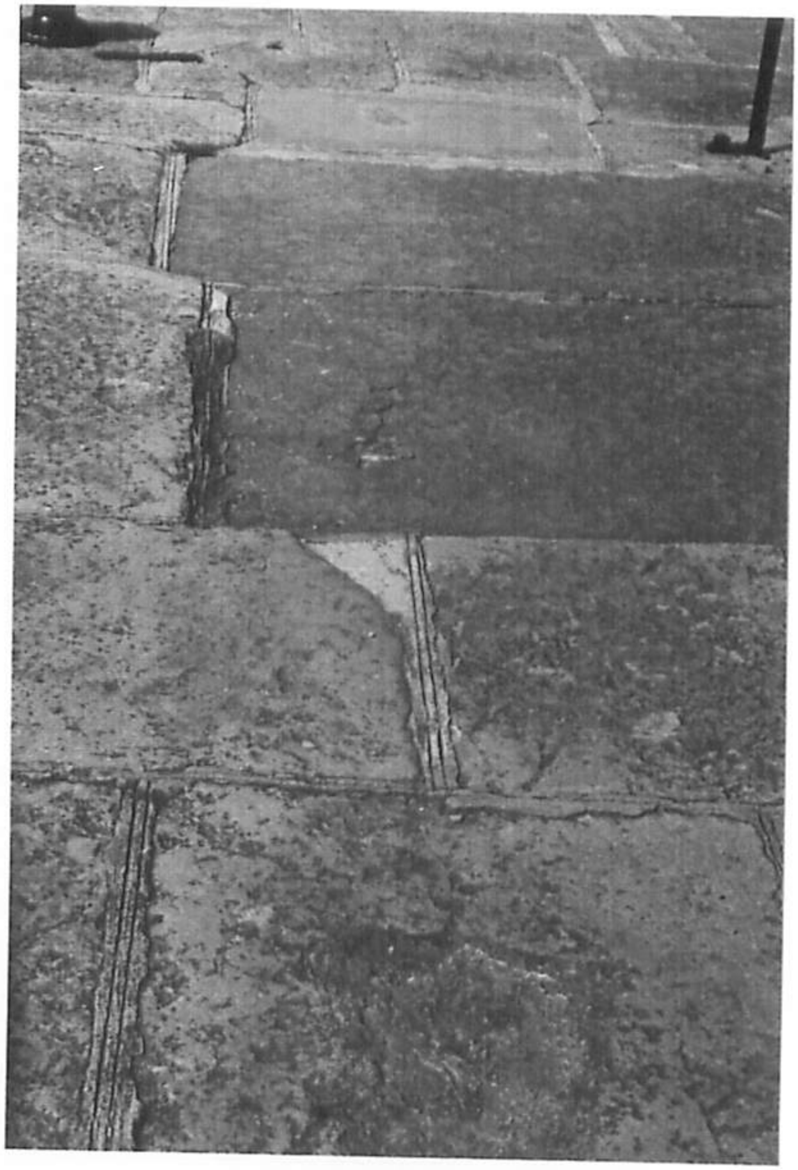
Lock 47 - Upstream Sill
Note Water Boils On Top Of Sill

Lock 47 - Showing Open Joints



Lock 47 - Showing Open Joints
Note 25 Cent Piece In Open Joint

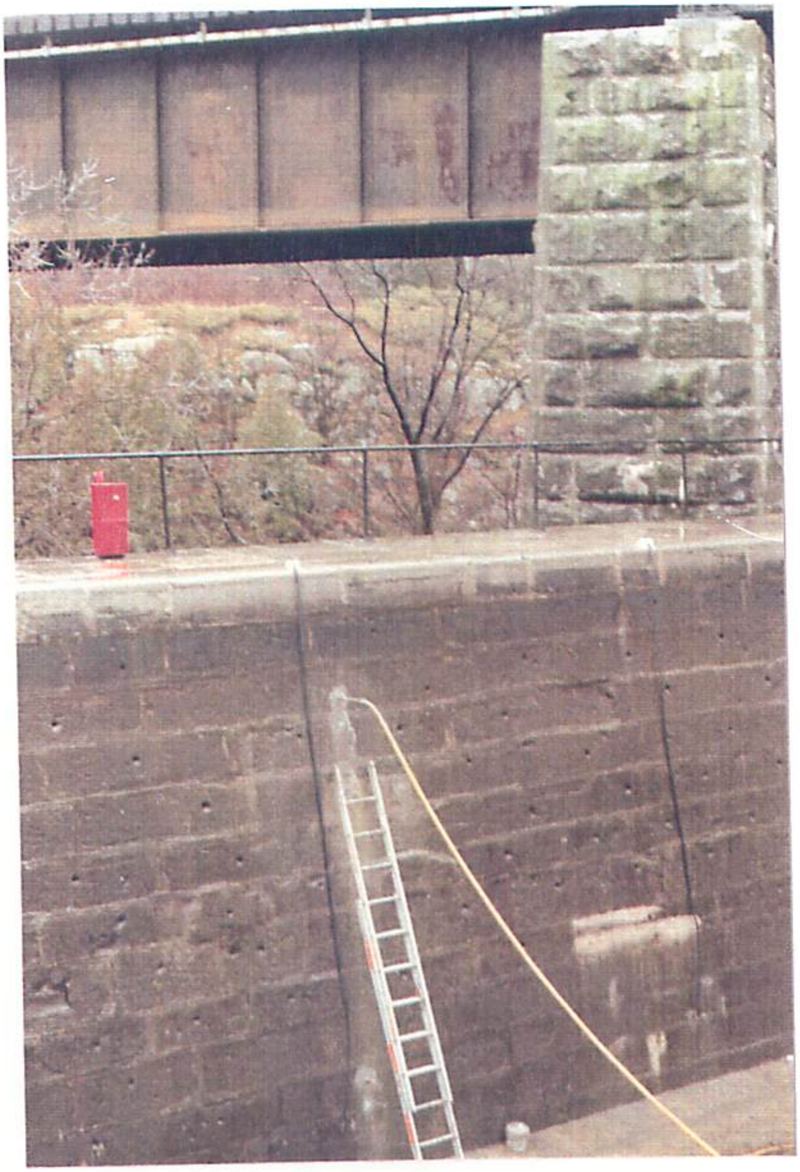
Lock 47 - Showing Open Joints



Lock 47 - Evidence of Previous Grouting Operation
- Unsuccessful Due To Unsuitable Methodology & Product Selection

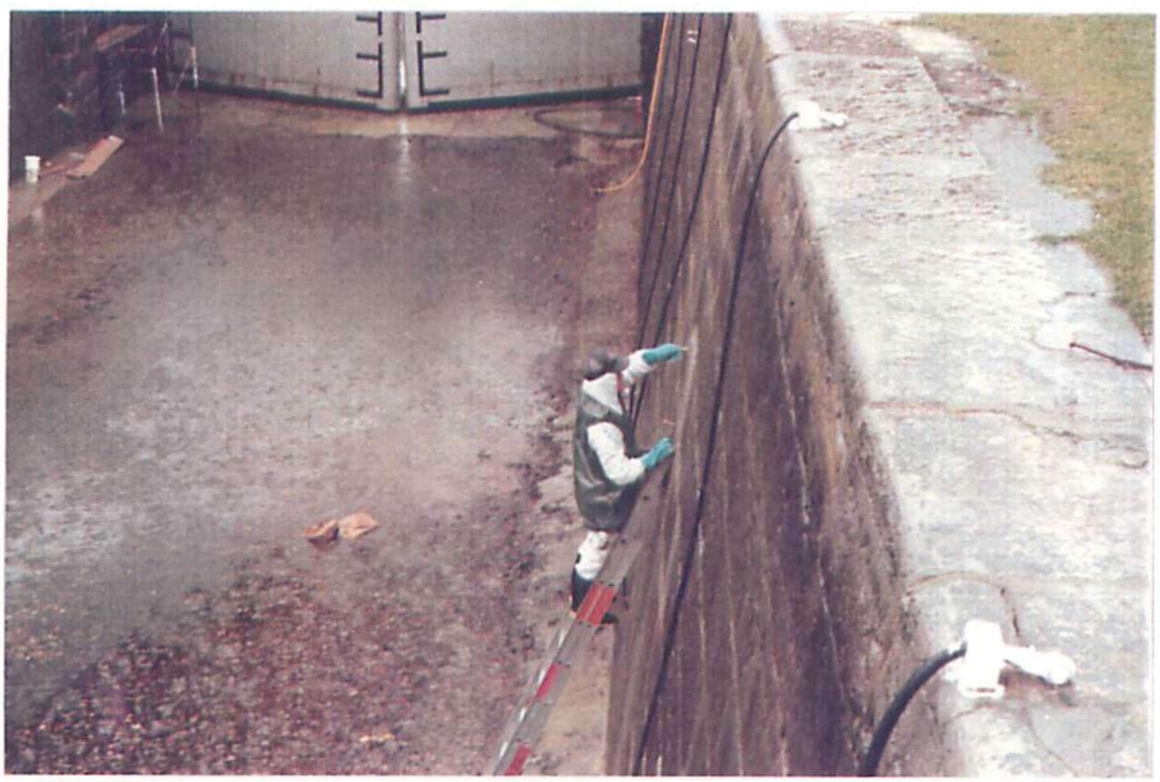
Lock 47 - East Wall
Dish-Shaped Settlement Due To Loss Of Fines

Lock 47 - West Wall Near Test Hole H - 1
Showing Grout Coming To Surface While Grouting

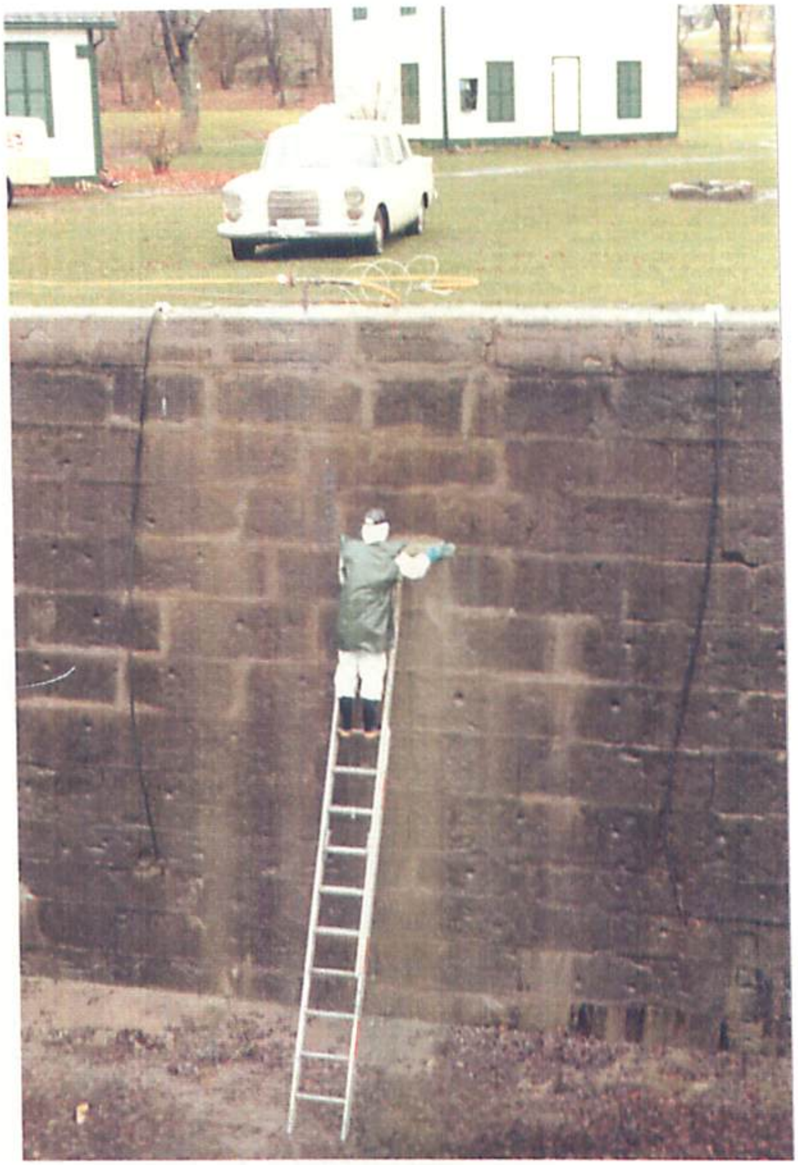


Lock 47 - East Wall
Grouting Test Hole H - 11 With Type C Flyash Formulation

Lock 47 - On West Wall Looking East At Test Hole V - 1
Showing Top Of Hole Assembly & Pressure Gauge While Grouting



Lock 47 - West Wall Near Test Hole V - 1
Repointing Prior To Grouting With Fast Setting Cement Mortar

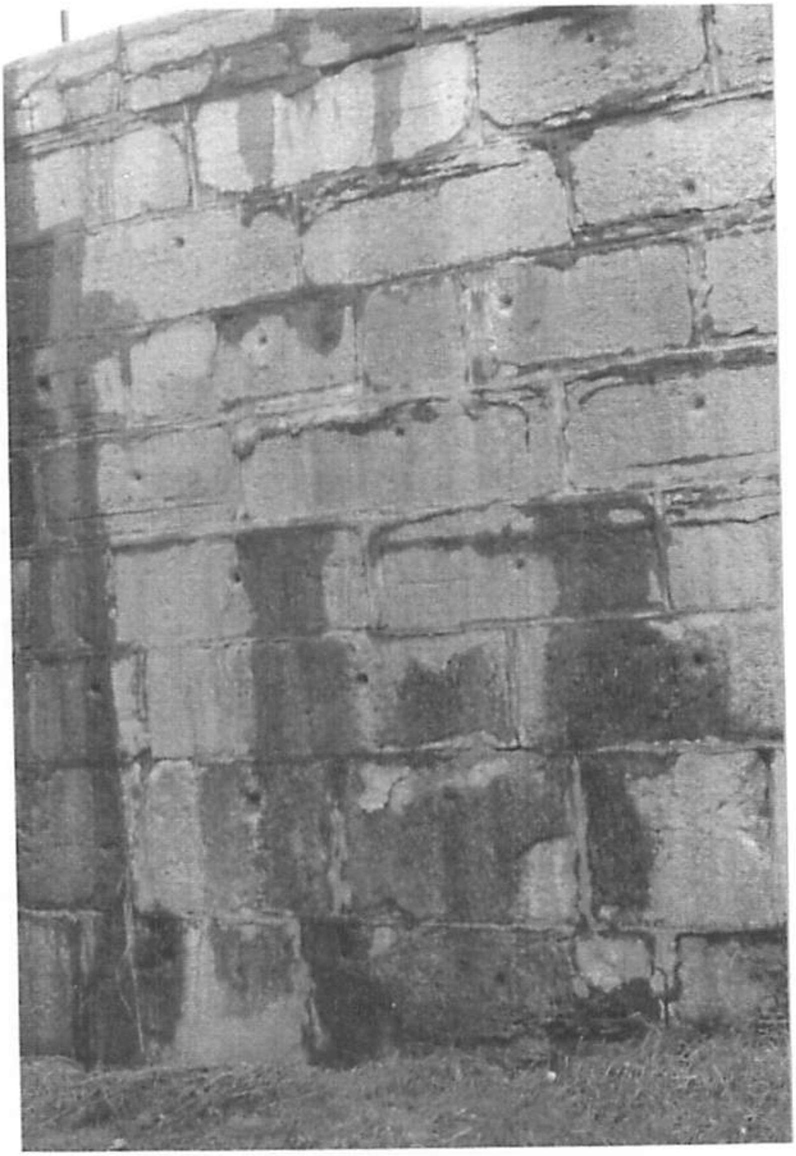


Lock 47 - West Wall Below Test Hole V - 1
Repointing With Fast Setting Cement Mortar

Lock 47 - West Wall Downtream Abutment At Test Hole V - 2
Showing Installation Of Grout Pipes

Lock 47 - East Wall Downstream Abutment
Grouting Manifold At Test Hole V - 5

Lock 47 - East Wall Downstream Abutment
Results Of Repointing & Grouting At Test Hole V - 5



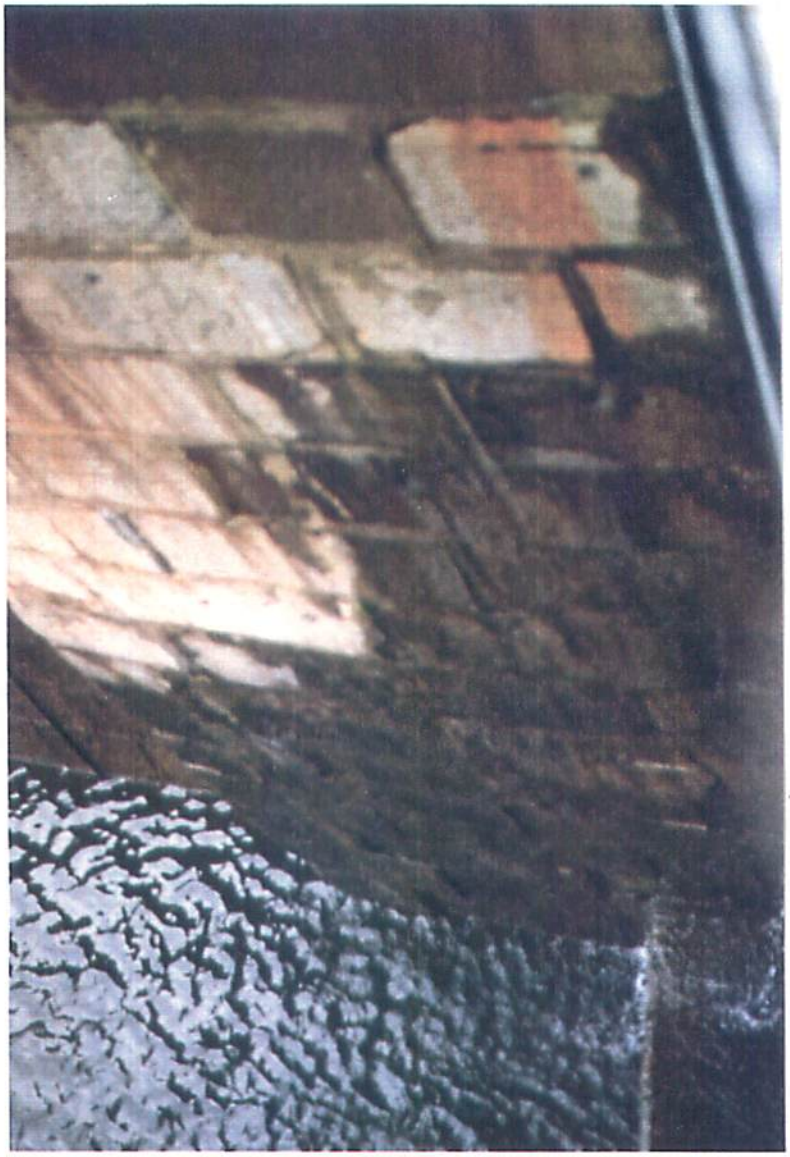
Lock 47 - Exterior East Wall At Midpoint
Shows Leaking Grout 3 Metres Away From Test Hole H - 11

Trow

LOCK 48
SHOWING CONDITIONS

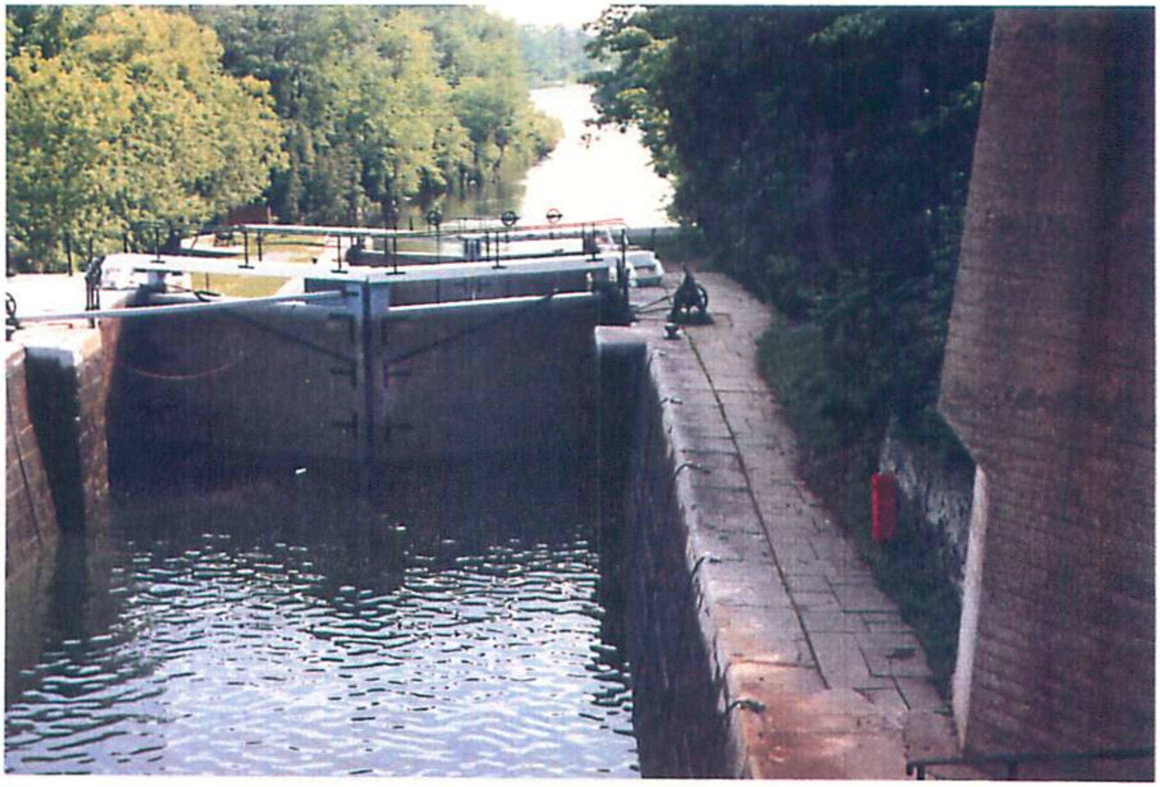
Lock 48 - East Wall Upstream Abutment
Note Movement Of Masonry Blocks Due To Freeze/Thaw Action

Lock 48 - Flood Gate In East Wall Upstream Abutment
Note Open Joints Between Masonry Blocks



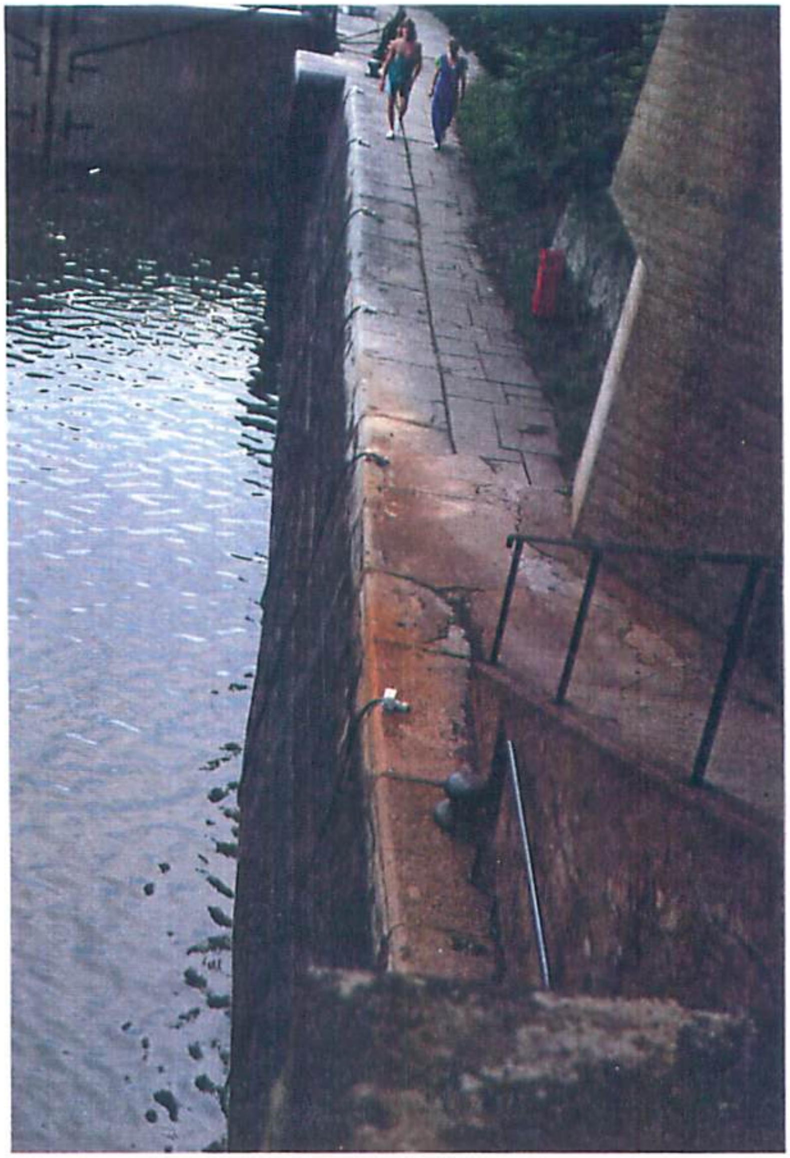
Lock 48 - West Wall At Upstream Gates
Showing Seepage Through Abutment from Lock 47

Lock 48 - West Wall At Upstream Gates
Showing Seepage Through Abutment from Lock 47



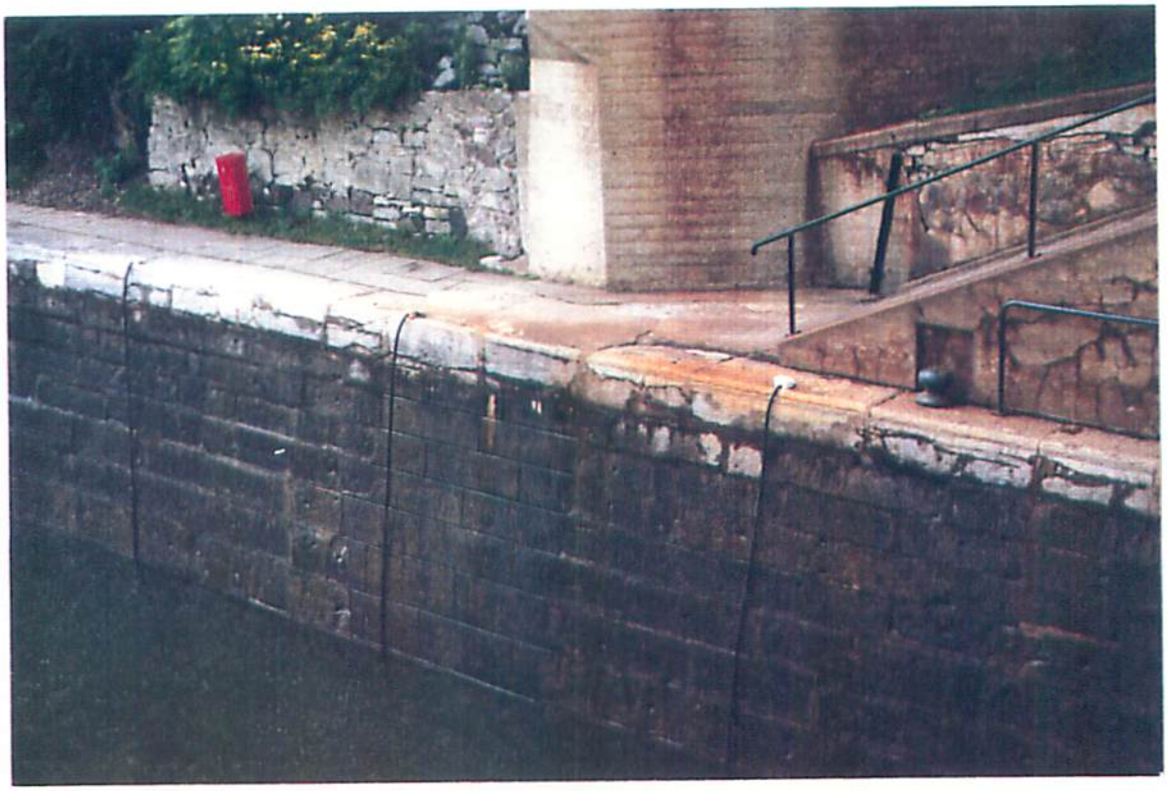
Lock 48 - West Wall Looking Downstream

Lock 48 - East Wall Exterior
In Much Better Condition Than Lock 47



Lock 48 - West Wall
Showing Bulge Due To Freeze/Thaw Action

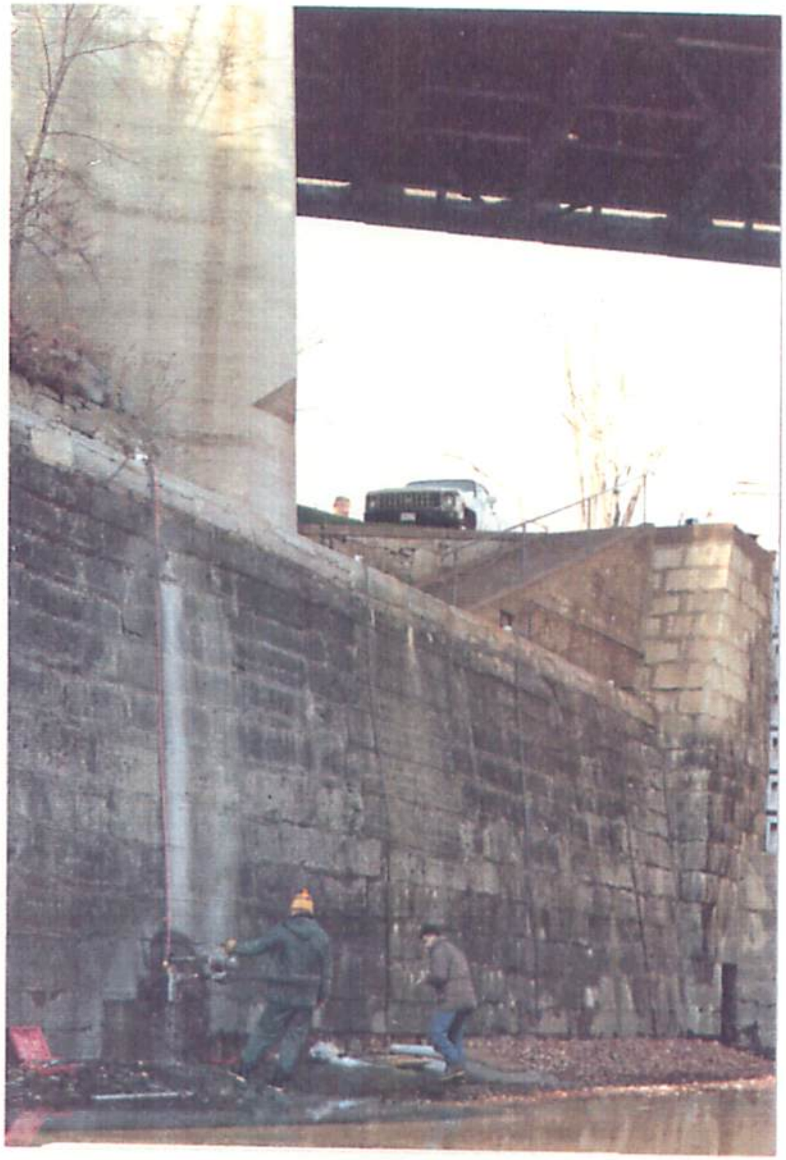
Lock 48 - West Wall
In Area Of Bulge



Lock 48 - West Wall
In Area Of Bulge

Lock 48 - West Wall & Upstream Sill
Note Drilling Of Test Hole H - 7 At Left

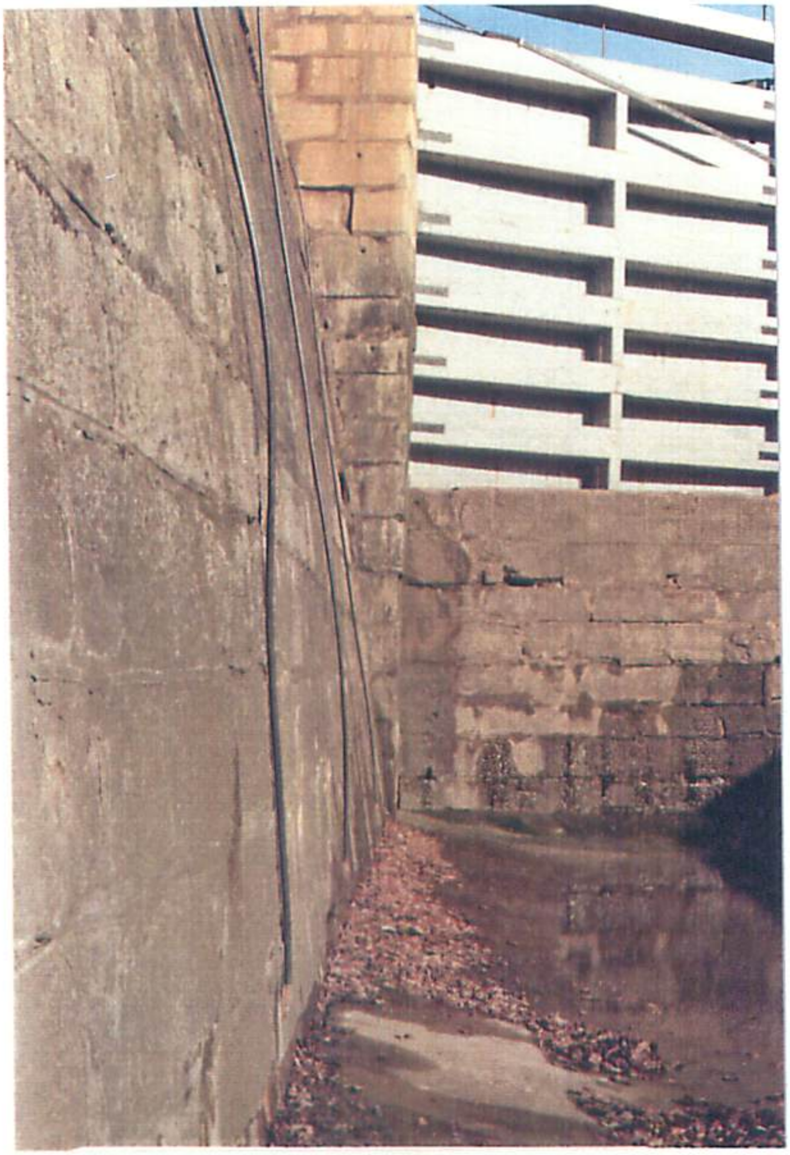
Lock 48 - West Wall Looking South
Showing Open Joints and Block Movement In Foreground
Note Bulge & Water Draining From Test Hole H - 7 In Background



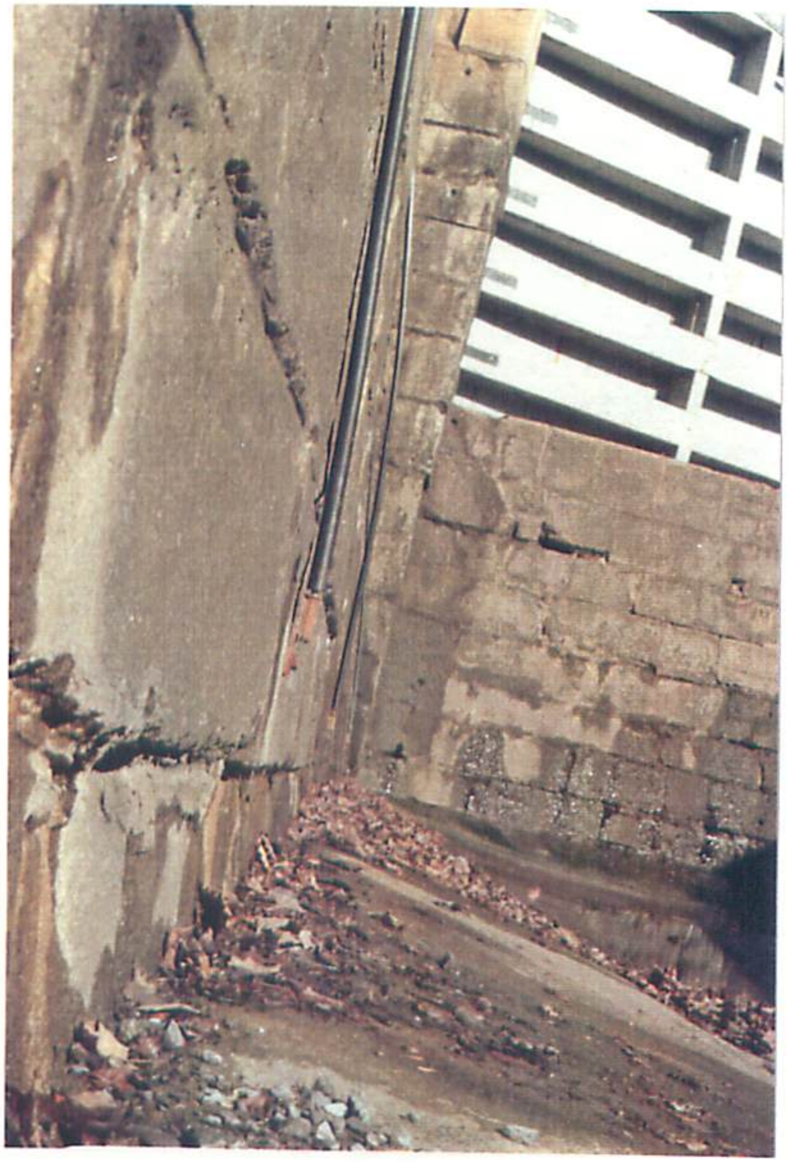
Lock 48 - West Wall
Drilling Test Hole H - 7 With H - 6 Above
Note Bulge In Centre Of Photograph

Lock 48 - West Wall

Test Hole H - 6 In Vicinity Of Bulge



Lock 48 - West Wall
Showing Bulge Due To Freeze/Thaw Action
Also Note Movement Of Masonry Blocks
And Poor Condition Of Sill With Open Joints & Seepage

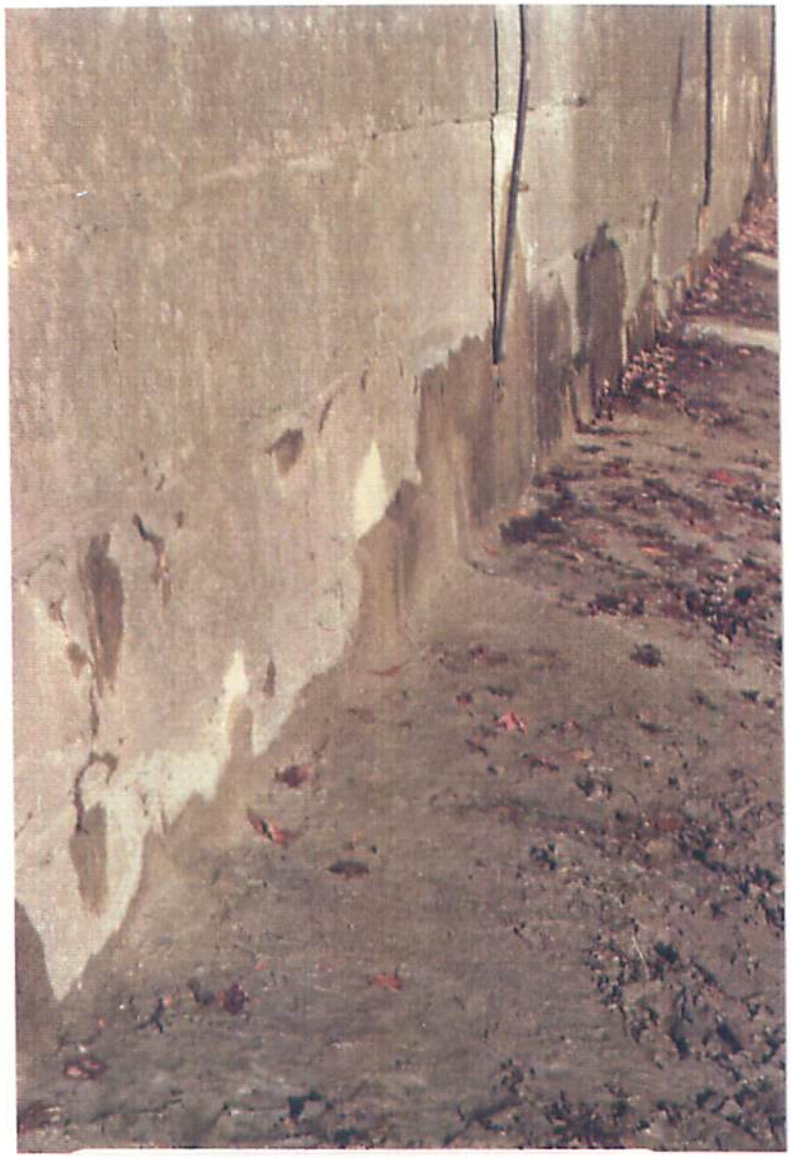


Lock 48 - West Wall
Close Up Showing Bulge Due To Freeze/Thaw Action
Also Note Movement Of Masonry Blocks
And Poor Condition Of Sill With Open Joints & Seepage

Lock 48 - West Wall

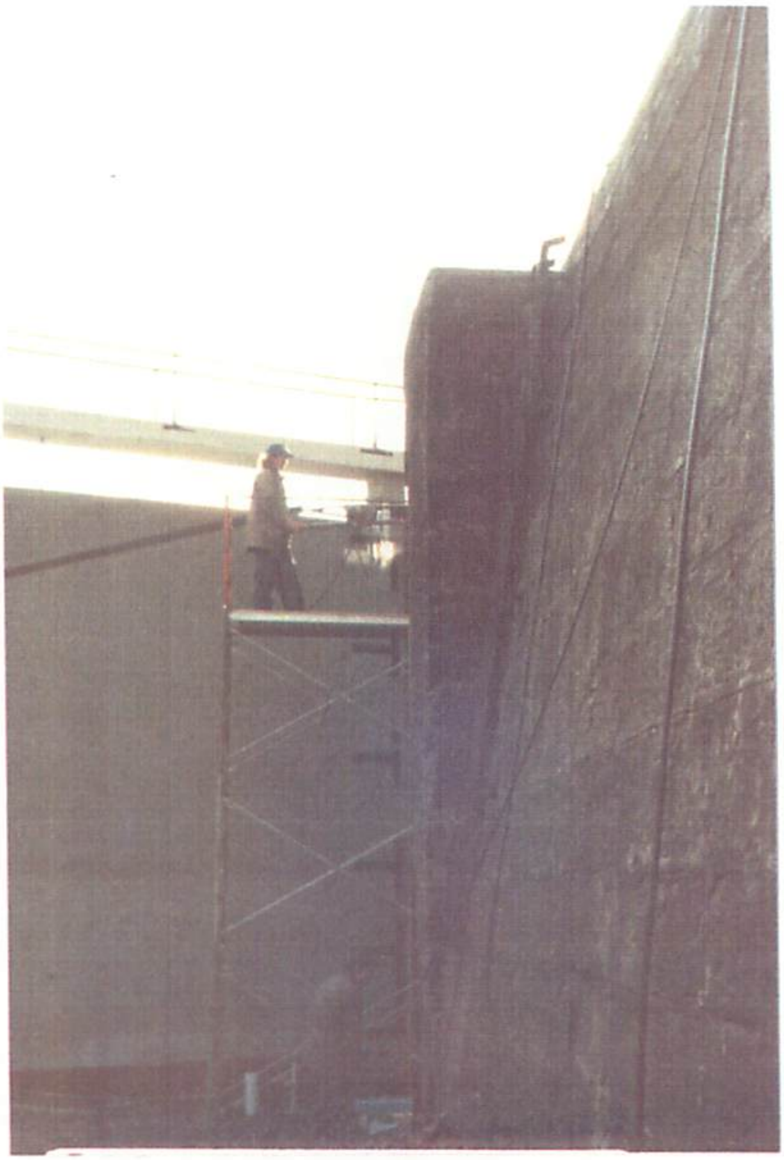
Test Hole H - 7 Showing Free Flow Of Water

Note Seepage Beneath Pointing & Missing Jointing



Lock 48 - West Wall

Test Hole H - 7 Showing Free Flow Of Water



Lock 48 - West Wall Downstream Abutment
Drilling Test Holes H - 8 (Upper) & H - 10 (Lower)

Lock 48 - West Wall Downstream Abutment
Drilling Test Hole H - 8

Lock 48 - West Wall Downstream Abutment
Showing Grout Leak Through Joint 2 m From Test Hole H - 9

Lock 48 - Exterior East Wall Upstream Abutment
Showing Water Leak Through Masonry Prior To Grouting

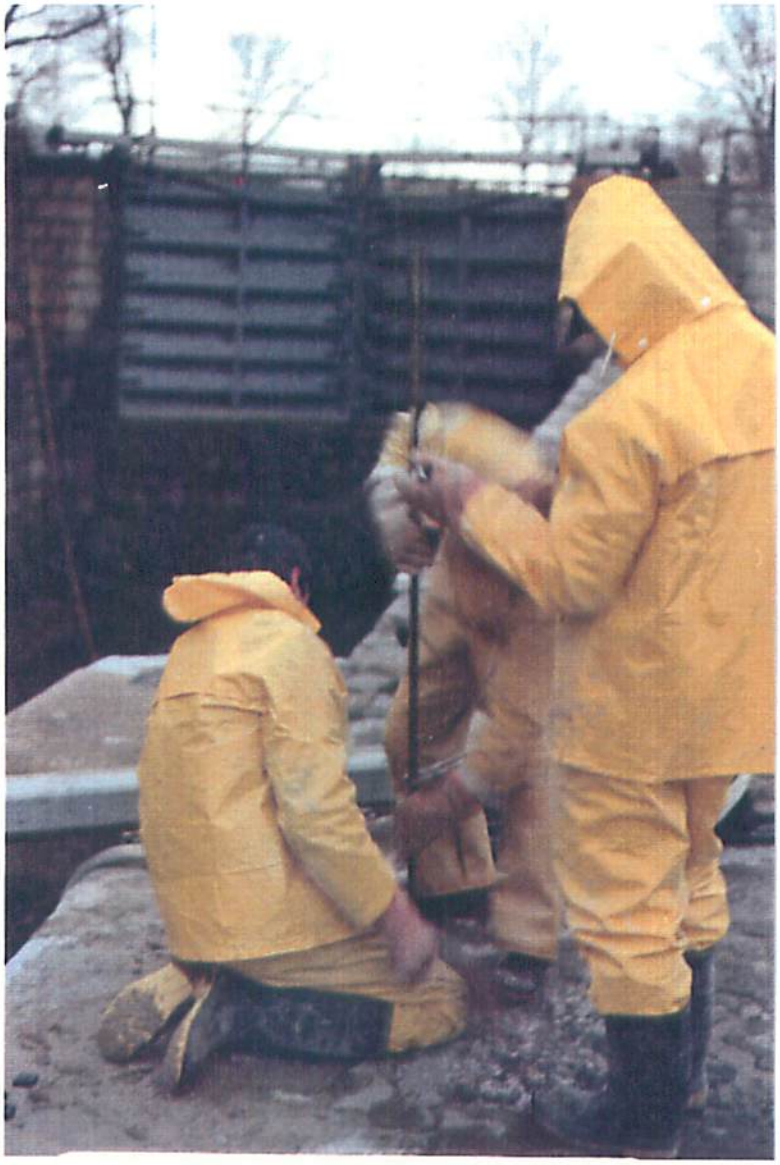
Lock 48 - Exterior East Wall Upstream Abutment
Showing Grout From Test Hole H - 15 (125 mm Below Surface)

Lock 48 - Exterior East Wall Downstream Abutment
Test Hole H - 18 (At Left) With Joints Leaking While Grouting

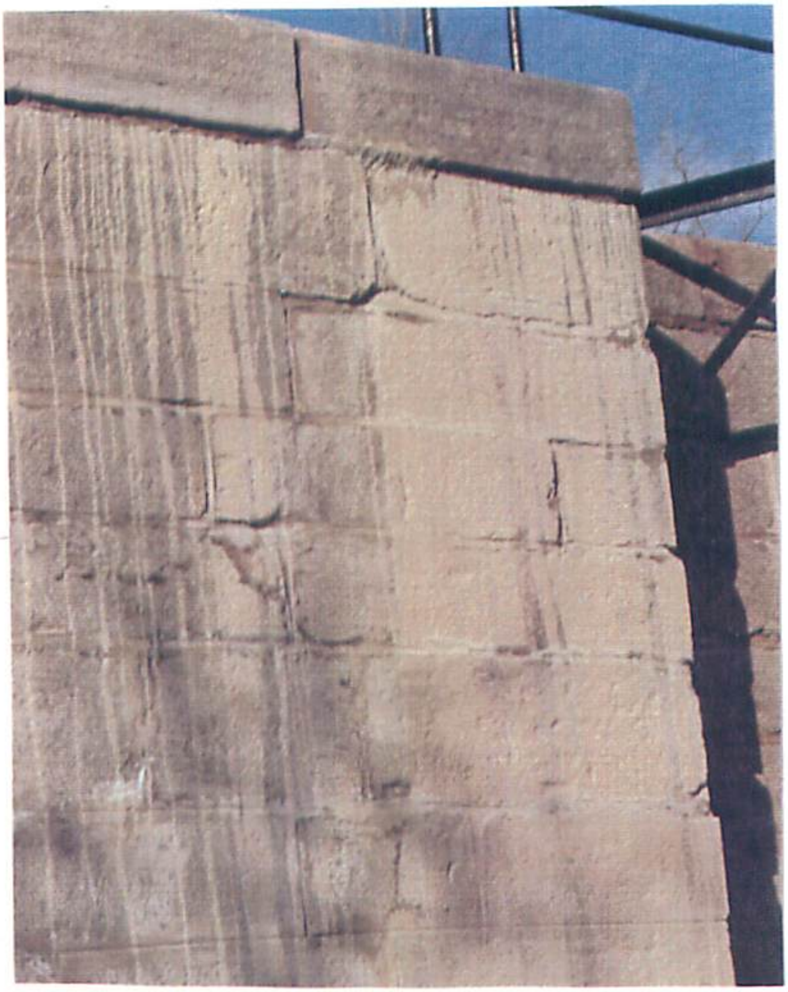
Lock 48 - Exterior East Wall Downstream Abutment
Successful Plugging Of Leaks By Grouting At Test Hole H - 20

Lock 48 - West Wall Near Test Hole V - 3
Showing Seepage Prior To Grouting

Lock 48 - West Wall Near Test Hole V - 3
Showing Seepage Prior To Grouting



Lock 48 - East Wall Downstream Abutment
Pulling Grout Pipe At Test Hole V - 6



Lock 48 - Exterior East Wall Downstream Abutment
Shows Leaking Grout From Test Hole H - 18

Lock 48 - East Wall Downstream Abutment
Grouting At Test Hole V - 6

GROUTING PLANT & EQUIPMENT

Colloidal Grouting Plant Shown During Test Program
Kingston Mills - Nov 1989

General Arrangement Of Grouting Material And Equipment
At Lock 47 - West Wall Downstream Abutment Test Hole V - 2

Colloidal Grouting Plant Shown During Test Program
Kingston Mills - Nov 1989

Colloidal Mixer on Left; Agitator on Right
Grout Pump At The Rear

Single Inflatable Packer Assembly
Showing Inflatable Packer (At Left) & Grout Pipe (Black)
With Inflation Tube (White) Taped To Grout Pipe

Testing Inflatable Packer (Shown Inflated)

APPENDIX

D-4 1999 QUONTACON ASSOCIATES INVESTIGATION

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GOVERNMENT OF CANADA

PARKS CANADA

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

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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.

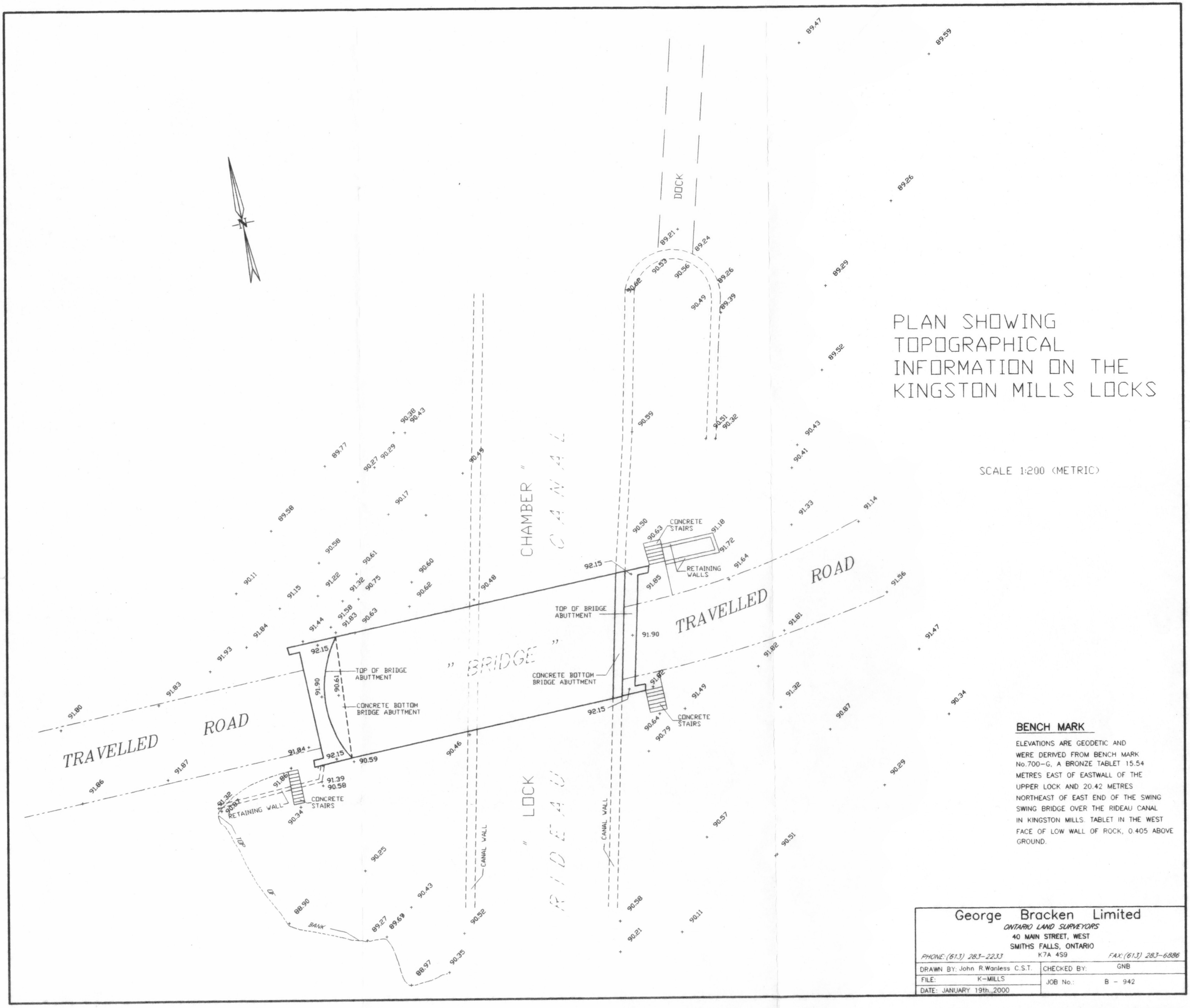
Option 2

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)

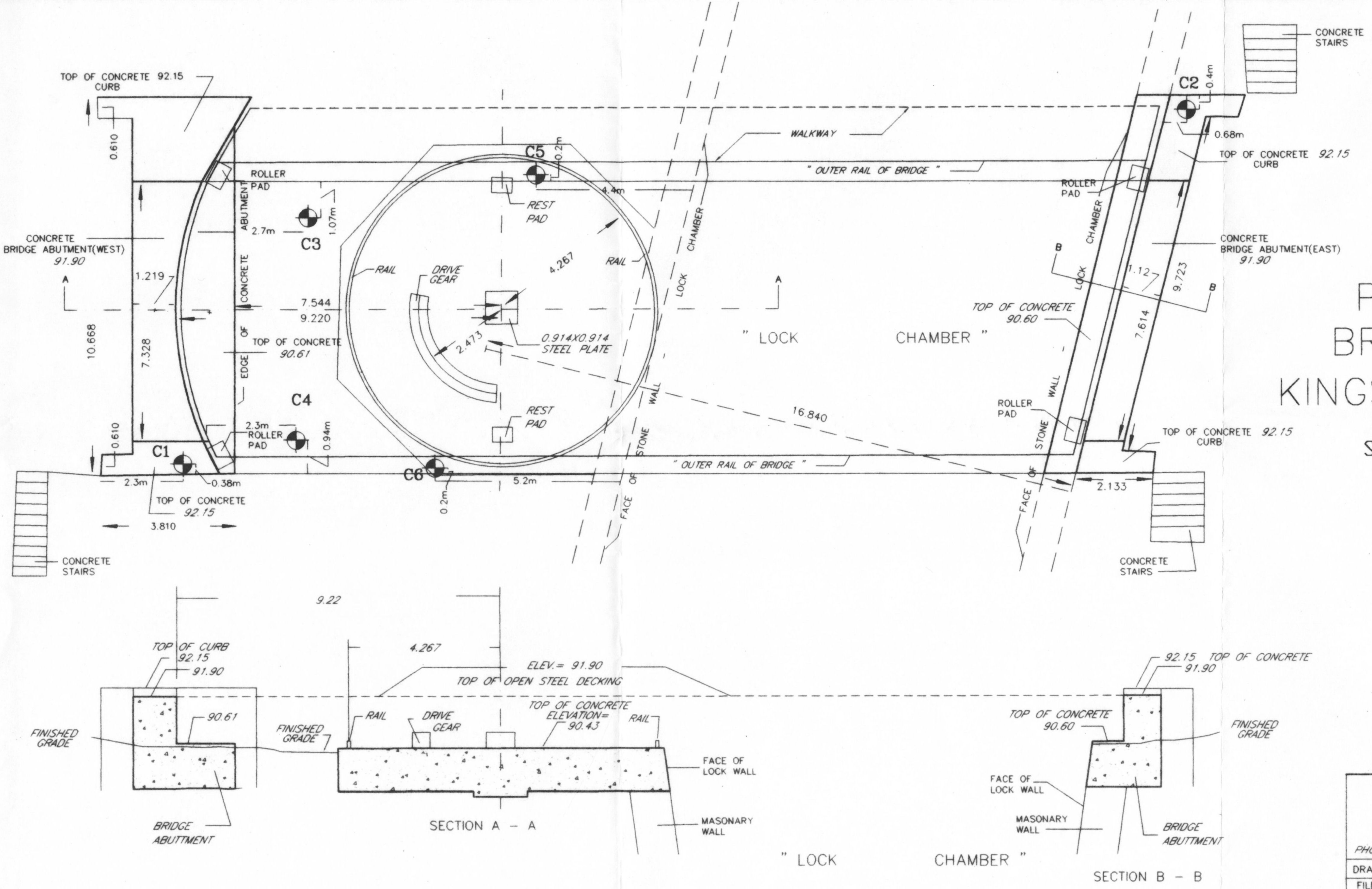
This report recommends Option 1. Based on past experience, the previous rehabilitation carried out in 1977 has served well for the past 22 years, despite concrete, not having air entrainment according to CSA Standards. Refacing with well-consolidated, quality concrete having air-entrainment according 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)
DOCK
" CHAMBER "
CANAL
" BRIDGE "
" LOCK
RIDEAU
TRAVELLED ROAD
TRAVELLED ROAD
TOP OF BRIDGE ABUTTMENT
CONCRETE BOTTOM BRIDGE ABUTTMENT
TOP OF BRIDGE ABUTMENT
CONCRETE BOTTOM BRIDGE ABUTTMENT
CONCRETE STAIRS
RETAINING WALLS
CONCRETE STAIRS
CONCRETE STAIRS
RETAINING WALL
CANAL WALL
CANAL WALL
TOP OF BANK
N
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
40 MAIN STREET, WEST
SMITHS FALLS, ONTARIO
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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(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:	<u>Material</u>	<u>Percentage</u>
	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: <u>Material</u>
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,

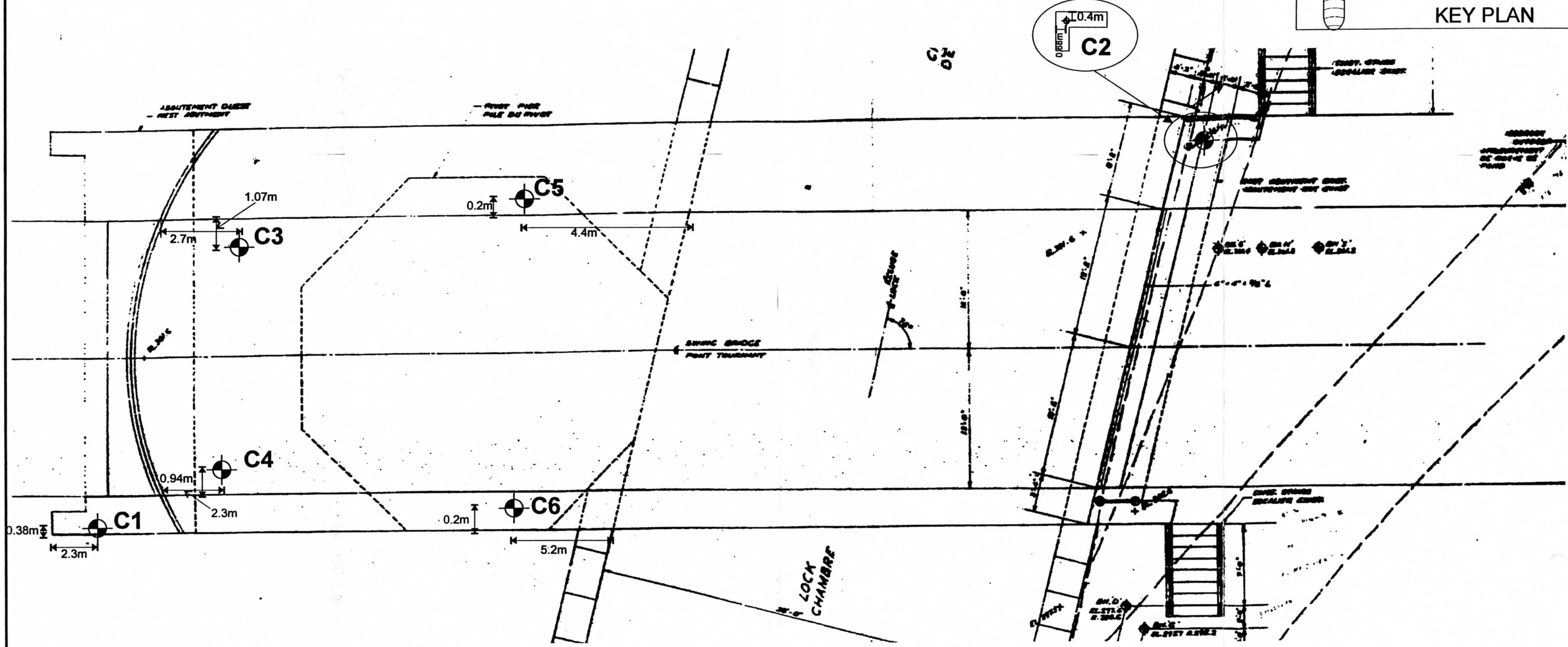
INSPEC-SOL INC.

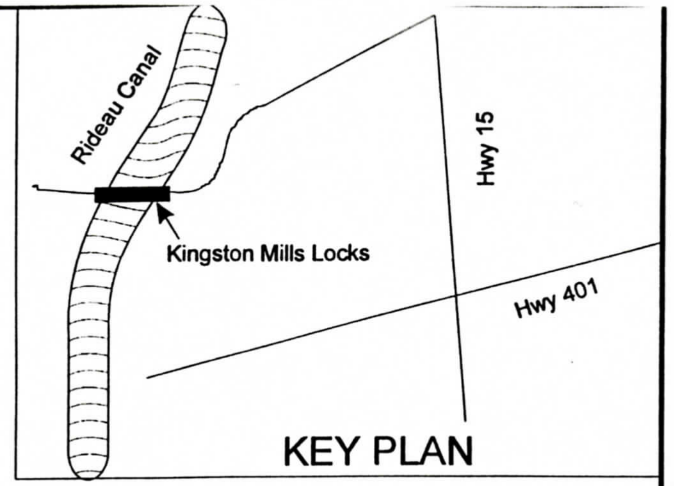
Prepared by: Myles Carter

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

By fax/mail (514) 694-5835

MC/lp





INSPEC-SOL

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**

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

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 N	COMMENTS
		302.4		GROUND SURFACE							
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
5	1,0			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.
8.0	2,0			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".							
15	4,0										

REFERENCE No.: 20226-K-6160B — ENCLOSURE No.: 2

INSPEC-SOL

BOREHOLE No.: C-2
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 30, 1999 — DATE (FINISH): December 1, 1999

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

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 N	COMMENTS
		302.4		GROUND SURFACE							
0 – 6.0	0 – 1,0			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.
6.0 – 12.3	2,0 – 3,0			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.
12.3 – 15	4,0			END OF BOREHOLE. NOTE: SAMPLE TAKEN FOR COMPRESSIVE STRENGTH TEST BETWEEN 1'2" AND 1'10".							

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. NO MICRO FISSURES OR REACTION RIMS VISIBLE. DELAMINATION AT 4" FROM SURFACE.		RC-1	100		100%		COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 55% CRYSTALLINE LIMESTONE: 5 % ARGILLACEOUS LIMESTONE: 40 % FINE AGGREGATE: 0- ¼", SUB ANGULAR TO SUB ROUNDED, GRANITIC.
				END OF BOREHOLE							

Depth scale: Feet 0, 1.0, 5, 10, 15; Metres 0, 1,0, 2,0, 3,0, 4,0

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		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 – 1.3	0			ASPHALT 1½", THICK, NO MEMBRANE PRESENT. CONCRETE WITH AIR ENTRAINMENT, NORMALLY CONSOLIDATED. NO MICRO FISSURES OR REACTION RIMS VISIBLE. DELAMINATION AT 10" FROM SURFACE. 5/8" ROUND, DEFORMED REBAR AT 4" Ø FROM SURFACE. GOOD BOND WITH PASTE, NO CORROSION.		RC-1	100		100%		COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 25% CRYSTALLINE LIMESTONE: 10 % ARGILLACEOUS LIMESTONE: 65 %
				END OF BOREHOLE							
				NOTE: SAMPLE SUBMITTED FOR COMPRESSIVE STRENGTH TESTING.							

Depth scale: Feet 0, 5, 10, 15; Metres 0, 1,0, 2,0, 3,0, 4,0

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 (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
Feet	Metres	296.62		GROUND SURFACE			%			N	
0–5.5	0–1,0			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.
5.5–7.5	2,0			POSSIBLE FILL: BROWN CLAYEY SILT TILL, WITH TRACES OF GRAVEL, MOIST		SS-1	100				
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".

Depth scale: Feet 0, 5, 10, 15; Metres 0, 1,0, 2,0, 3,0, 4,0

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		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	296.7		GROUND SURFACE			%			N	
0	0 1,0			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.		RC-1	100		COMP.STR. SAMPLE 93% AIR VOID SAMPLE TENSILE 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.
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											

REFERENCE No.: 20226-K6160

Rideau Canal- KINGSTON MILLS LOCKS.

CONCRETE CONDITION INVESTIGATION

KINGSTON, ONTARIO

Photo No. 1-

CORE NUMBER C-1

Photo No. 2

CORE NUMBER C-2

INSPEC-SOL

FO-010.09-IA/08-95

Rideau Canal- KINGSTON MILLS LOCKS.

CONCRETE CONDITION INVESTIGATION

KINGSTON, ONTARIO

Photo No. 3

CONCRETE CORE C3 AND C4

REFERENCE No.: 20226-K6160

Rideau Canal- KINGSTON MILLS LOCKS.
CONCRETE CONDITION INVESTIGATION
KINGSTON, ONTARIO

Photo No. 4

CORE NUMBER C-5

Photo No. 5

CORE NUMBER C-6

INSPEC-SOL

FO-010 09-IA/08-95

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

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

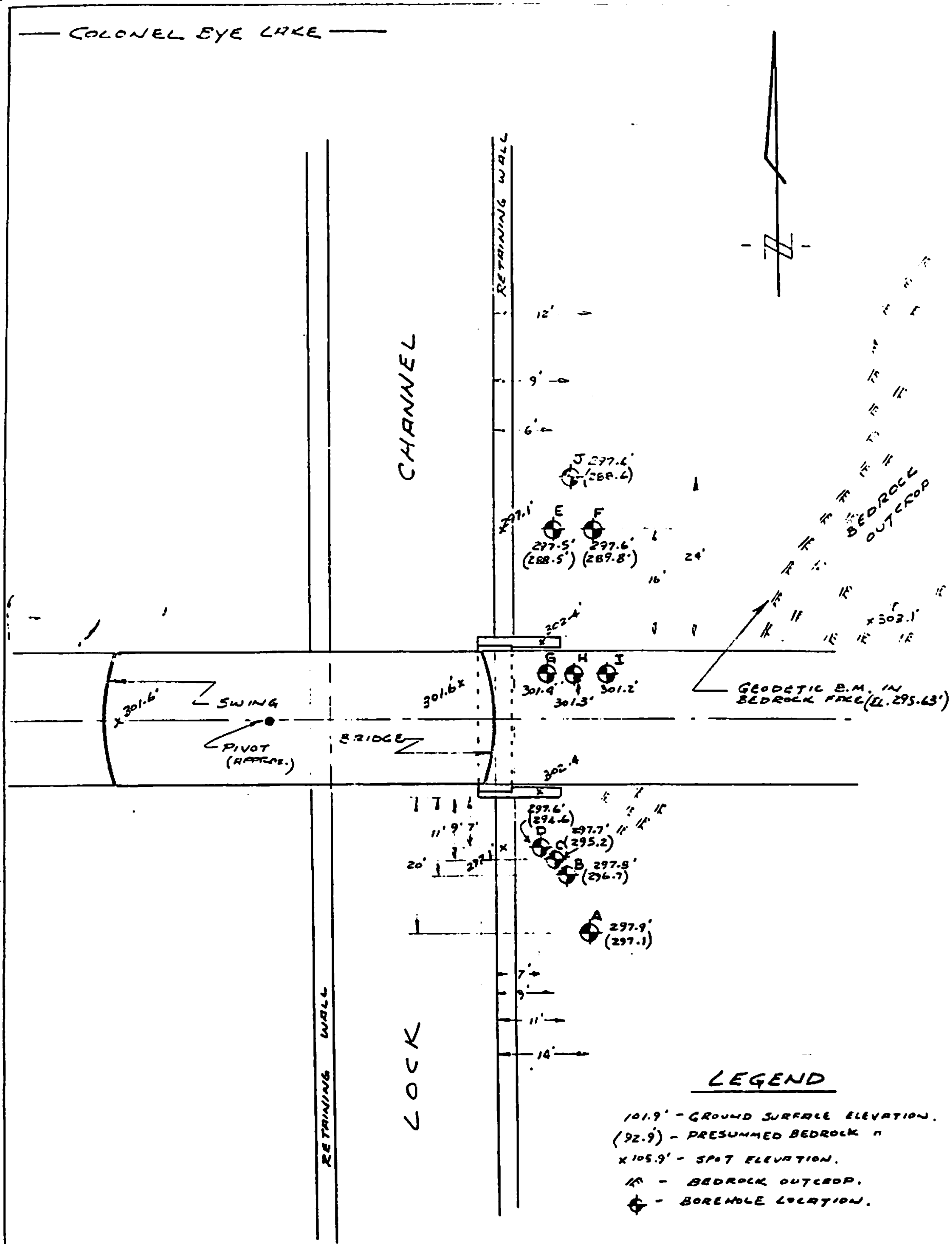
4 c.c.

TABLE 1

KINGSTON MILLS SWING BRIDGE

SUMMARY OF PROBE HOLES

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



SITE INVESTIGATION SERVICES LIMITED

KINGSTON MILLS SWING BRIDGE ABUTMENT STUDY	SCALE: 1" = 20' ±	DATE: JAN. 1977
— LOCATION PLAN —	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 - 3¼" H.S.A. (D-8)

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

LEGEND
Gravel, Sand, Silt, 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	ELEVATION IN FEET	DEPTH IN FEET
	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	
REFUSAL OF CONE PROBE		10
		15
		20

Columns: SOIL SYMBOL; MOISTURE CONTENT and ATTERBERG LIMITS (%) — Plastic Limit, Moisture Content, Liquid Limit; LAB. TESTS; SAMPLE TYPE AND NUMBER

— 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

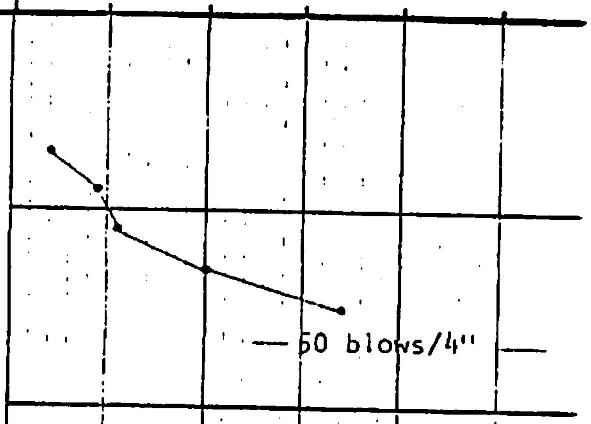
Date - December 23, 1976
Elevation Datum -
Type of Drill - 3¼" H.S.A. (D-8)

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	MOISTURE CONTENT and ATTERBERG LIMITS (%) — Plastic Limit, Moisture Content, Liquid Limit	LAB. TESTS	SAMPLE TYPE AND NUMBER
		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						
CONE PROBE REFUSAL		293.0				
			10			
			15			
			20			

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 - □



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

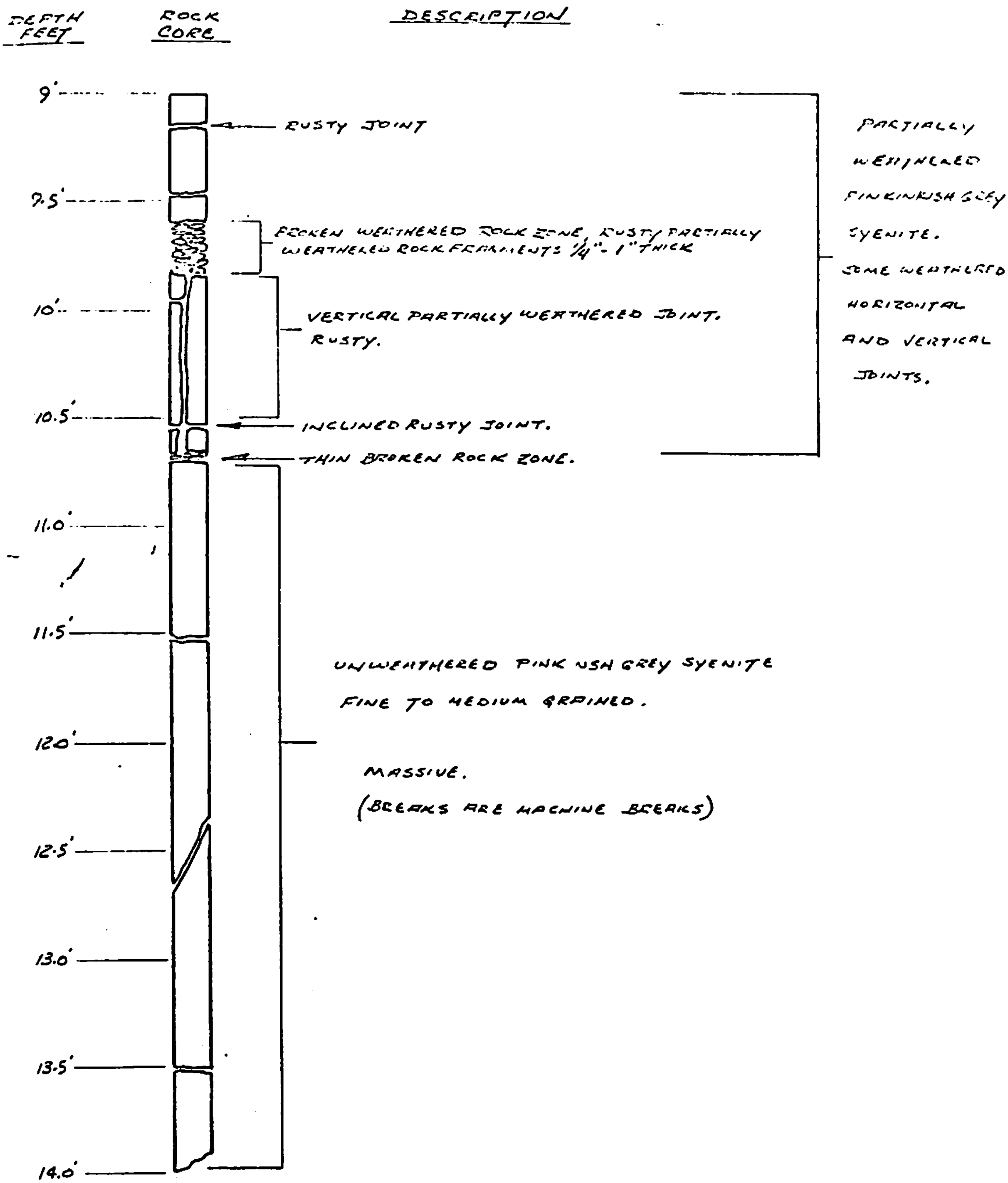
LEGEND
Gravel, Sand, [silt symbol], Clay
(See Appendix "A" for Other Symbols)

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 - □

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

SOIL DESCRIPTION	SOIL SYMBOL	ELEVATION IN FEET	DEPTH IN FEET	LAB. TESTS	SAMPLE TYPE AND NUMBER	PENETRATION RESISTANCE / SHEAR STRENGTH
		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	98+% Recovery
END OF HOLE (Note: standpipe installed to 14' depth)			15			
			20			



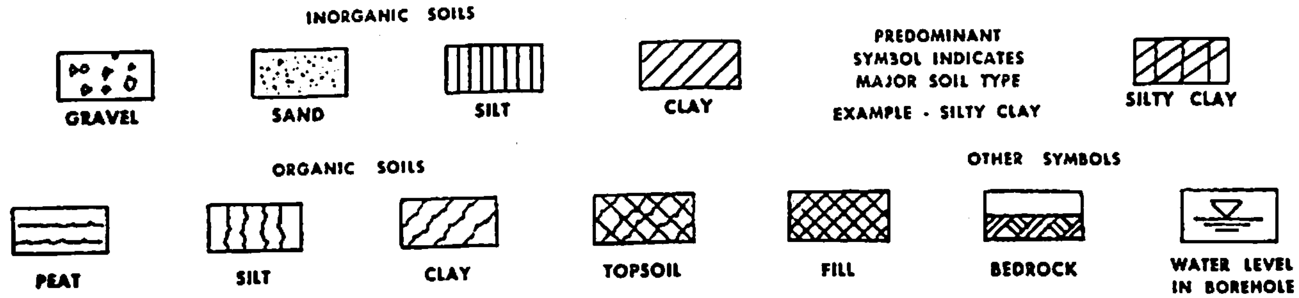
SITE INVESTIGATION SERVICES LIMITED

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

EXPLANATION OF SYMBOLS AND TEST DATA

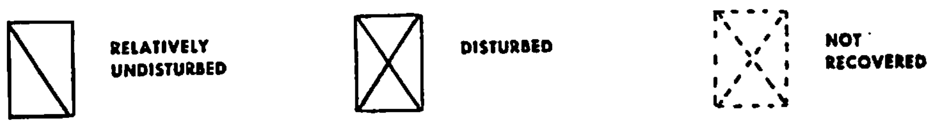
SOIL DESCRIPTION

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:



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_l - 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] seasonal 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

APPENDIX

D-5 2005 JACQUES WHITFORD INVESTIGATION

Jacques Whitford

Engineering, Scientific, Planning and Management Consultants

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Ottawa Ontario
Canada K1B 1A7

Bus 613 738 0708
Fax 613 738 0721

www.jacqueswhitford.com

April 8, 2005

Project File: ONO11783

Ms. Kathleen Murphy
Mechanical Engineer
Heritage Canals & Engineering Works
2630 rue Sheffield Road
Ottawa, ON K1B 3V7

Dear Ms. Murphy:

RE: Project No. 304498 – Geotechinical Investigation for the Kingston Mills Lock No. 46, Kingston, Ontario

1.0 INTRODUCTION

At your request, Jacques Whitford Limited has carried out core drilling, core examination and testing for the above noted project. The purpose of this work is to determine the extent of concrete deterioration at this site. The field drilling was carried out between January 24 and 31, 2005. OGS Drilling Inc. was hired to carry out the drilling. A Jacques Whitford representative was on site at all times to supervise the operations and collect field information. A total of eleven (11) cores were taken at locations selected by your Engineer. Specimens were selected from these cores for testing for compressive strength and Petrographic analysis. This letter has been prepared to present the factual field observations and laboratory testing results.

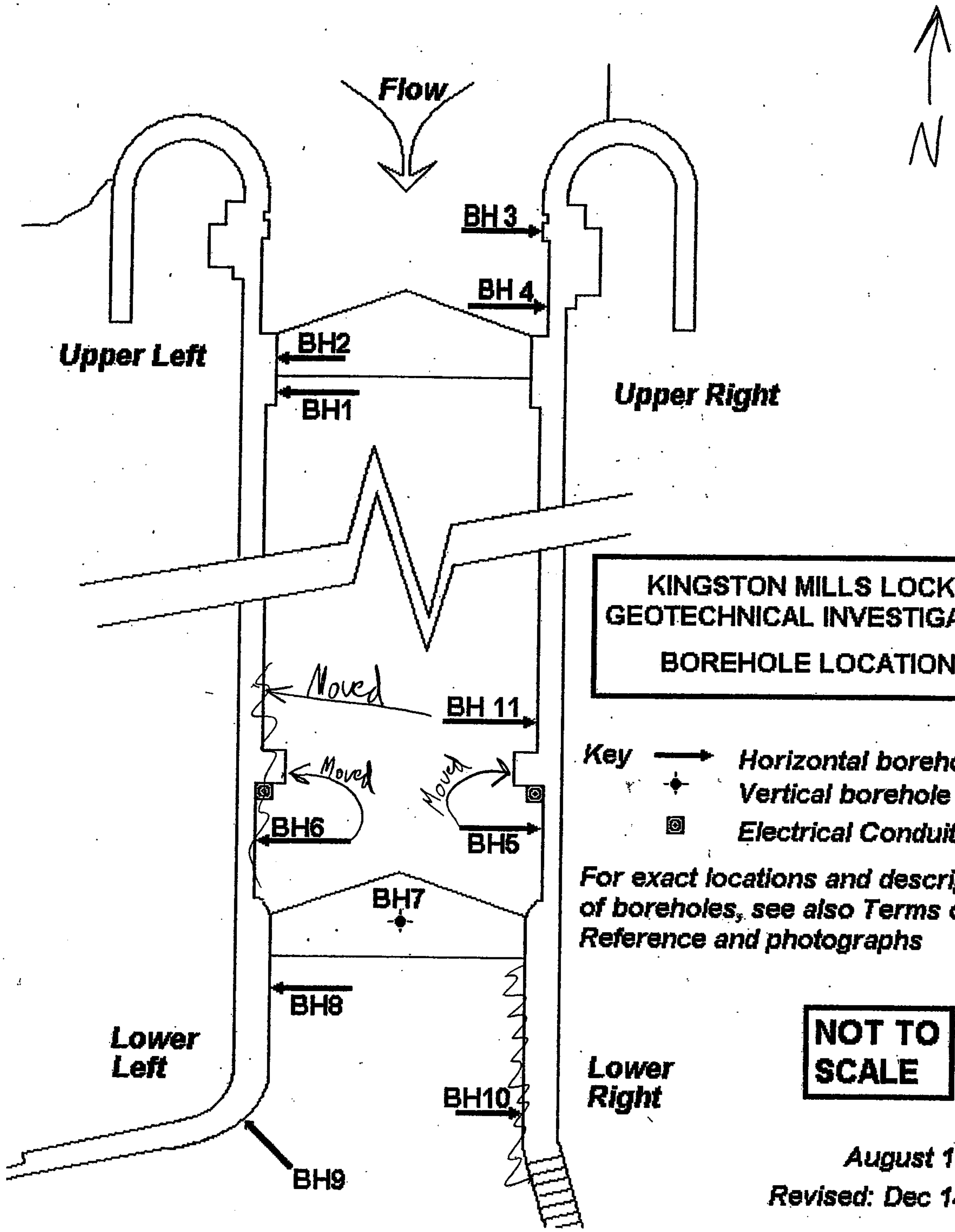
2.0 BACKGROUND INFORMATION

It is understood that Lock No. 46 at Kingston Mills was originally constructed in 1820's with masonry stone blocks as part of the Rideau Waterway. In early part of 20th century, an extensive repair was undertaken. The repair used un-reinforced masonry concrete blocks sized to the original stone blocks. These concrete blocks have undergone extensive deterioration in the forms of spalling, scaling, map cracking and efflorescence. Investigation at other locks indicated that such deterioration is limited at the surface of the blocks. The PWGSC wanted to determine to what extent the deterioration has occurred at this lock so that a repair program can be prioritized.

3.0 METHODOLOGY

3.1 Field Sampling

OGS Drilling Inc. carried out the drilling. An experienced engineering technician from JW was on site to supervise the coring process. Core information was recorded during drilling and retrieval. The engineering technician ensured the cores were properly retrieved, labeled, photographed and protected, then stored and transported to the JW laboratory in Ottawa. The core holes were backfilled using high quality non-shrinkage construction grouts.



KINGSTON MILLS LOCK 46
GEOTECHNICAL INVESTIGATION

BOREHOLE LOCATIONS

Key

Horizontal borehole

Vertical borehole

Electrical Conduit

For exact locations and descriptions of boreholes, see also Terms of Reference and photographs

NOT TO SCALE

August 12, 2004
Revised: Dec 14, 2004

BOREHLOE LOCATIONS AND DESCRIPTIONS

BH No.	Location Descriptions	Elevation m	Depth of Borehole, m
BH05-1	In first concrete block above the sluice exit	95.40	1.65 (5'5")
BH05-2	About 1 foot to 18 inches above sill, about 2 feet back from the gate quoin	96.35	1.75 (5'9")
BH05-3	2/3 or more of the way down from summer waterline	96.30	1.75 (5'9")
BH05-4	Second concrete block down from repaired coping	98.75	1.68 (5'6")
BH05-5	At gate pilaster on west wall, between ½ and 2/3 of the way down from summer waterline	95.55	0.91 (3'0")
BH05-6	At gate pilaster on east wall, between ½ and 2/3 of the way down from summer waterline	95.6	0.97 (3'2")
BH05-7	Vertically down through sill, about the middle of the sill, about 1 foot into bedrock	93.50	1.19 (3'11")
BH05-8	At or below summer waterline	95.00	0.89 (2'11")
BH05-9	Through badly spalled area below repaired concrete	99.16	1.55 (5'1")
BH05-10	Through spalled area below repaired coping	98.50	0.91 (3'0")
BH05-11	Through spalled sandstone 5th row down from summer high water line and just upstream of access ladder near pilaster on the west wall.	95.45	0.86 (2'10")

* All boreholes are horizontal except BH05-7

** Benchmark was taken at top of repair copping above BH05-4 as 100 m. All other elevations are relative to this bench mark

Appendix 2

Field Borehole Records

Jacques Whitford

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Topsoil	-	mixture of soil and humus capable of supporting good vegetative growth
Peat	-	fibrous aggregate of visible and invisible fragments of decayed organic matter
Till	-	unstratified glacial deposit which may range from clay to boulders
Fill	-	any materials below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	-	having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure
Varved	-	composed of regular alternating layers of silt and clay
Stratified	-	composed of alternating successions of different soil types, e.g. silt and sand
Layer	-	>75 mm
Seam	-	2 mm to 75 mm
Parting	-	< 2 mm
Well Graded	-	having wide range in grain sizes and substantial amounts of all intermediate particle sizes
Uniformly Graded	-	predominantly of one grain size

Terminology describing soils on the basis of grain size and plasticity is based on the Unified Soil Classification System (USCS) (ASTM D-2488). The classification excludes particles larger than 76 mm (3 inches). This system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%

The standard terminology to describe cohesionless soils includes the compactness (formerly "relative density"), as determined by laboratory test or by the Standard Penetration Test 'N' - value.

Relative Density	'N' Value	Compactness %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

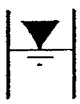
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests, or occasionally by standard penetration tests.

High - Weathering extends throughout rock mass. Rock is friable.

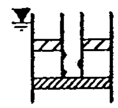
STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:

WATER LEVEL MEASUREMENT



Borehole or Standpipe



Piezometer

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
PS	Piston sample
BS	Bulk sample
WS	Wash sample

HQ, NQ, BQ, etc. Rock core samples obtained with the use of standard size diamond drilling bits.

N - VALUE

Numbers in this column are the results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and 'N' values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75).

OTHER TESTS

S	Sieve analysis
G_s	Specific gravity of soil particles
k	Permeability (cm/sec)
↓	Single packer permeability test; test interval from depth shown to bottom of borehole
I	Double packer permeability test; test interval as indicated
↓	Falling head permeability test using casing
↓	Falling head permeability test using well point or piezometer
H	Hydrometer analysis
ā	Unit weight
C	Consolidation
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct shear
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$; the index corrected to a reference diameter of 50 mm)

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BOREHOLE RECORD

05-1

CLIENT Public Works and Government Services Canada

BOREHOLE No. 05-1

LOCATION Kingston Mills - Lock #46

PROJECT No. ONO11783

DATES: BORING 01-26-05 WATER LEVEL

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES: TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
		Horizontal Borehole at Elevation 95.4 m						
0		Concrete			RC	1	100%	89%
		Mortar joint						
		Limestone Masonary Stone			RC	2	96%	100%
1								
		Limestone Masonary Stone			RC	3	100%	100%
		End of Borehole						
2								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

□ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ______

Date ______

WEL.G 02/05 11783

Jacques Whitford

BOREHOLE RECORD

05-3

CLIENT Public Works and Government Services Canada

LOCATION Kingston Mills - Lock #46

DATES: BORING 01-24-05 WATER LEVEL

BOREHOLE No. 05-3

PROJECT No. ONO11783

DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
		Horizontal Borehole at Elevation 96.3 m						
0		Concrete, crumbled upon removal from corebarrel			RC	1	50%	0%
		Concrete			RC	2	82%	65%
		Limestone cobble						
		Granite boulders with mortar in joints - Possible Infilled Stone Rubble			RC	3	100%	68%
1								
		Granite - Possible Bedrock or Masonary Stone			RC	4	100%	100%
		End of Borehole						
2								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

□ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ________

Date ________

Jacques Whitford

BOREHOLE RECORD

05-9

CLIENT: Public Works and Government Services Canada

LOCATION: Kingston Mills - Lock #46

DATES: BORING 01-28-05 WATER LEVEL

BOREHOLE No. 05-9

PROJECT No. ONO11783

DATUM Local

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

Horizontal Borehole at Elevation 99.16 m

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	TYPE	NUMBER	RECOVERY (mm)	N-VALUE OR RQD
0		Concrete - upper 300 mm badly cracked			RC	1	100%	0%
					RC	2	100%	100%
					RC	3	100%	100%
		Limestone and Granite with Concrete Infilling - Masonary Stone or is Infilled Stone Rubble						
1					RC	4	100%	28%
					RC	5	100%	50%
		End of Borehole						
2								

▽ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

□ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ________

Date ________

Jacques Whitford

BOREHOLE RECORD

05-11

CLIENT Public Works and Government Services Canada — BOREHOLE No. 05-11

LOCATION Kingston Mills - Lock #46 — PROJECT No. ONO11783

DATES: BORING 01-26-05 WATER LEVEL — DATUM Local

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION	STRATA PLOT	WATER LEVEL	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm)	SAMPLES N-VALUE OR RQD
		Horizontal Borehole at Elevation 95.45 m						
0		Limestone block with horizontal and vertical cracks - Masonary Stone						
		Granite with mortar in cracks - Possibly Infilled Stone Rubble			RC	1	100%	47%
		End of Borehole						
1								
2								

UNDRAINED SHEAR STRENGTH - kPa: 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC PENETRATION TEST, BLOWS/0.3m ★

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90

∇ Inferred Groundwater Level

▼ Groundwater Level Measured in Standpipe

□ Field Vane Test, kPa

□ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

App'd ________

Date ________

1178... WEL.G... ..02/05

Appendix 4

Core Photographs

Jacques Whitford

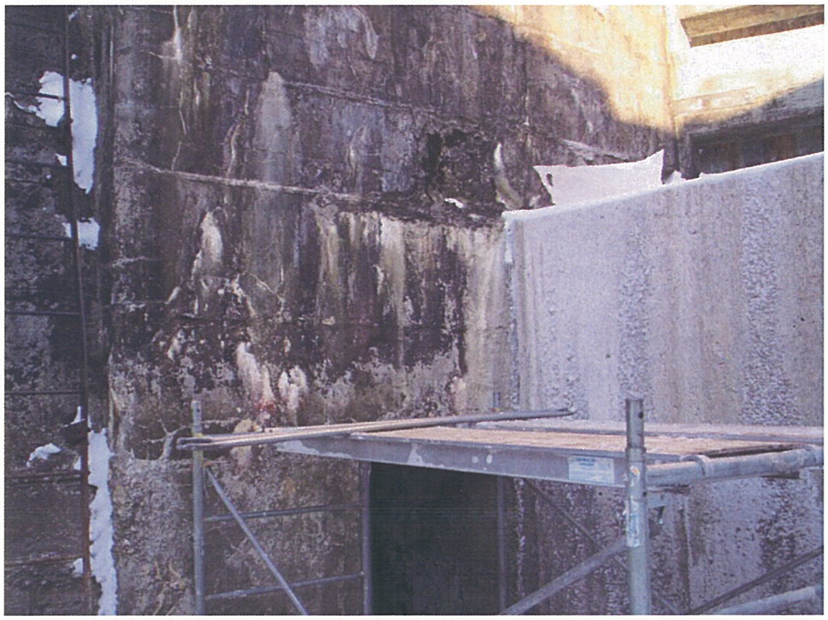

Photo 1 – BH 05-1 proposed location
second row of blocks above sluice on west wall

Photo 2 – BH 05-1, RC 1 – 0'0" to 2'3" – concrete

Jacques Whitford

Project No. ONO11783

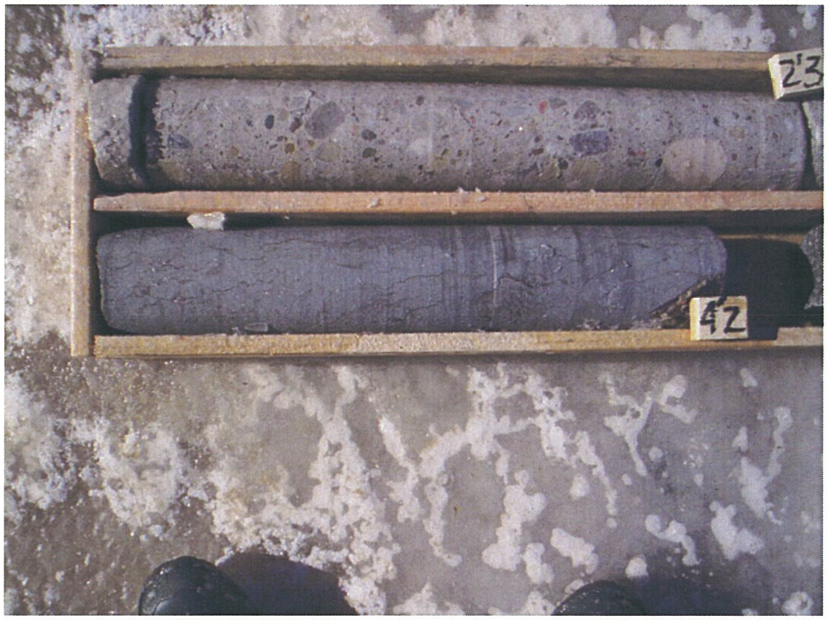

Photo 3 – BH 05-1, RC 2 – 2'3" to 4'2"
Limestone block with mortar joints at 2'1.5" to 2'3" and 4'1" to 4'2"
Specimen taken for compressive strength test (limestone)

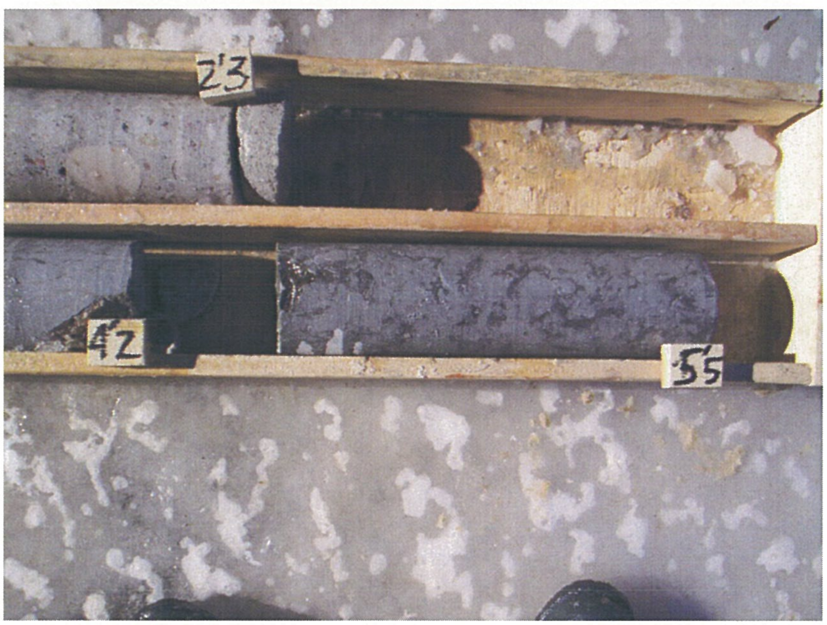

Photo 4 – BH 05-1, RC 3 – 4'2" to 5'5"
Limestone block

Photo 1 – BH 05-2 – Location of core
just south of upper gates, above sill

Photo 2 – BH 05-2, RC 1 – 0'0" to 2'6"
10" – 2'6" contains mortar and concrete

Photo 3 – BH 05-2, RC 2 – 2’6” to 5’9”
last 1’8” is a limestone block

Photo 4 – BH 05-2, Repair using drypack non-shrinkage grout
Compacted with heavy steel rammer

Photo 1 – Proposed location of BH 05-3

Photo 2 – Start of coring BH 05-3

Photo 3 – Coring BH 05-3
borehole drilled in a severely spalled area

Photo 4 – Close-up of BH 05-3
severe spalling at the concrete surface

Photo 5 – BH 05-3, Block being cored through
Note severe spalling at the corner

Photo 6 – BH 05-3, 0'0" to 5'9"
complete core

Photo 7 – BH 05-3, RC 1 – 0'0" to 1'4" (16")
crumbled concrete

Photo 8 – BH 05-3, RC 2 – 1'4" to 2'9"
concrete with a limestone cobble at last 3"
specimen taken for Petrographic analysis

Photo 9 – BH 05-3, RC 3 – 2'9" to 4'7"
granite boulder with some concrete

Photo 10 – BH 05-3, RC 4 – 4'7" to 5'9"
granite stone or bedrock

Photo 1 – Proposed location for BH 05-4

Photo 2 – Coring BH 05-4

Photo 3 – BH 05-4, RC 1 – 0’0” –2’3” (wall thickness)
0’0” – cracking in concrete
1’7” – 2’3” - specimen taken for compressive strength test

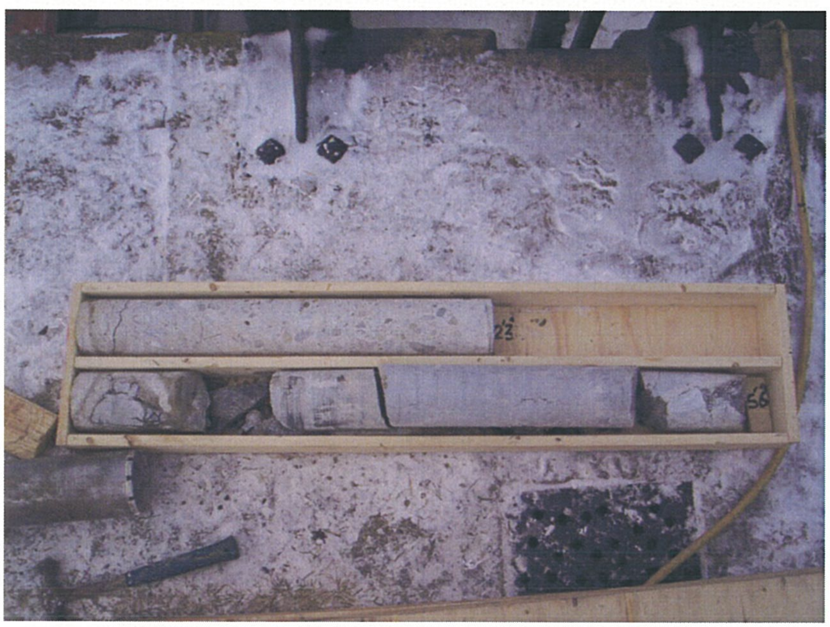

Photo 4 – BH 05-4, RC 2 – 2’3” to 5’6”
limestone boulders with concrete infilling fractures

Photo 1 – BH 05-5
east wall pilaster just north of lower gate

Photo 2 – BH 05-5, RC1 – 0'0" to 1'0"
concrete, severe cracking at the first 4"

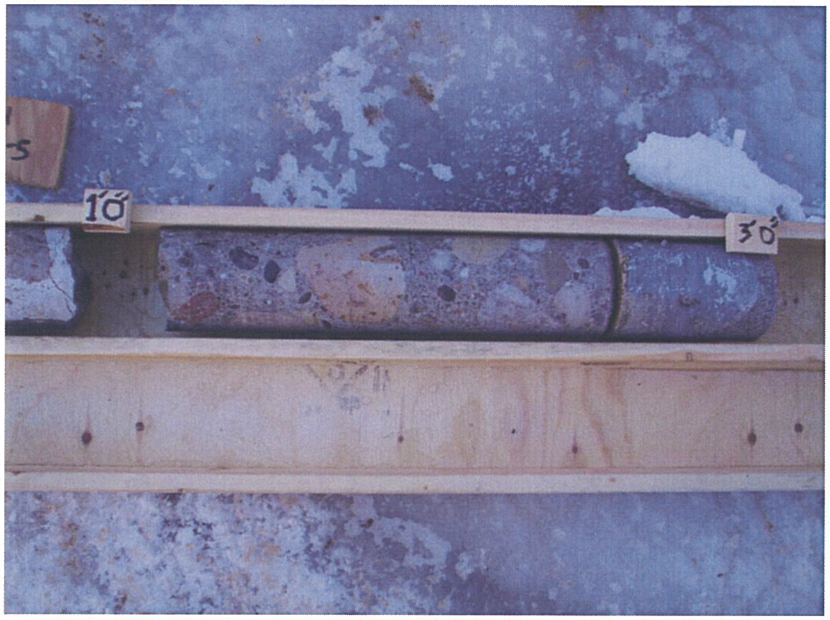

Photo 3 – BH 05-5, RC 2 – 1'0" to 3'0"
1'0" – 2'5" - concrete
2'5" to 3'0" - limestone boulder with mortar in fissures

Photo 4 – BH 05-5, north side of core hole
Cracking at 2" parallel to surface

Photo 5 – BH 05-5, upper side of core hole
Cracking at 2” depth parallel to surface

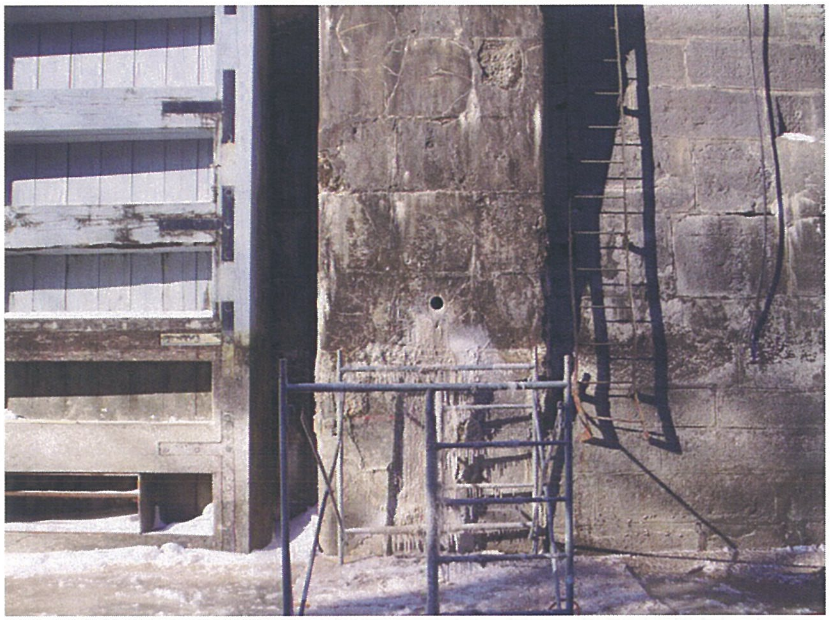

Photo 1 – BH 05-6
located just north of lower gates on west wall

Photo 2 – BH 05-6, RC 1, 0'0" to 1'11"
concrete in relatively good shape, some cracking
1'0" -1'11" – specimen for Petrographic analysis

Photo 3 – BH 05-6, RC 2 – 1'11" to 3'2"
half of core is limestone block and other half mortar joint

Photo 4 – BH 05-6, north face of core hole
Surface cracking

Photo 1 – BH 05-7
start of coring through sill at lower gates

Photo 2 – BH 05-7
coring through sill at lower gates

Photo 3 – BH 05-11, north side of core hole
Horizontal crack in limestone block

Photo 4 – BH 05-11, south side of core hole
Horizontal crack in limestone block

Photo 1 – BH 05-8, start of coring

Photo 2 – BH 05-8, RC 1 – 0'0" to 2'7"
severe cracking for first 5" and at 8"

Photo 3 – BH05-08, RC 2 – 2'7" to 2'11"
granite boulder or stone

Photo 4 – BH 05-8
Surface condition at core location

Photo 1 – BH 05-9
start of coring

Photo 2 – BH 05-9, RC 1 – 0'0" to 1'0"
0" to 8" - cracking in concrete
9" - rebar
8" to 12" - broken into pieces

Photo 3 – BH 05-9, RC 2 – 1'0" to 1'10"
Concrete solid, used for testing of compressive strength

Photo 4 – BH 05-9, RC 3 – 1'10" to 2'7"
Cracking at 2'3' in concrete

Photo 5 – RC 4, 2'7" to 4'1"
limestone block with mortar joints

Photo 6 – BH 05-9, RC 5, 4'1" to 5'1"
4'1" to 4'7" - granite
4'7" to 5'1" - broken limestone

Photo 1 – BH 05-10, RC 1 – 0'0" to 2'4"
0 to 20" is mortar with limestone and granite rock
20" to 28" is granite rock with mortar

Photo 2 – BH 05-10, RC 2 – 2'4" to 3'0"
Broken limestone pieces with mortar

Photo 1 – BH 05-11, Core location
in damaged limestone block on west wall

Photo 2 – BH 05-11, RC 1 – 0'0" – 2'10"
0'0" to 1'4" - limestone block with vertical and horizontal cracking
1'4" to 2'10" - granite boulders with grout in fissures

Appendix 5

Petrographic Analysis Report

Jacques Whitford

Materials & Petrographic Research G-B Inc.

Geotechnical Investigation, Project No. 304498
Petrographic Report
on
Concrete Cores from the
Kingston Mills Lock No. 46, Kingston, Ontario,

CLIENT REPORT

Prepared for

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Report No. GB-05-JW-2
Report date: March 30th, 2005.

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Geotechnical Investigation, Project No. 304498

Petrographic Report
on
Concrete Cores from the
Kingston Mills Lock No. 46, Kingston, Ontario,

Executive Summary

Two concrete cores taken from the Kingston Mills Lock No. 46, Kingston Ontario, were received for petrographic evaluation in February 2005. The cores are labeled BH05-03 and BH05-6. The purpose of the petrographic investigation was to investigate the condition of the concrete in the cores, and if deterioration had occurred, to determine its probable cause. The condition of the concrete in the cores was evaluated using the Damage Rating Index (DRI) method on the polished surfaces of the cores. The mineralogical composition of the fine and coarse aggregates was determined by examination of petrographic thin sections. The nature of the cement paste was also determined from the thin sections. The mean Damage Rating Index of 124 that was obtained indicates that there has been significant damage to the concrete due to alkali-silica reaction. There was no evidence of significant frost damage to the concrete. The calculated estimate of the amount of expansion that has occurred, to date, in the structure is in the range of 0.1 to 0.2%. It is probable that expansion in the structure will continue at the current rate for the foreseeable future provided that the alkali content of the concrete is high enough to sustain the reaction. The coarse aggregates consist mainly of a mixture of Potsdam sandstone, dolostone and biotite hornblende granite. The fine aggregate, natural sand, contains mono-mineralic particles of quartz and feldspar and assorted rock fragments. The same types of rocks that were observed in the coarse aggregate occur in the fine aggregate. The cement paste is dense and mostly un-carbonated. It must be emphasized that the conclusions of this report are based on the examination of only two cores. The applicability of the results to the concrete in the structure depends on the representativeness of the concrete in the cores.

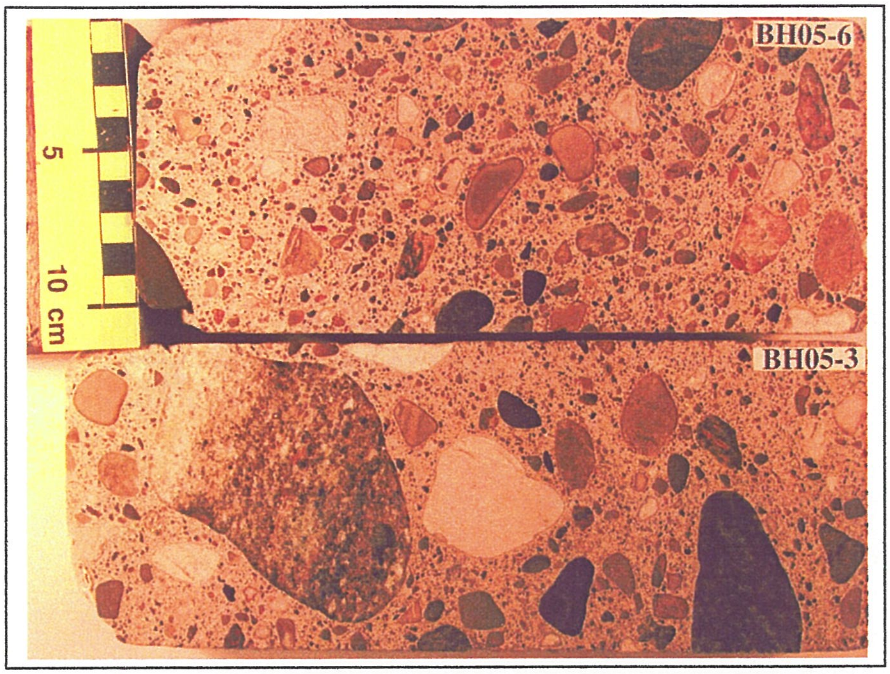


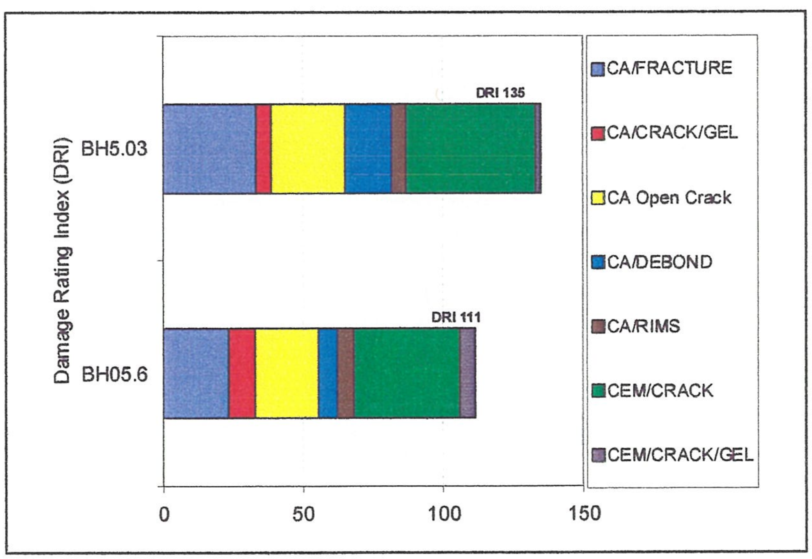
Fig.1: Surfaces of polished cores showing the size, shape and color of the aggregate particles. Scale divisions in cm.

RESULTS

Damage Rating Indices (DRI's)

The DRI's of the two cores are shown in Figure 2.

Fig. 2:
DRI's of cores.
CA – coarse aggregate.
Cem- cement paste



The DRI's of cores BH 05 03 and BH 05 6 are 136 and 111 respectively, with a mean value of 124. DRI's greater than ~40 are considered to be indicative of significant alkali-silica reaction

(ASR) provided there is evidence for ASR in the cores. Alkali-silica gel was observed in some cracks, in the coarse aggregate particles, Figure 3a. Cracks, without gel, were observed in the cement paste, Figure 3b. Reaction rims were observed around some particles of coarse aggregate, Figure 3c. These observations confirm the occurrence of ASR in the concrete cores.

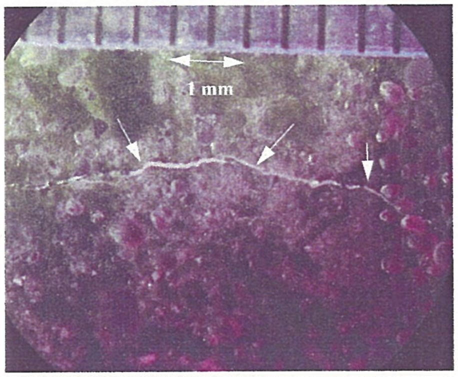


Fig. 3a: Gel filling a crack in the coarse aggregate and extending through the cement paste. Photograph taken in UV light in which the gel treated with uranyl acetate fluoresces

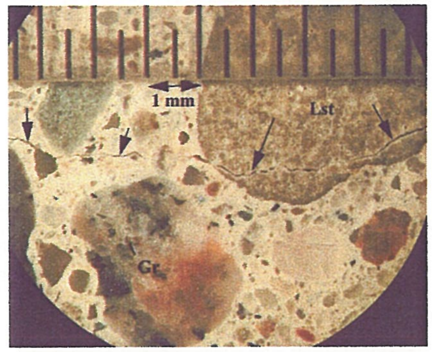


Fig. 3b: Crack indicated by arrows in the cement paste.
Gr – granite, Lst - limestone

Reaction rims were observed around a number of sandstone particles, Figure 3c. Open cracks in coarse aggregate particles, frequently observed in concrete affected by ASR were observed in some sandstone and limestone particles, figures 3 c and d.

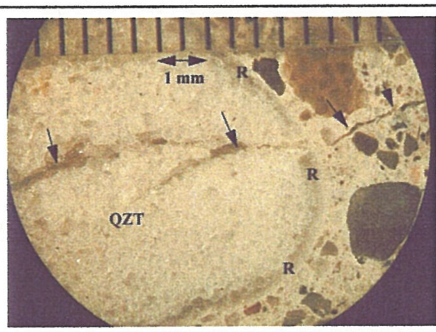


Figure 3c: Sandstone particle (QZT) with an open crack, indicated by arrows. There is a reaction rim ® around the sandstone particle.

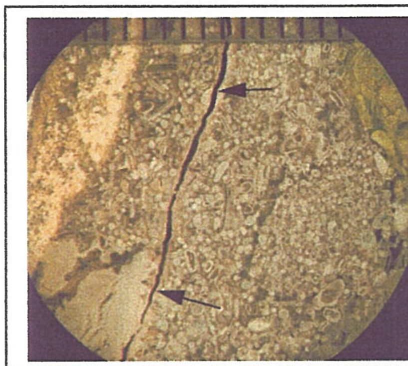


Fig 3d: Oolitic limestone particle with an open crack indicated by arrows.

Coarse Aggregates

The coarse aggregate consists dominantly of sub-rounded gravel particles, Figure 1. Due to the relatively small combined area (240 cm^2) of the four thin sections and the large size of some coarse aggregate particles, (10 cm in diameter), it was not possible to determine precisely the mineralogical composition of the aggregate. However, an estimate was obtained by counting the numbers of grains of different rock types in the thin sections. The results are listed in Table 2.

Table 2: Composition of the coarse aggregate

Rock Type	% of Agg
Calcareous sandstone	10
Siliceous sandstone	32
Granite Gneiss	26
Dolostone	23
Sandy micritic limestone	6
Sparitic limestone	3

The siliceous Potsdam sandstone is potentially alkali-silica reactive (ASR). This rock is identical to that which caused large expansion and cracking of the concrete in the Beauharhois dam and powerhouse Baillivy et al. The difference between Beauharnois and the Kingston Mills Lock is that in Beauharnois the coarse aggregate is composed entirely of Potsdam sandstone. The reactivity of the sandstone is due to the presence of quartz overgrowths around the original grains, Figure 4a. The crystal lattices of the overgrowths are strained. This increases their solubility and hence reactivity.
One grain of potentially reactive micro-crystalline quartz was also observed in the sandstone, Figure 4b.

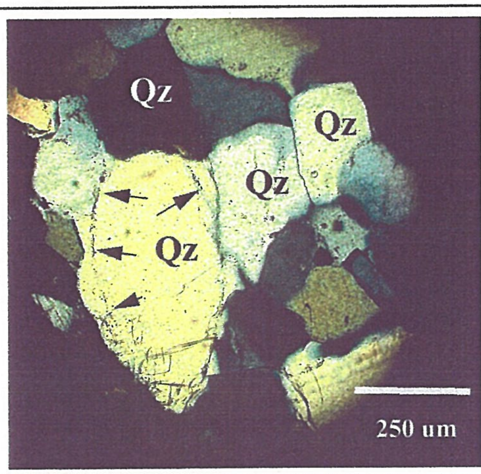


Fig: 4a. Thin section of Potsdam sandstone showing quartz overgrowth around the original grain. The boundary of the original grain is indicated by arrows. QZ- quartz.

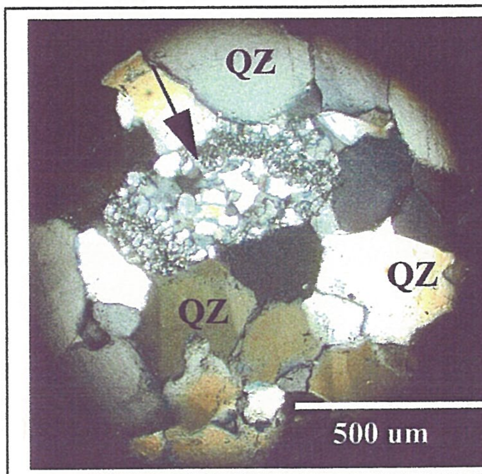


Fig. 4b:Thin section of a particle of micro-crystalline quartz in the sandstone, indicated by the arrow. Quartz - QZ.

A small amount of granite and gneiss also occurs in the coarse aggregate, Figure 4c. Limestones comprise about 32% of the coarse aggregates. Three types of limestone were observed: Coarse grained sparite, Figure 4d, a sandy micrite, possibly potentially reactive, and dolostone, Figure 4e.

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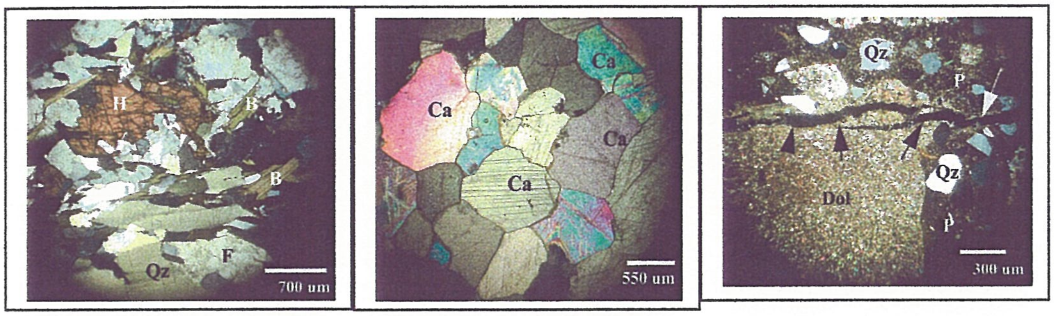


c d e

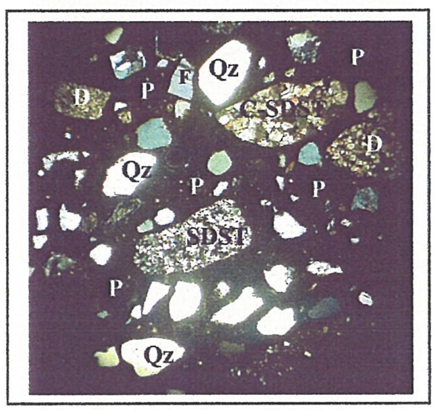
Fig. 4 continued:

c) Biotite hornblende granite. Hornblende – H, biotite – B, quartz QZ.
d) Sparitic limestone calcite grains – Ca.
e) Dolostone grain –Dol with an open crack, indicated by arrows, extending into the cement paste –P. Quartz grains –QZ.

Fine Aggregate

The fine aggregate is a naturally occurring sand with a grain size of ~35 to 700 μm. The finer fraction is composed mostly of mono-mineralic grains of quartz and feldspar with a minor amount of biotite mica. The coarser fraction is composed mainly of fragments of rock: dolostone, gneiss and Potsdam sandstone. A photograph of a thin section of the fine aggregate is shown in Figure 5.

Fig. 5: Photograph of thin section of fine aggregate, viewed between crossed polarizers. Quartz – QZ, Dolomite – D, siliceous sandstone – SDST, calcareous sandstone – CSDST, cement paste – P.



Cement Paste

The cement paste is mostly dense and uncarbonated, appearing black in the photograph, Figure 5. The concrete is not air-entrained but contains some entrapped air.

DISCUSSION

The mean DRI of 124 is an indicator that the concrete represented by the three cores has been significantly affected by alkali-silica reaction. Cracking and expansion of the structure would be expected. However, it is not known to what extent the concrete, in the two cores that were evaluated, is representative of the concrete in the structure as a whole. The difference between the DRI's of the two cores, 111 and 135 is not significant. Judging from the DRI chart Figure 2, the concrete in the two cores does not appear to have been significantly affected by frost action.

However, the two cores were taken from a depth of over 300 mm in the concrete and thus do not contain near-surface concrete that may be affected by cycles of freezing and thawing. Typically, cycles of freezing and thawing lead to gaps around particles of coarse aggregate due to expansion of the cement paste. The gaps are referred to as "debonding" in the DRI chart, Figure 2. Some debonding is almost always associated the damage to the concrete due to ASR. The amount observed in these cores is typical of that associated with ASR, but there may be some contribution due to frost action.

Estimated Expansion of Concrete in the Structure

An equation was developed, by measurement of the expansions and DRI's of concrete prisms stored at 38°C and ~100% humidity, relating the percentages expansion to the DRI's, equation i.

$$\text{Expansion \%} = 1.45E^{-3}(\text{DRI}) + 0.047 \quad \text{Eqn. i.}$$

Using the mean DRI of the three cores of 123, a value of 0.23% is obtained for the total expansion in the structure to date. However, it must be realized that this result is based on measurements of expansions at 38°C., while the mean temperature of concrete in the structure might be in the range of 13°C. The rate of expansion would be expected to be significantly lower at 13° than at 38°. The expansion of the concrete prisms in the laboratory is unconfined expansion, while in the lock there would be some degree of confinement. The effect of temperature on the rate of expansion is not known, and furthermore it is probably dependent on the type of aggregate in the concrete. However, if one assumes that the expansion would be halved, due to the effect of low temperature and confinement, the total expansion, to date, would be about 0.1% a not insignificant amount, e.g., 2.5 cm in a 25-meter span. The expansion in a 25-meter span would be 5.8 cm if the value of 0.23% expansion were used. These results indicate that the estimated amount of expansion in a 25-meter span could be in the range of 2 to 6 cm.

Prediction of Future Expansion in the Structure

Prediction of the future expansion of concrete in the structure is difficult. Assuming that 0.1% expansion has occurred in the past 40 years and that the rate of expansion has been constant, a rate of 0.0025% p.a., is obtained. Again, assuming that this rate will be maintained for the foreseeable future, which is reasonable based on experience with many large dams, the possible expansion for the next 25 years would be ~0.06%. However, the continuing expansion of the concrete in the structure depends on the alkali content of the concrete being high enough to sustain the reaction, typically over 2 kg/m^3. If considered necessary, the alkali content of the cores could be determined.

CONCLUSIONS

The following conclusions are based on the examination of only two cores. The applicability of the results to the concrete in the structure, as a whole, depends on the representativeness of the concrete in the cores.

- Evidence of alkali-silica reaction was observed in both cores.
- The mean damage rating index (DRI) of 124 indicates that there is significant damage to the concrete due to alkali-silica reaction.

- The calculated estimate of the amount of expansion, which has occurred in the concrete to date, is in the range of 0.1 to 0.2%
- It is probable that expansion of the concrete will continue at the current rate for the foreseeable future, provided the alkali content of the concrete is high enough.
- The estimated expansion over the next 25 years is ~0.06%.
- The coarse aggregate is a mixture of biotite and hornblende granite, Potsdam sandstone, sparitic limestone and dolostone.
- The Potsdam sandstone is the probable reactive component in the coarse aggregate.
- The fine aggregate, natural sand composed of mono-mineralic quartz and feldspar grains and assorted rock fragments that include Potsdam sandstone, granite, gneiss and dolostone.
- The concrete is not air-entrained but contains some entrapped air.
- No evidence of significant damage to the concrete due to frost action was found. However, frost damage would not be expected in concrete taken from over 300 mm below the surface.
- The cement paste is relatively dense, and for the most part, uncarbonated.

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APPENDIX

D-6 *2015 GOLDER ASSOCIATES INVESTIGATION*

November 2015

REPORT ON

Geotechnical Investigation Proposed Structural Rehabilitation Kingston Mills Swing Bridge Kingston, Ontario

Submitted to:
MMM Group Ltd.
920 Princess Street, Suite 101
Kingston, Ontario
K7L 1H1

REPORT

Report Number: 1528810

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APPENDIX B
Record of Borehole Sheets and Laboratory Testing Results – Previous Investigations

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Photographs of Bedrock Core

1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the proposed rehabilitation of the Kingston Mills Swing Bridge located along Kingston Mills Road in Kingston, Ontario (see Key Plan inset on Figure 1).

The purpose of the investigation was to assess the general subsurface conditions for the proposed bridge rehabilitation by means of three horizontal boreholes, one vertical borehole, and four test pits, and carrying out in situ testing and laboratory testing on selected samples. Based on an interpretation of the factual information obtained, a general description of the subsurface conditions in the area of the structure is presented. These interpreted subsurface conditions and available project details were used to prepare engineering guidelines on the geotechnical design aspects of the project.

The reader is referred to the 'Important Information and Limitations of This Report' which follows the text of the report but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

Parks Canada has requested that a structural assessment be carried out for the Kingston Mills Swing Bridge which crosses the first (upper) of four locks which connect Colonel By Lake to the Cataraqui River in Kingston, Ontario.

The existing bridge was originally constructed in 1956 and consists of a single span cantilevered structure with a total length of about 30 metres. Existing information from previous geotechnical investigations by others indicate that the west abutment of the bridge is founded on bedrock, and the east abutment bears in part on the east lock wall and possibly on fill (see Plate 1). There is conflicting information with regards to the subgrade beneath the foundation of the pivot pier. The subgrade has been described as either medium to stiff clay with trace gravel over bedrock, or as fill consisting of clayey silt and gravel.

It is understood that the objective of the rehabilitation assessment component of the project is to bring the bridge into compliance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC) to carry full loading.

Previous investigations carried out within the area of the Kingston Mills Swing Bridge consist of the following:

- Report by Quontacon Associates to Government of Canada, Parks Canada titled "Kingston Mills Swing Bridge, Rideau Canal, Geotechnical Investigations" dated January 2000 (Report Number 51173-30001725)
- Report by J.D. Lee Engineering Limited to Indian and Northern Affairs, Parks Canada, Ontario Region titled "The Foundation Condition of the Swing Bridge at Kingston Mills, Ontario" dated September 1976 (Report Number 1176)
- Letter Report by J.D. Lee Engineering Limited tilted "Swing Bridge at Kingston Mills" dated March 17, 1977 (Job Number 1608).

Based on published geological maps and existing records of nearby boreholes, the subsurface conditions at this site are expected to consist of limited thicknesses of fill overlying seyenite or granite bedrock. The bedrock surface is indicated to be at about 0 to 3 metres depth.

3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the bridge assessment was carried out between May 8 and May 14, 2015 and on May 26, 2015 during which time three horizontal boreholes (numbered 15-202 to 15-204, inclusive), one vertical borehole (numbered 15-101), and four test pits (numbered 15-1, 15-1A, 15-2, and 15-2A) were advanced at the locations shown on the Site Plan, Figure 1. A fourth horizontal borehole (numbered 15-201) was planned but was not advanced due to time restrictions for in-lock works.

Boreholes 15-202 to 15-204 were advanced horizontally using portable drilling equipment owned and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. Boreholes 15-202 and 15-203 were advanced at a height of about 2.3 metres above the floor of the canal lock and borehole 15-204 was advanced at a height of about 4.6 metres above floor of the canal lock in line with the outside bridge girders. The boreholes were advanced through the canal wall a total distance ranging from 2.6 to 4.0 metres horizontally from the edge of the canal wall.

Borehole 15-101 was advanced vertically using a truck-mounted hollow-stem auger drill rig supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario. The borehole was advanced to a depth of about 2.7 m below the existing ground surface in the overburden. The borehole was then advanced into the bedrock an additional 2.5 metres using NQ-size coring equipment.

Standard Penetration Tests (SPTs) were carried out at regular intervals of depth within the vertical borehole and samples of the soils encountered were recovered using drive open sampling equipment.

The test pits were advanced using a mini-excavator supplied and operated by Robert J. Lappan Excavating of Kingston, Ontario. The test pits were advanced to the bedrock surface, which was encountered at depths of about 0.3 to 3.0 metres below the existing ground surface.

The soil exposed on the sides of the test pits were classified by visual and tactile examination. The groundwater seepage conditions were observed in the open test pits and the test pits were loosely backfilled upon completion of excavating and sampling.

The fieldwork was supervised by members of our technical staff who located the boreholes and test pits, supervised the drilling, excavating, and in situ testing operations, logged the boreholes, test pits, and samples, and took custody of the soil samples and bedrock core retrieved.

The soil and bedrock samples were identified in the field, placed in appropriate containers, labelled, and transported to our Ottawa geotechnical laboratory where the samples underwent further detailed visual examination by the project engineer and laboratory testing. The laboratory testing included: natural water content determinations, grain size distribution and Atterberg limits testing.

4.0 SUBSURFACE CONDITIONS

4.1 General

Information on the subsurface conditions is provided as follows:

- Grain size distribution testing results, provided on Figure 2.
- Record of Test Pits from the current investigation, provided in Table 1 and on Figures 3 and 4.
- Record of Borehole and Drillhole Sheets for the current investigation, provided in Appendix A.
- Record of Borehole Sheets from previous investigations, provided in Appendix B.
- Photographs of the bedrock core, provided in Appendix C.

In general, the subsurface conditions at the west pier location consist of up to 3 metres of fill over bedrock. The subsurface conditions at the east pier location consist of about 2.5 metres of fill over bedrock.

The following sections present a more detailed overview of the subsurface conditions encountered in boreholes and test pits put down for current investigation and the relevant boreholes which were put down during the previous investigations.

4.2 Pavement Structure, Topsoil, and Fill

Topsoil was encountered at the ground surface at test pits 15-1, 15-1A, 15-2, and 15-2A and ranged in thickness from about 30 to 200 millimetres.

Borehole 15-101 was advanced through the pavement structure of Kingston Mills Road. The pavement structure consists of 100 millimetres of asphaltic concrete over 150 millimetres of gravelly sand base.

A layer of fill was encountered below the topsoil and pavement structure, where encountered in borehole 15-101 and test pits 15-1, 15-1A, 15-2, and 15-2A. The fill generally consists of silty clay with varying amounts of sand and gravel to silty sand with varying amounts of gravel.

Standard penetration tests carried out within the fill at borehole 15-101 gave SPT 'N' values ranging from 8 to 29 blows per 0.3 metres of penetration, indicating a loose to compact state of packing.

Boreholes 15-201, 15-202, and 15-203 were advanced horizontally through the east limestone block wall of the Rideau Canal Lock. The limestone masonry blocks at the cored locations range in depth from about 310 to 660 millimetres.

A layer of grouted rubble fill was encountered behind the limestone block retaining wall in boreholes 15-202, 15-203, and 15-204. The rubble fill has two significantly different grout types, one, possibly the original grout consists of light grey cement grout with coarse sand aggregate; the second consists of a medium grey fine grained grout, probably the more recent of the two.

4.3 Bedrock

Bedrock was encountered beneath the fill in borehole 15-101 at a depth of about 2.7 metres. The test pits for the current investigation encountered refusal to excavating on the bedrock surface at depths ranging from about 0.3 to 3.0 metres below the ground surface. Sketches of the subsurface conditions encountered in TP 15-1A and 15-2A are provided on Figures 3 and 4 respectively.

Bedrock was also encountered behind the grouted rubble fill in the horizontal boreholes 15-202, 15-203, and 15-204 at depths ranging from 1.6 to 2.8 metres.

Previous boreholes advanced on the site encountered refusal to auger advancement and the bedrock surface at depths ranging from 0.3 to 4.6 metres below the existing ground surface.

The following table summarizes the depth to refusal/bedrock surface and bedrock surface elevations as well as the ground surface elevation encountered in the boreholes and test pits from the current and previous investigation.

Testhole Number	Location	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
BH 15-101	East Side	±91.9	2.7	±89.2
TP 15-1	West Side	±90.6	1.9	±88.7
TP 15-1A	West Side	±90.6	3.0	±87.6
TP 15-2	West Side	±90.6	0.3	±90.3
TP 15-2A	West Side	±90.6	0.9	±90.6
BH A	East Side	90.8	0.3	90.5
BH B	East Side	90.8	0.3	90.5
BH C	East Side	90.7	0.8	89.9
BH D	East Side	90.7	0.9	89.8
BH E	East Side	90.7	2.7	88.0
BH F	East Side	90.7	2.4	88.3
BH G	East Side	91.9	2.4	89.5
BH H	East Side	91.8	2.5	89.3
BH I	East Side	91.8	1.3	90.5
BH J	East Side	90.7	2.7	88.0
BH K	West Side	90.5	4.6	85.9
BH L	West Side	90.4	2.1	88.3
BH C1	West Side	92.2	2.4	89.8

The bedrock consists predominately of fresh, massive, pink and dark grey, medium to coarse grained, non-porous, slightly foliated, granite. The Rock Quality Designation (RQD) values measured on the recovered bedrock core samples from the current investigation were quite variable and ranged widely from 0 to 100 percent; however, were more generally between 75 and 100 percent, indicating a good to excellent rock quality.

Photographs of the bedrock core retrieved for the current investigation are provided in Appendix C.

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation engineering guidelines and recommendations pertaining to the proposed structural rehabilitation of the Kingston Mills Swing Bridge in Kingston, Ontario.

The reader is referred to the 'Important Information and Limitations of This Report' which follows the text but forms an integral part of this document.

5.2 Summary of Existing Bridge Foundations

5.2.1 West Pivot Pier

Based on the information provided by MMM Group, the pivot (west) pier is an octagonal shaped foundation measuring about 9 metres in diameter and about 1.5 metres in height. Pictures of the original construction of the pivot pier are shown below.

Photograph 1 – Showing the construction of original pier founded on granular fill and the Canal Lock wall on the east side

Photograph 2 – Showing the construction of original pier founded on the bedrock on the west side with dowels into the bedrock.

From the photographs provided, the pivot pier appears to be founded on the bedrock surface on the west side and the canal wall on the east side, with fill between those two relatively rigid supports. The fill appears to be granular fill up to the underside of the footing based on the photographs, but the previous investigations indicate that the fill below the pier consists of predominantly clayey silt with cobbles and boulders. The fill beneath the pier would have limited capacity to support additional loading from the pier.

The test pit excavations in the current investigation were extended to the surface of the bedrock and it was found that a pivot pier foundation extends to the surface of the bedrock. Based on the test pits, it appears that the pivot pier may have been underpinned at some point in time after the original construction. The pivot pier may therefore not be supported on fill as indicated in the historical photographs and previous boreholes, or it has been underpinned since the original construction.

5.2.2 East Abutment Pier

The East abutment is about 10 metres by 2 metres in plan dimension. The east abutment is founded on the canal lock wall and the wall backfill which consists of grouted granitic rubble fill between the canal wall and the original bedrock excavation. Two types of grout are present. One is an older grout and consists of light grey cement with coarse sand aggregate. The other is a newer grout consisting of a medium grey fine textured grout. Both grouts are evident between the rubble fragments and the newer grout fills voids in the older grout. Core recovery within the grouted rubble was about 60 to 100 percent. During coring, however, it was noted that flush water was being returned throughout the coring and, as such, the poorer recovery might be the result of grinding and washing of old grout during the coring process.

It was recommended in previous reports to pressure grout the backfill to consolidate the fill under the abutment. The results from the horizontal boreholes indicate that post construction grouting was carried out although there is no record of the grouting work being completed.

5.3 West Pivot Pier

For the rehabilitation of the west pivot pier, it is suggested that the entire foundation be excavated around the perimeter of the foundation to assess the extent and quality of underpinning. Any repairs or additional underpinning for support should be made at that time.

5.3.1 Excavations

Excavations for the west pivot pier rehabilitation would extend up to about 3 metres depth through the surficial fill materials to the surface of the bedrock.

No unusual problems are anticipated with excavating the fill material using conventional hydraulic excavating equipment. However, some boulder excavation/removal should be expected in the lower portions of the fill. Any exposed boulders larger than 0.3 metres in diameter should be removed from the excavation slopes, for worker safety purposes.

The fill consists generally of Type 3 soils in accordance with the Occupational Health and Safety Act of Ontario and therefore temporary excavation side slopes should be cut no steeper than 1H:1V (horizontal to vertical). Alternatively, the excavations could be carried out using steeper side slopes with all manual labour carried out within a fully braced steel trench box for worker safety. If the excavation will remain open for an extended period of time, or be exposed to freeze and thaw periods, flatter slope angles may be required. Further guidance on that issue can be provided, as required.

5.4 East Abutment

The east abutment is supported partially on the canal wall and partially on the cemented rubble fill behind the wall. The horizontal holes indicate that the cemented rubble fill was placed directly against the granite bedrock. The drilling operations indicate that the grouted rubble is intact (all drilling flush water was returned). The lost core could be the result of grinding during coring and washing of weaker grout zones.

The following should be considered for the rehabilitation of the east abutment:

- Although the grouted rubble behind the wall seems to be intact, there should be an allowance item in the contract for re-grouting the grouted rubble behind the wall. The need for grouting could be determined by pressure packing the grout hole with water. Cave of the grout hole could occur even if rotary drilling is used to advance the grout hole but more so if percussion methods are used to advance the hole. The hole may need to be flushed or redrilled to clean it out: and,
- Installing rock anchors to tie the east abutment wall into the bedrock.

5.4.1 Grouting

The backfill materials behind the east abutment should be consolidation grouted using micro-fine cementitious grout placed under pressure. Previously grouted material can be difficult to grout, and a closely spaced pattern of grout holes, including primary, secondary and possibly tertiary grout holes, may be required to achieve the desired degree of consolidation.

The micro-fine grout should be prepared using a high shear colloidal mixture and placed using pressures sufficiently low to avoid jacking of the abutment. Monitoring of the abutment should be undertaken on a real time basis during grouting to confirm that abutment is stable in the horizontal and vertical directions.

The grouting should be carried out by a specialist contractor with at least ten years of recent experience grouting similar structures and in similar conditions. Lugeon testing should be used to confirm the quality of the grouting.

5.4.2 Rock Anchors

If required, the installation of the rock anchors should be carried out after grouting. Since the existing grouted rubble if unconsolidated in areas, may be difficult to drill for installation of the anchors. Casing of the anchor holes may be required if the grouting is not completed prior to installation of the anchors.

The rock anchors should consist of grouted anchors.

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

i) Failure of the steel tendon or top anchorage

ii) Failure of the grout/tendon bond

iii) Failure of the rock/grout bond

iv) Failure within the rock mass, or rock cone pull-out

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion, particularly in the submerged environment at this site. For permanent rock anchors, Class I double corrosion protection should be provided as per OPSS 942.

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as 1,000 kilopascals for ULS design purposes. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance should be calculated based on the buoyant weight of the potential mass of rock which could be mobilised by the anchor. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \phi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

Where:

Q_r	=	factored uplift resistance of the anchor, kilonewtons;
ϕ	=	resistance factor, 0.4;
γ'	=	effective unit weight of rock, use 27 kilonewtons per cubic metre above groundwater level, 17 kilonewtons per cubic metre below the groundwater level;
D	=	anchor length in metres; and,
θ	=	½ of the apex angle of the rock failure cone, use 30 degrees.

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. In the case of group effects for a series of rock anchors in a rectangle with width "a" and length "b" installed to a depth "D", the equation for the volume of the truncated trapezoid failure zone would be as follows:

$$V = \frac{4}{3} D^3 sin^2\varphi + aD^2 \sin\varphi + bD^2 \sin\varphi + abD$$

Where: V = Volume of the truncated trapezoid failure zone in cubic metres;

D = Depth of anchor group in metres;

a = Width of anchor group in metres;

b = Length of the anchor group in metres; and,

φ = ½ of the apex angle of the rock failure cone, use 30 degrees.

The ULS factored geotechnical resistance for the truncated trapezoid failure formed by the group of anchors can then be calculated based on the following equation:

$$Q_r = \varphi\gamma' V$$

Where: Q_r = Factored uplift resistance of the anchor, kilonewtons;

φ = Resistance factor, use 0.3;

γ' = Effective unit weight of rock, use 17 kilonewtons per cubic metre; and,

V = Volume of truncated trapezoid in cubic metres.

The method described above does not explicitly consider the tensile strength of the rock that must be overcome prior to mobilization of the weight of the rock mass. If required, the tensile strength of the rock mass can be assessed based on the unconfined compressive strength, recovery, and quality of bedrock core obtained.

Where the bedrock is at depth, the weight of the overlying overburden can also be considered in the calculation of the resistance to potential failure of the rock mass. Further guidance on this issue can be provided, if required.

It is suggested that proof load tests be carried out on anchors to confirm their proof load capacity. The proof load tests should be carried out to 1.3 times the anchor service loads, and at least 10 percent of the anchors should be tested in this manner. The testing procedure for rock anchors should be as per OPSS 942.

The rock anchor design should be reviewed by a geotechnical engineer and it is suggested that the installation and testing of the anchors be supervised by the geotechnical engineer. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grout area with a minimum of voids. Probing of the holes should be carried out by the geotechnical engineer to ensure that the anchors are being installed in rock of adequate quality. It is also suggested that the anchor holes be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to ensure an adequate bond between the grout and the rock.

6.0 SEISMIC SITE CLASS

According to Table A.3.1.1 of the 2006 CHBDC, the zonal acceleration ratio, A, applicable to this site is 0.1. The corresponding acceleration related seismic zone, Z_a, is 2.

Based on the subsurface conditions that include bedrock near ground surface, the site is considered to be Soil Profile Type I, which corresponds to a Site coefficient, S, of 1.0 as defined in Section 4.4.6 of the 2006 CHBDC.

7.0 ADDITIONAL CONSIDERATIONS

The samples obtained for this investigation will be retained in storage for a period of 3 months following issuance of this report. They will then be disposed of, unless direction for extended storage is provided.

No chemical analysis of the soil quality (or groundwater quality) in relation to the disposal options was carried out as part of this geotechnical investigation.

Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

GOLDER ASSOCIATES LTD.

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n:\active\2015\3 proj\1528810 mmm group limited kingston swing bridge\05_reporting\1528810 rpt-001 final november 2015.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, MMM Group Ltd. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

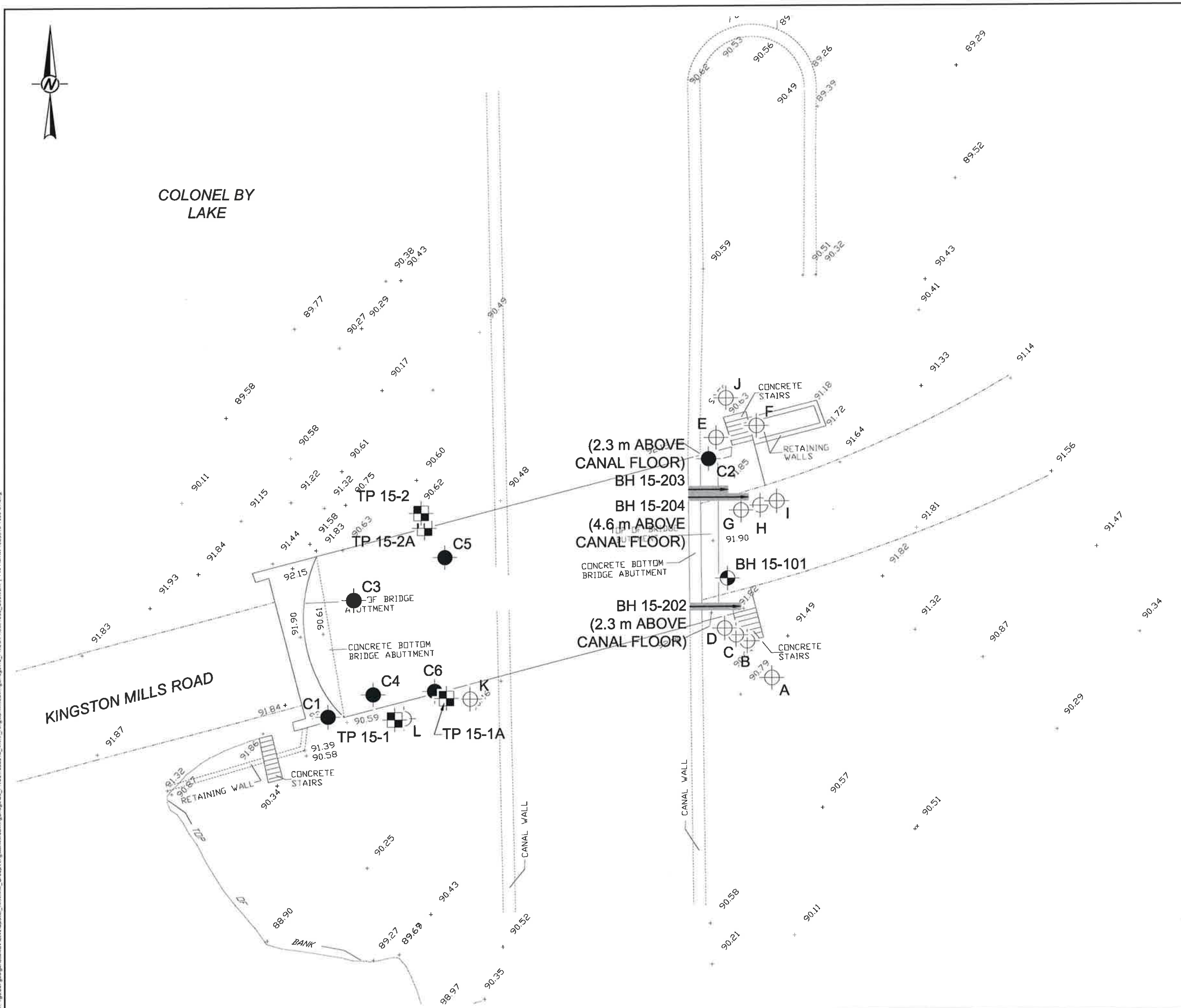
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.



KEY MAP

SCALE 1:40,000

LEGEND

- APPROXIMATE BOREHOLE LOCATION, CURRENT INVESTIGATION
- APPROXIMATE TEST PIT LOCATION, CURRENT INVESTIGATION
- HORIZONTAL DRILLHOLE
- APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION BY QUONTACON ASSOCIATES, 1999
- APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION BY SITE INVESTIGATION SERVICES LIMITED, 1977

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 1528810

REFERENCE(S)

1. BASE PLAN SUPPLIED IN ELECTRONIC FORMAT BY MMM GROUP LIMITED

0 5 10
1:250 METRES

CLIENT
MMM GROUP

PROJECT
GEOTECHNICAL INVESTIGATION
KINGSTON MILLS SWING BRIDGE
KINGSTON, ONTARIO

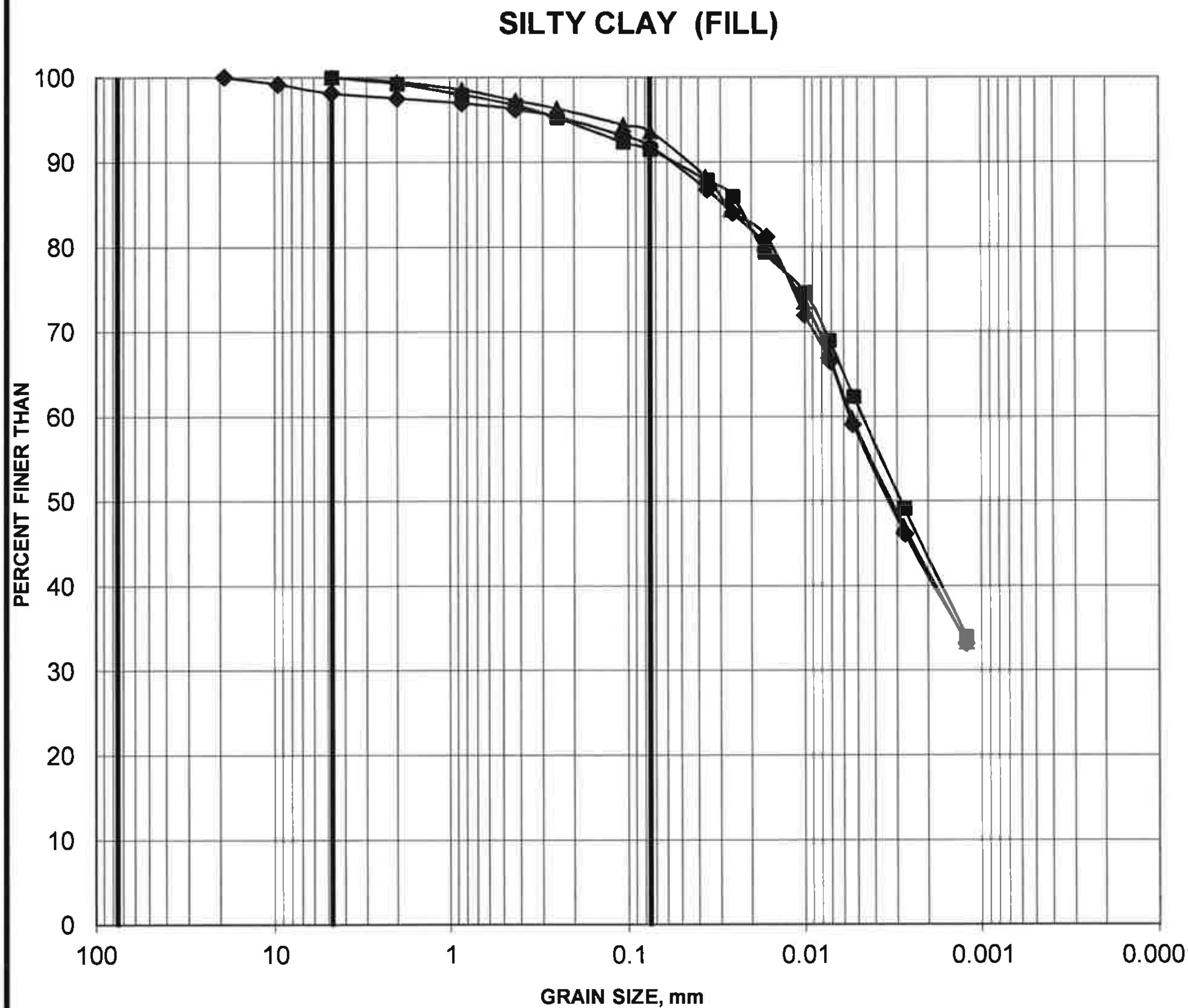
TITLE
SITE PLAN

CONSULTANT
Golder Associates

YYYY-MM-DD	2015-06-03
DESIGNED	----
PREPARED	JM
REVIEWED	TJN
APPROVED	WC

PROJECT NO.	PHASE	REV.	FIGURE
1528810	1000	A	1

GRAIN SIZE DISTRIBUTION	FIGURE 2



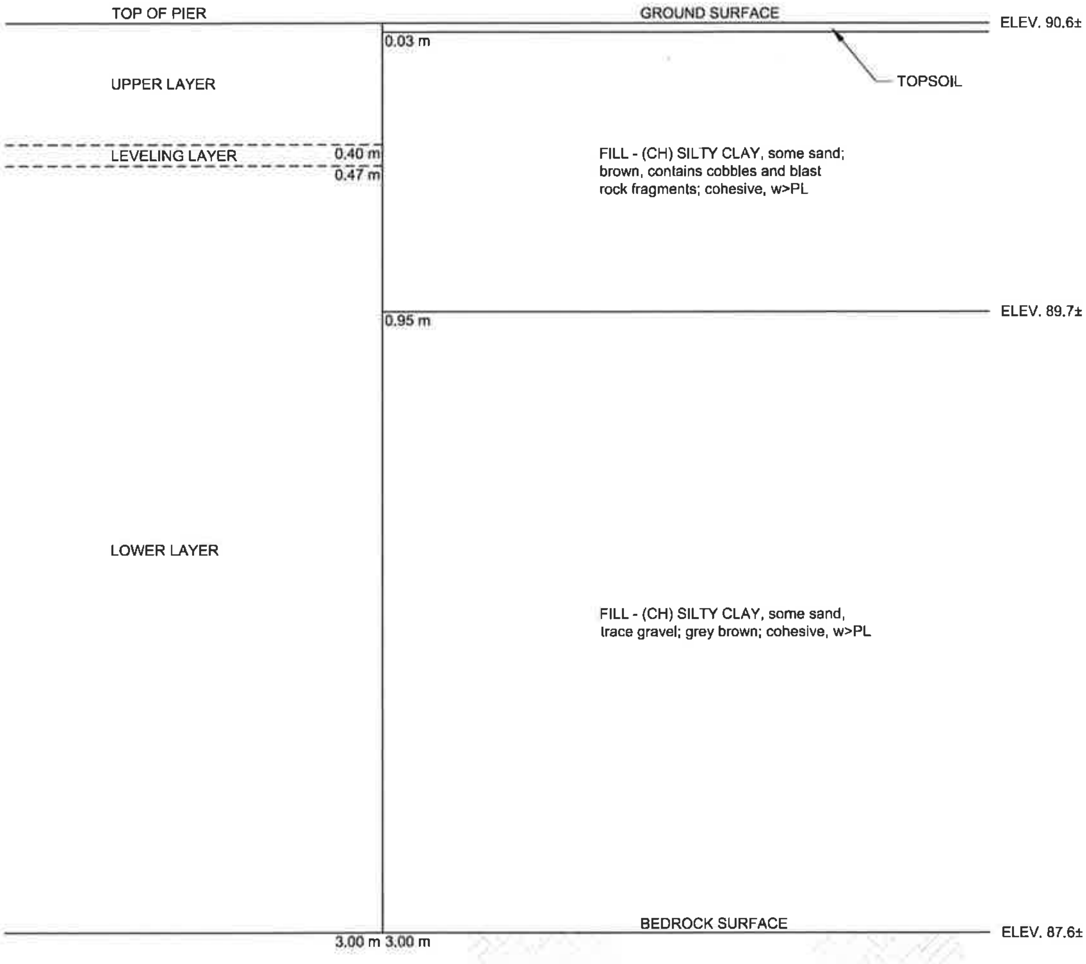
Cobble	coarse	fine	coarse	medium	fine	SILT AND CLAY
Size	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)
15-1A (■)	2	0.47-0.95
15-1A (◆)	3	1.10-1.80
15-1A (▲)	4	1.80-2.80

Project: 1528810

Golder Associates

Created by: CNM
Checked by: WAM



NOT TO SCALE

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 1528810

CLIENT	PROJECT
MMM GROUP	GEOTECHNICAL INVESTIGATION KINGSTON MILLS SWING BRIDGE KINGSTON, ONTARIO

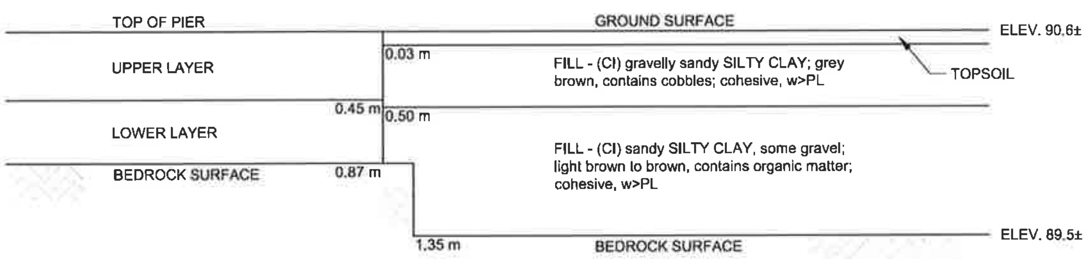
CONSULTANT: Golder Associates

YYYY-MM-DD	2015-07-16
DESIGNED	TJN
PREPARED	JM
REVIEWED	TJN
APPROVED	WC

TITLE

TEST PIT 15-1A SECTION

PROJECT NO.	PHASE	REV.	FIGURE
1528810	1000	A	3



NOT TO SCALE

NOTE(S)

1. THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 1528810

CLIENT
MMM GROUP

CONSULTANT

Golder Associates

YYYY-MM-DD	2015-07-16
DESIGNED	TJN
PREPARED	JM
REVIEWED	TJN
APPROVED	WC

PROJECT
**GEOTECHNICAL INVESTIGATION
KINGSTON MILLS SWING BRIDGE
KINGSTON ONTARIO**

TITLE
TEST PIT 15-2A SECTION

PROJECT NO.	PHASE	REV.	FIGURE
1528810	1000	A	4

APPENDIX A

List of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Record of Borehole and Drillhole Sheets – Current Investigation

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.*, SAND and GRAVEL, SAND and CLAY)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.).

Cone Penetration Test (CPT)
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size
TP	Thin-walled, piston – note size
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 - 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects.
2. Definition of compactness descriptions based on SPT 'N' ranges from Terzaghi and Peck (1967) and correspond to typical average N_{60} values.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 0 analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)
The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)
The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)
The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index
A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes
An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN	Joint	PL	Planar
FLT	Fault	CU	Curved
SH	Shear	UN	Undulating
VN	Vein	IR	Irregular
FR	Fracture	K	Slickensided
SY	Stylolite	PO	Polished
BD	Bedding	SM	Smooth
FO	Foliation	SR	Slightly Rough
CO	Contact	RO	Rough
AXJ	Axial Joint	VR	Very Rough
KV	Karstic Void		
MB	Mechanical Break		

PROJECT: 1528810

RECORD OF BOREHOLE: 15-101

LOCATION: See Site Plan

BORING DATE: May 14, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE									
0	Power Auger 200 mm Diam. (Hollow Stem)	ASPHALTIC CONCRETE		0.00							
		FILL - (SW) gravelly SAND; grey (PAVEMENT STRUCTURE)		0.10	1	GRAB	-				
		FILL - (CI) SILTY CLAY, some sand, trace gravel; dark brown, contains organic matter; cohesive, w>PL		0.25	2	GRAB	-				
1		FILL - (SM) gravelly SILTY SAND; dark brown, contains cobbles and boulders; non-cohesive, moist, loose to compact		0.61	3	SS	8				
2					4	SS	29				
					5	SS	17				
3		Borehole continued on RECORD OF DRILLHOLE 15-101		2.73							
4											
5											
6											
7											
8											
9											
10											

MIS-BHS 001 1528810.GPJ GAL-MIS.GDT 11/13/15 JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: RI

CHECKED: WC

PROJECT: 1528810

RECORD OF DRILLHOLE: 15-101

LOCATION: See Site Plan

DRILLING DATE: May 14, 2015

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

JN - Joint, FLT - Fault, SHR - Shear, VN - Vein, CJ - Conjugate, BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage, PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular, PO - Polished, K - Slickensided, SM - Smooth, Ro - Rough, MB - Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	NOTES
		BEDROCK SURFACE					
3	Rotary Drill, NQ Core	Fresh, massive, pink and dark grey, medium to coarse grained, non-porous, slightly foliated, strong GRANITE, with occasional thin (<1 mm) calcite veins	2.73	1	100	JN,PL,RO; JN,PL,RO; JN,UN,RO; JN,PL,RO	
		- Possible lost core from 3.37 m to 3.42 m		2	100	JN,CU,RO; JN,CU,RO; JN,PL,RO	
4						JN,PL,RO	
5		- Possible lost core from 5.11 m to 5.12 m		3	100-0	JN,PL,RO; JN,PL,RO; JN,PL,RO; JN,UN,RO; JN,PL,RO	
		End of Drillhole	5.21				WL in open borehole at 3.25 m depth below ground surface upon completion of drilling

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: RI

CHECKED: WC

RECORD OF BOREHOLE: 15-202

PROJECT: 1528810

SHEET 1 OF 2

LOCATION: See Site Plan

BORING DATE: May 13, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢ ⊖W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		WALL FACE									
0	Rotary Drill, 95 mm Thin Wall	LIMESTONE BLOCK MASONRY		0.00							
		TCR = 92%, SCR = 76%			1	RC	DD				
	Rotary Drill, 67 mm Thin Wall	GRANITE fragments and grout (Grouted Rubble Infill)		0.66							
1		TCR = 78%, SCR = 7%			2	RC	DD				
2		TCR = 82%, SCR = 45%			3	RC	DD				
		Borehole continued on RECORD OF DRILLHOLE 15-202		2.11							
3											
4											
5											
6											
7											
8											
9											
10											

MIS-BHS 001 1528810.GPJ GAL-MIS.GDT 11/13/15 JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: RI

CHECKED: WC

PROJECT: 1528810

RECORD OF DRILLHOLE: 15-202

SHEET 2 OF 2

LOCATION: See Site Plan

DRILLING DATE: May 13, 2015

DATUM: Geodetic

INCLINATION: 0° AZIMUTH:

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: Marathon Drilling

JN - Joint, FLT - Fault, SHR - Shear, VN - Vein, CJ - Conjugate, BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage, PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular, PO - Polished, K - Slickensided, SM - Smooth, Ro - Rough, MB - Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE % / SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		BEDROCK SURFACE											
	Rotary Drill, 43 mm Thin Wall	Fresh, massive, pink and dark grey, medium to coarse grained, non-porous, slightly foliated, strong GRANITE, with occasional thin (<1 mm) calcite veins - Fracture from 2.56 m to 2.83 m and 2.79 m to 2.98 m		2.11	4	100				JN,PL,RO JN,PL,RO JN,PL,RO JN,PL,RO			
3					5	100				JN,PL,RO JN,UN,RO JN,PL,RO			
		End of Drillhole		3.35									
4													
5													
6													
7													
8													
9													
10													
11													
12													

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: RI

CHECKED: WC

PROJECT: 1528810

RECORD OF BOREHOLE: 15-203

LOCATION: See Site Plan

BORING DATE: May 11, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp I—○W—I Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		WALL FACE									
0	Portable Drill / AW Rods	LIMESTONE BLOCK MASONRY		0.00							
		GRANITE fragments and grout (Grouted Rubble Infill) TCR = 100%, SCR = 62%		0.31	1	RC	DD				
1		TCR = 100%, SCR = 0%			2	RC	DD				
		TCR = 61%, SCR = 0%			3	RC	DD				
		TCR = 100%, SCR = 0%			4	RC	DD				
		TCR = 83%, SCR = 0%			5	RC	DD				
		Borehole continued on RECORD OF DRILLHOLE 15-203		1.62							
2											
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: RI

CHECKED: WC

MIS-BHS 001 1528810.GPJ GAL-MIS.GDT 11/13/15 JM

RECORD OF DRILLHOLE: 15-203

PROJECT: 1528810

SHEET 2 OF 2

LOCATION: See Site Plan

DRILLING DATE: May 11, 2015

DATUM: Geodetic

INCLINATION: 0° AZIMUTH:

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: Marathon Drilling

JN - Joint, FLT - Fault, SHR - Shear, VN - Vein, CJ - Conjugate, BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage, PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular, PO - Polished, K - Slickensided, SM - Smooth, Ro - Rough, MB - Mechanical Break, BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		BEDROCK SURFACE																	
2	Portable Drill, AW Rods	Fresh, massive, pink and dark grey, medium to coarse grained, non-porous, slightly foliated, strong GRANITE, with occasional thin (<1 mm) calcite veins		1.62	6	100													
					7	100													
		End of Drillhole		2.62															

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: RI

CHECKED: WC

MIS-RCK 004 1528810.GPJ GAL-MISS.GDT 11/13/15 JM

PROJECT: 1528810

RECORD OF BOREHOLE: 15-204

SHEET 1 OF 2

LOCATION: See Site Plan

BORING DATE: May 12, 2015

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		WALL FACE									
0	Portable Drill, AW Rods	LIMESTONE BLOCK MASONRY TCR = 100%, SCR = 100%		0.00	1	RC	DD				
		TCR = 82%, SCR = 75%			2	RC	DD				
		GRANITE fragments and grout (Grouted Rubble Infill) TCR = 85%, SCR = 30%		0.51	3	RC	DD				
1		TCR = 100%, SCR = 5%			4	RC	DD				
2		TCR = 100%, SCR = 58%			5	RC	DD				
		TCR = 100%, SCR = 51%			6	RC	DD				
		Borehole continued on RECORD OF DRILLHOLE 15-204		2.75							
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: RI
CHECKED: WC

MIS-BHS 001 1528810.GPJ GAL-MIS.GDT 11/13/15 JM

PROJECT: 1528810

RECORD OF DRILLHOLE: 15-204

LOCATION: See Site Plan

DRILLING DATE: May 12, 2015

DATUM: Geodetic

DRILL RIG: Portable Drill

INCLINATION: 0° AZIMUTH:

DRILLING CONTRACTOR: Marathon Drilling

JN - Joint, FLT - Fault, SHR - Shear, VN - Vein, CJ - Conjugate; BD - Bedding, FO - Foliation, CO - Contact, OR - Orthogonal, CL - Cleavage; PL - Planar, CU - Curved, UN - Undulating, ST - Stepped, IR - Irregular; PO - Polished, K - Slickensided, SM - Smooth, Ro - Rough, MB - Mechanical Break; BR - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH COLOUR % RETURN	TOTAL CORE %	SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.25 m	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jcon	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.
		BEDROCK SURFACE																	
3	Portable Drill, AW Rods	Fresh, massive, pink and dark grey, medium to coarse grained, non-porous, slightly foliated, strong GRANITE, with occasional thin (<1 mm) calcite veins - Fracture from 2.56 m to 2.83 m and 2.79 m to 2.98 m - Fracture from 3.30 m to 3.79 m and 3.63 m to 3.88 m		2.75	6	100							,JN,PL,RO						
					7	100							,JN,PL,RO ,JN,ST,RO ,JN,ST,RO ,JN,PL,RO ,JN,PL,RO						
4		End of Drillhole		3.95															
5																			
6																			
7																			
8																			
9																			
10																			
11																			
12																			

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: RI

CHECKED: WC

APPENDIX B

Record of Borehole Sheets and Laboratory Testing Results – Previous Investigations

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

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 N	COMMENTS
		302.4		GROUND SURFACE							
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: ½" Ø ROUND DEFORMED BAR AT 1'8" AND 4'7", NO CORROSION. COMPRESSIVE STRENGTH SPECIMEN
5	1,0			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.
8.0	2,0			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							
	4,0			NOTE: SAMPLE TAKEN FOR COMPRESSIVE STRENGTH TEST BETWEEN 2'8" AND 3'4".							

REFERENCE No.: 20226-K-6160B ENCLOSURE No.: 2

INSPEC-SOL

BOREHOLE No.: C-2
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 30, 1999 DATE (FINISH): December 1, 1999

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

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 N	COMMENTS
		302.4		GROUND SURFACE							
0 – 6.0	0 – 1,0			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 8'. REACTION RIMS VISIBLE ON EXPOSED CONCRETE SURFACE TO DEPTH OF 6'. SOME SILICA GEL DEPOSITED IN OPEN PORES IN UPPER 1'-8".		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.
6.0 – 12.3	2,0 – 3,0			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.
12.3	4,0			END OF BOREHOLE NOTE: SAMPLE TAKEN FOR COMPRESSIVE STRENGTH TEST BETWEEN 1'2" AND 1'10"							

Depth scale: Feet 0, 5, 6.0, 10, 12.3, 15; Metres 0, 1,0, 2,0, 3,0, 4,0

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 (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			2" ASPHALT OVER CONCRETE WITH AIR ENTRAINMENT, NORMALLY CONSOLIDATED. NO MICRO FISSURES OR REACTION RIMS VISIBLE. DELAMINATION AT 4" FROM SURFACE.		RC-1	100		100%		COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 55% CRYSTALLINE LIMESTONE: 5 % ARGILLACEOUS LIMESTONE: 40 %. FINE AGGREGATE: 0- ¼", SUB ANGULAR TO SUB ROUNDED, GRANITIC.
				END OF BOREHOLE							

Depth scale: Feet 0, 1.0, 5, 10, 15; Metres 0, 1,0, 2,0, 3,0, 4,0

REFERENCE No.: 20226-K-6160B ENCLOSURE No.: 4

INSPEC-SOL

BOREHOLE No.: C-4
ELEVATION: 301.47 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		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			ASPHALT 1½", THICK, NO MEMBRANE PRESENT. CONCRETE WITH AIR ENTRAINMENT, NORMALLY CONSOLIDATED. NO MICRO FISSURES OR REACTION RIMS VISIBLE. DELAMINATION AT 10" FROM SURFACE. 5/8" ROUND, DEFORMED REBAR AT 4" Ø FROM SURFACE. GOOD BOND WITH PASTE, NO CORROSION.		RC-1	100	[illegible]	100%		COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 25% CRYSTALLINE LIMESTONE: 10 % ARGILLACEOUS LIMESTONE: 65 %
1.3											
	1,0			END OF BOREHOLE NOTE: SAMPLE SUBMITTED FOR COMPRESSIVE STRENGTH TESTING.							
5											
	2,0										
10	3,0										
	4,0										

REFERENCE No.: 20226-K-6160B

ENCLOSURE No.: 5

INSPEC-SOL

BOREHOLE No.: C-5

ELEVATION: 296.62 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): 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 (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
		296.62		GROUND SURFACE			%			N	
0–5.5	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		82% AIR VOID SAMPLE COMP.STR. SAMPLE TENSILE STR. SAMPLE		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.
5.5–7.5				POSSIBLE FILL: BROWN CLAYEY SILT TILL, WITH TRACES OF GRAVEL, MOIST		SS-1	100				
7.5				END OF BOREHOLE							

Depth scale: Feet 0, 5, 5.5, 7.5, 10, 15; Metres 0, 1.0, 2.0, 3.0, 4.0

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".

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

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 (N)	COMMENTS
		296.7		GROUND SURFACE							
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.		RC-1	100		COMP.STR. SAMPLE; 93%; AIR VOID SAMPLE; TENSILE STR. SAMPLE		COARSE AGGREGATE: ¾" NOMINAL MICRITIC LIMESTONE: 20 % CRYSTALLINE LIMESTONE: 5 % ARGILLACEOUS LIMESTONE: 75 %. FINE AGGREGATE: 0-¼", 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										
4.8 / 5				POSSIBLE FILL: BROWN CLAYEY SILT TILL, WITH TRACES OF GRAVEL, HUMID		SS-1	100				
6.3	2,0			END OF BOREHOLE							
10	3,0										
	4,0										

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".

BOREHOLE DATA and TEST SUMMARY

Project: KINGSTON MILLS
Location: SWING BRIDGE
Hole Location: SEE PLAN

Date: March 9, 1977
Elevation Datum:
Type of Drill: 3¼" H.S.A. (D-4)

JOB No: 1608 BOREHOLE No: K

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

PENETRATION RESISTANCE (Blows/Ft)
2" O.D. Split Spoon — 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
		294.8				
TOPSOIL						
CLAYEY SILT FILL - greyish brown sandy clayey silt fill, some pebbles and bricks below 7 feet - traces of organics - moist to wet - stiff						X X X X S D S
SAND - LIMESTONE			15			
NO FURTHER PROGRESS (presumed bedrock)			20			

N = 50 blows/0" Bouncing

Note: Hole dry on completion

SITE INVESTIGATION SERVICES Ltd. JOB No: 1608

BOREHOLE DATA an[illegible] TEST SUMMARY

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

Date - March 9, 1977
Elevation Datum -
Type of Drill - $3\frac{1}{4}$'' H.S.A. (D-4)

JOB No: 1608 BOR[illegible]E No: [illegible]

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

PENETRATION RESISTANCE (Blows/Ft)
2" O.D. Split Spoon —o—o— 2" O.D. Cone —•—•—

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	PENETRATION RESISTANCE / SHEAR STRENGTH
		294.4					
TOPSOIL			4"				
CLAYEY SANDY SILT FILL - brown pebbly clayey sandy silt fill - moist to wet						S	
NO FURTHER PROGRESS (presumed bedrock)							
Note: Tried Two holes			15				
			20				

TABLE 1

KINGSTON MILLS SWING BRIDGE

SUMMARY OF PROBE HOLES

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

BOREHOLE DATA AND TEST SUMMARY

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

Date - December 22, 1976
Elevation Datum -
Type of Drill - 3¼" H.S.A. (D-8)

JOB No: 1608
BOREHOLE No:

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	PENETRATION RESISTANCE / SHEAR STRENGTH
ASPHALT (4") over GRAVELLY SAND		301.4					
SAND FILL - brown sand fill mixed with cobbles							
CLAYEY SANDY SILT - dark brownish grey stony sandy clayey silt fill							
ROCKFILL			5				
REFUSAL IN ROCKFILL							
REFUSAL OF CONE PROBE		293.7					50 blows/9"
			10				
			15				
			20				

BOREHOLE DATA a[illegible] TEST SUMMARY

Project - KINGSTON MILLS

Location - SWING BRIDGE ABUTMENT

Hole Location - SEE PLAN

Date - December 23, 1976

Elevation Datum -

Type of Drill - $3\frac{1}{4}$'' H.S.A. (D-8)

JOB No: 1608 BOREHOLE No: H

LEGEND

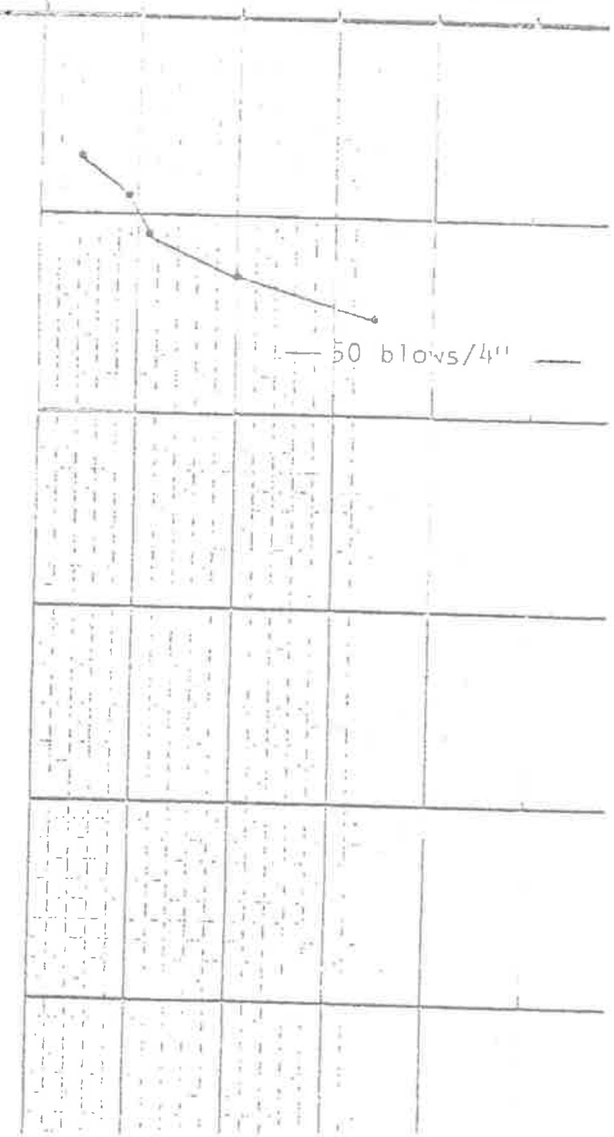
Gravel Sand [illegible] Clay

(See Appendix "A" for Other Symbols)

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

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 - □



BOREHOLE DATA a TEST SUMMARY

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)

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	PENETRATION RESISTANCE (Blows/Ft) 2" O.D. Split Spoon / 2" O.D. Cone (10 20 30 40 50); SHEAR STRENGTH (Kips/Ft²) Field Vane - X, Unconfined Compression - □
TOPSOIL		297.6					
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	98+% Recovery
END OF HOLE (Note: standpipe installed to 14' depth)			15 20				

APPENDIX C

Photographs of Bedrock Core

BH 15-101 Wet
Cored Length of 2.73 to 5.21 metres
Core Box 1 and 2 of 2

2.73 m Top of Bedrock

5.21 m EOH

Golder Associates

Geotechnical Investigation

Kingston Mills Swing Bridge

Kingston, Ontario

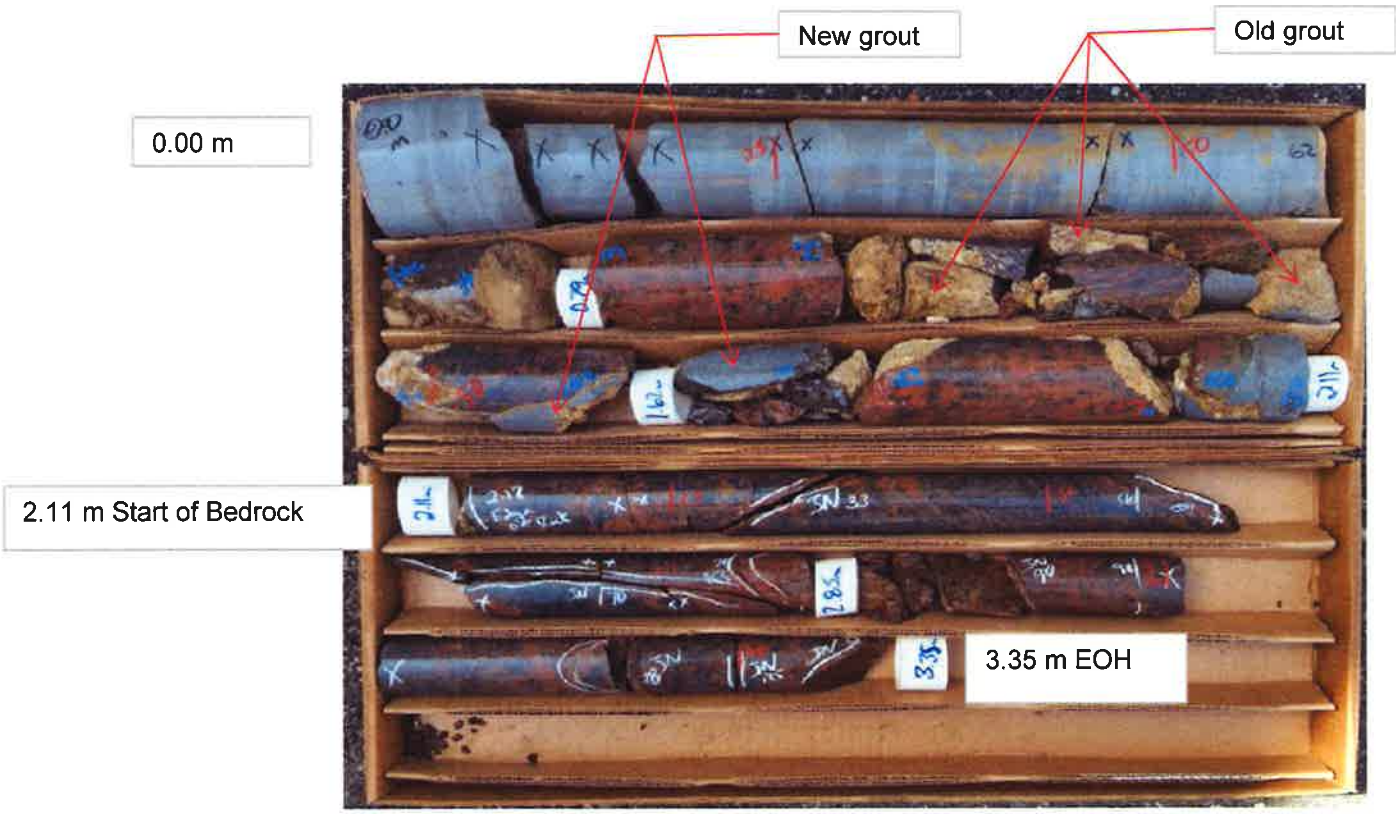
Project No.	1528810
Drawn:	WAM
Date:	7/14/2015
Checked:	TJN
Review:	WC

15-101
1 of 1

BH 15-202 Wet

Cored Length of 0.00 to 3.35 metres

Core Box 1 and 2 of 2



Note 1: Material in core box from 0.00 to 0.66 m is limestone block
Material in core box form 0.66 to 2.11 overburden (gravel and cobbles in fill).

Golder Associates	**Geotechnical Investigation** **Kingston Mills Swing Bridge** **Kingston, Ontario**	Project No. 1528810 Drawn: WAM Date: 7/14/2015 Checked: TJN Review: WC	**15-202** **1 of 1**

BH 15-203 Wet
Cored Length of 0.00 to 2.62 metres
Core Box 1 and 2 of 2

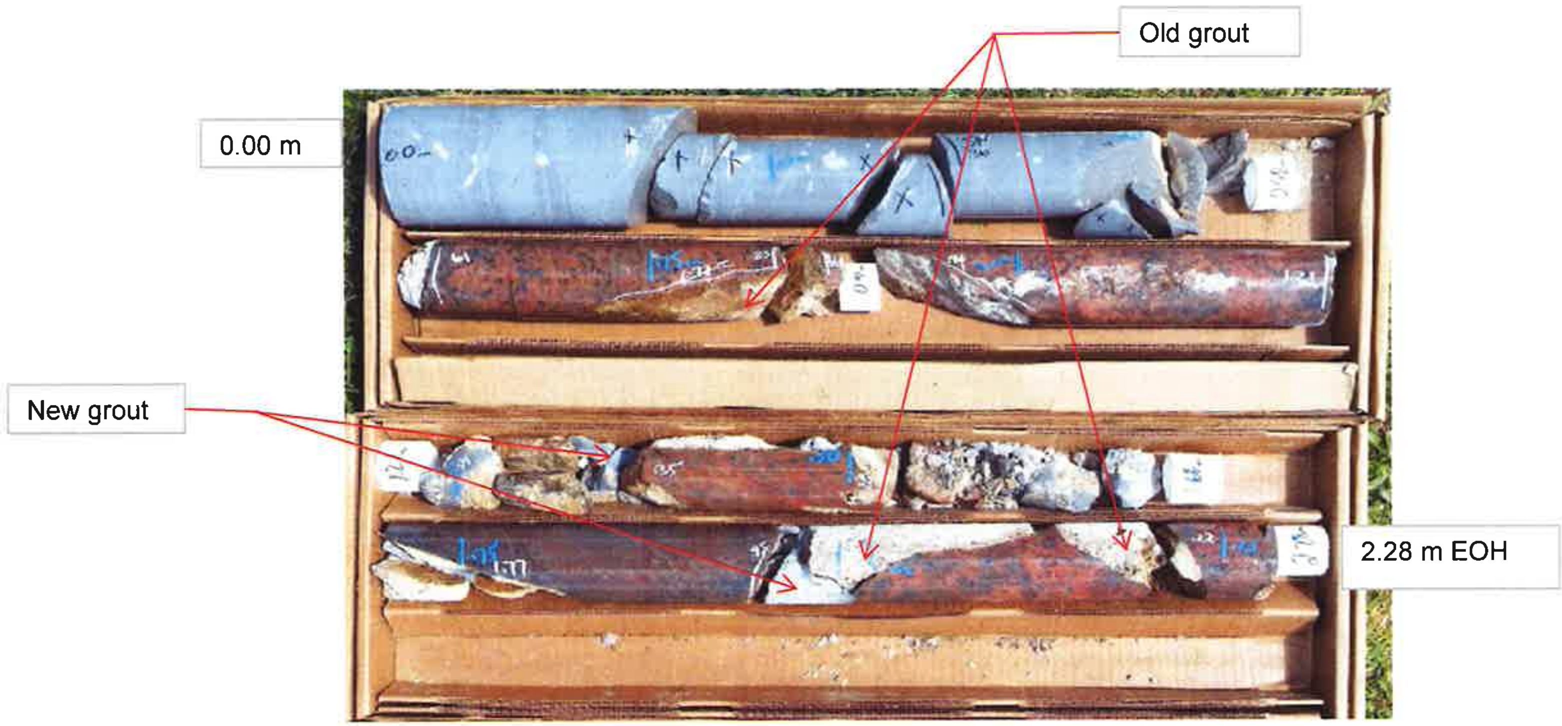
Note 1: Material in core box from 0.00 to 0.31 m is limestone block
Material in core box from 0.31 to 1.67 overburden (gravel and cobblesin fill).

Golder Associates	**Geotechnical Investigation** **Kingston Mills Swing Bridge** **Kingston, Ontario**	Project No. 1528810 Drawn: WAM Date: 7/14/2015 Checked: TJN Review: WC	**15-203** **1 of 1**

BH 15-204 Wet

Cored Length of 0.00 to 2.28 metres

Core Box 1 to 2 of 3



Note 1: Material in core box from 0.00 to 0.51 m is limestone block
Material in core box from 0.51 to 2.28 overburden (gravel and cobblesin fill).

Golder Associates	**Geotechnical Investigation** **Kingston Mills Swing Bridge** **Kingston, Ontario**	Project No. 1528810 Drawn: WAM Date: 7/14/2015 Checked: TJN Review: WC	**15-204** **1 of 2**

BH 15-202 Wet
Cored Length of 2.28 to 3.95 metres
Core Box 3 of 3

Note 1: Material in core box from 2.28 to 2.75 m overburden (gravel and cobblesin fill).

Golder Associates	**Geotechnical Investigation** **Kingston Mills Swing Bridge** **Kingston, Ontario**	Project No. 1528810 Drawn: WAM Date: 7/14/2015 Checked: TJN Review: WC	**15-204** **2 of 2**

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Golder Associates

APPENDIX

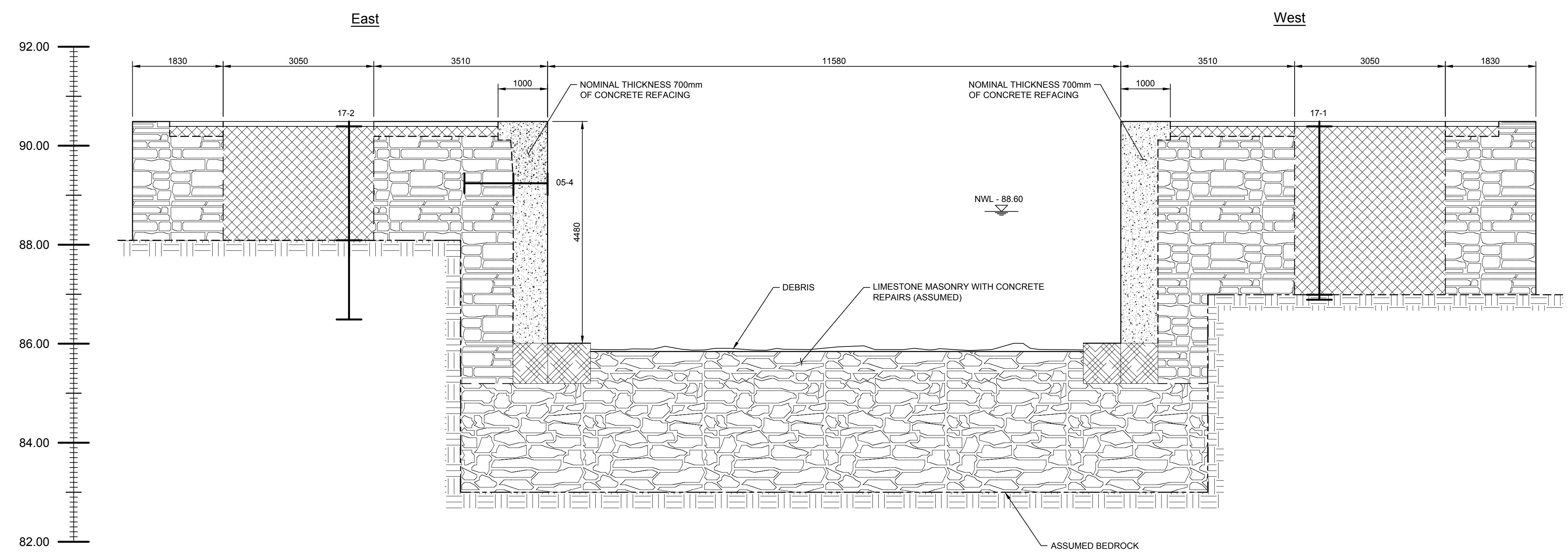
E

CROSS SECTION

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

- ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
- BASED ON GEOTECHINCAL INVESTIGATION
- BASED ON VISUAL 2017 INVESTIGATION
- EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
- TOPSOIL
- SANDY SILT / SANDY CLAY FILL
- CONCRETE
- FINISHED MASONRY / ASHLAR MASONRY
- GROUTED RUBBLE MASONRY
- GROUTED RUBBLE FILL
- BEDROCK
- NWL - XXX.XX NORMAL WATER LEVEL
- WWL - XXX.XX WINTER WATER LEVEL
- HWL - XXX.XX HIGH WATER LEVEL
- LWL - XXX.XX LOW WATER LEVEL



SECTION (A / S01) 1:50 LOCK 46 - UPPER GATE RECESS

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

Parks Canada
Parcs Canada

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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

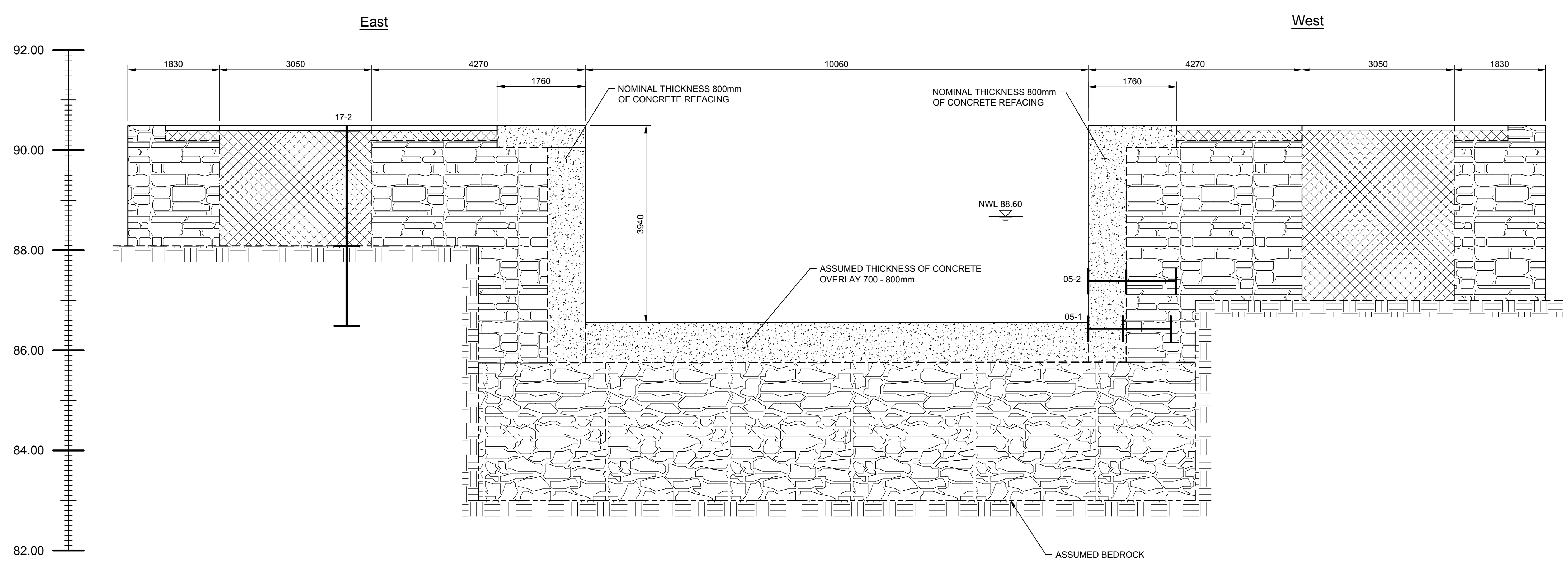
RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

LOCK 46 SECTION A

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number / Numéro du Dessin: GR-01
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1



SECTION B / S01 — 1:50 — LOCK 46 - UPPER GATE SILL

LEGEND:

(line type)	ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
(line type)	BASED ON GEOTECHINCAL INVESTIGATION
(line type)	BASED ON VISUAL 2017 INVESTIGATION
(line type)	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
(hatch)	TOPSOIL
(hatch)	SANDY SILT / SANDY CLAY FILL
(hatch)	CONCRETE
(hatch)	FINISHED MASONRY / ASHLAR MASONRY
(hatch)	GROUTED RUBBLE MASONRY
(hatch)	GROUTED RUBBLE FILL
(hatch)	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN — ONTARIO

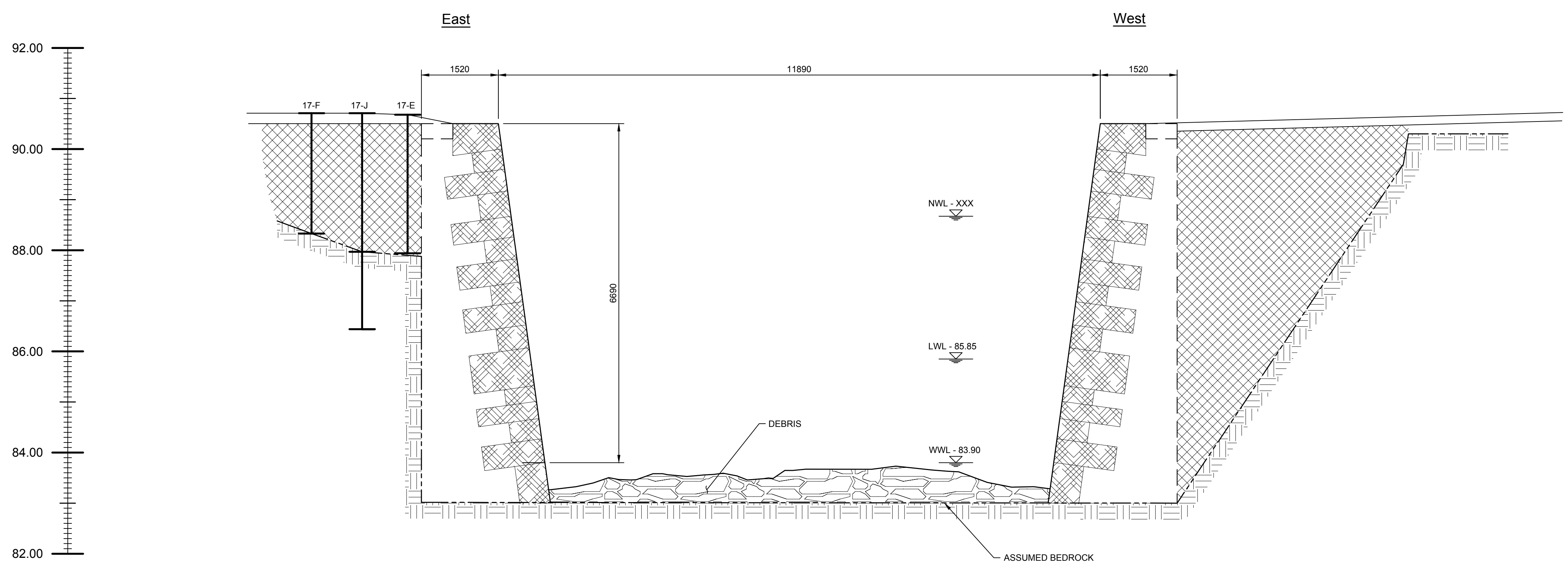
Drawing title / Titre du dessin

LOCK 46 SECTION B

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number / Numéro du Dessin: GR-02
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1

LEGEND:

— —	ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
– – – –	BASED ON GEOTECHINCAL INVESTIGATION
———	BASED ON VISUAL 2017 INVESTIGATION
—·—·—	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
	TOPSOIL
	SANDY SILT / SANDY CLAY FILL
	CONCRETE
	FINISHED MASONRY / ASHLAR MASONRY
	GROUTED RUBBLE MASONRY
	GROUTED RUBBLE FILL
	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



SECTION (C / S01) 1:50 LOCK 46 - US CHAMBER - EAST

Public Services and Procurement Canada / Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group / Parks Canada Infrastructure Directorate

Groupe Canaux historiques et travaux d'ingénierie / Direction de l'infrastructure de Parcs Canada

Parks Canada / Parcs Canada

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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

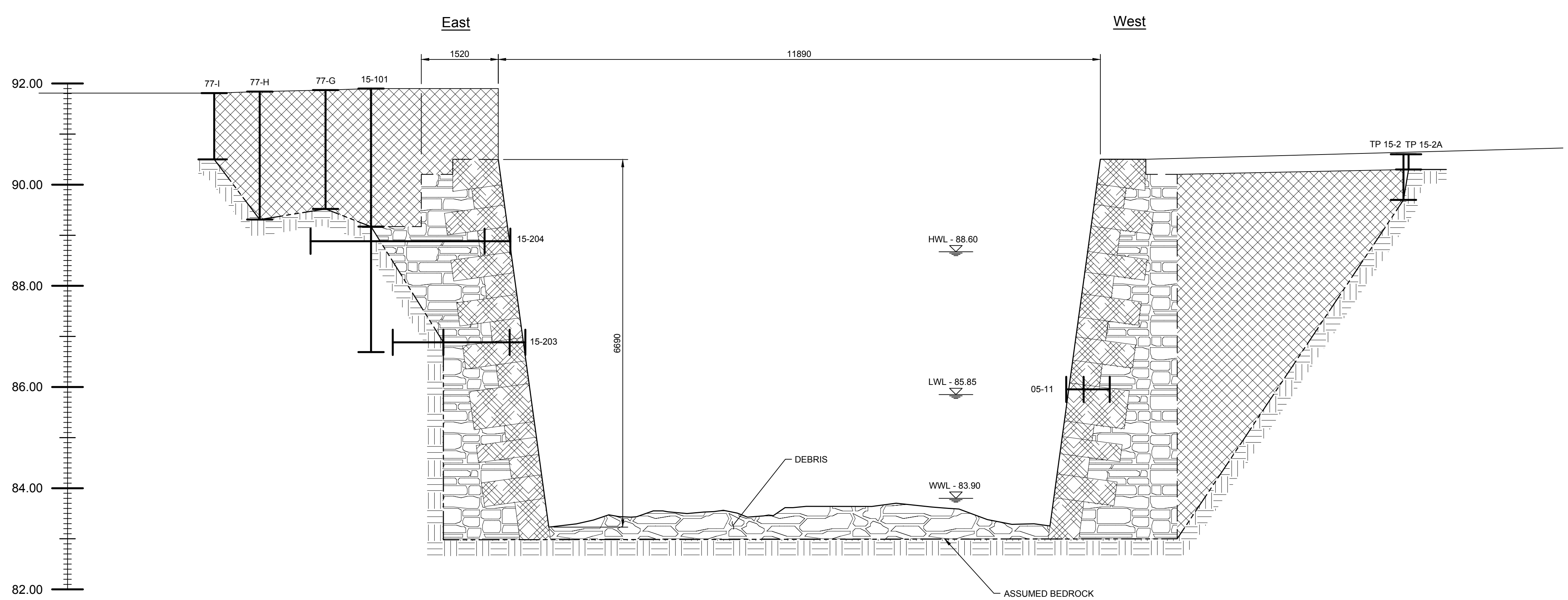
LOCK 46 SECTION C

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-03
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

— — (long dash)	ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
— — — (dashed)	BASED ON GEOTECHINCAL INVESTIGATION
——— (solid)	BASED ON VISUAL 2017 INVESTIGATION
—·— (dash-dot)	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
(hatch)	TOPSOIL
(hatch)	SANDY SILT / SANDY CLAY FILL
(hatch)	CONCRETE
(hatch)	FINISHED MASONRY / ASHLAR MASONRY
(hatch)	GROUTED RUBBLE MASONRY
(hatch)	GROUTED RUBBLE FILL
(hatch)	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



SECTION D / S01 LOCK 46 - MID CHAMBER
1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

Parks Canada
Parcs Canada
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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

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Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

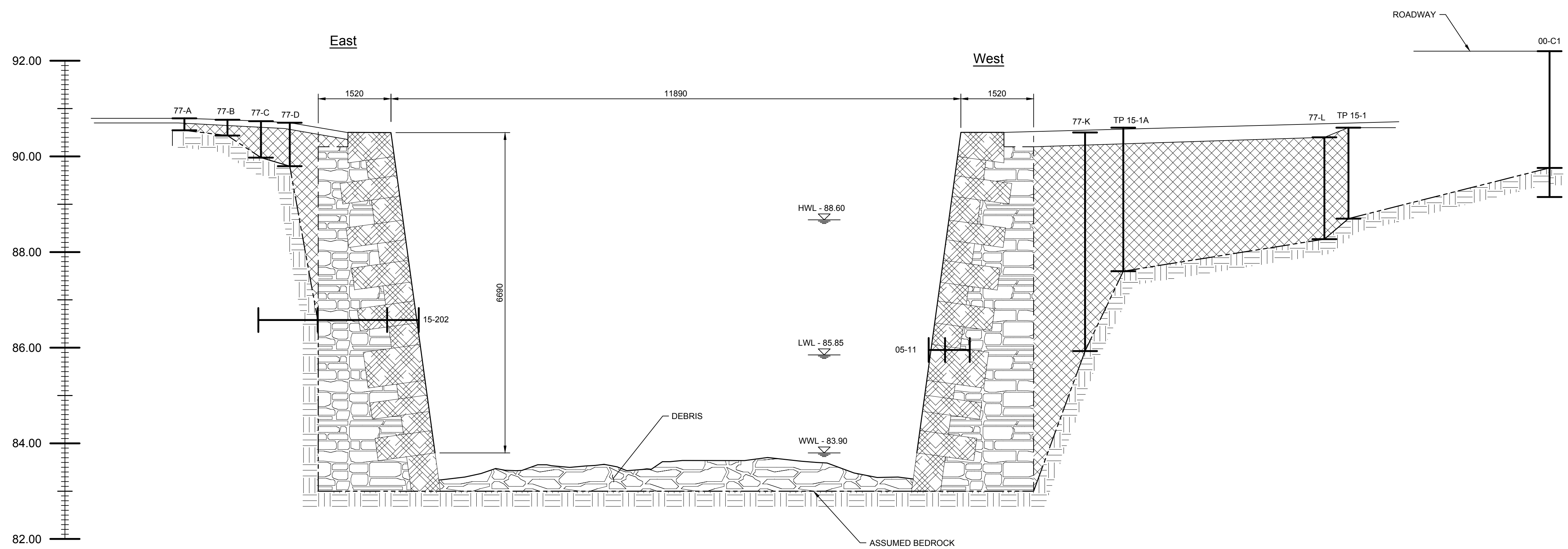
LOCK 46 SECTION D

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-04
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

— —	ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
– – – –	BASED ON GEOTECHINCAL INVESTIGATION
——	BASED ON VISUAL 2017 INVESTIGATION
—·—	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
	TOPSOIL
	SANDY SILT / SANDY CLAY FILL
	CONCRETE
	FINISHED MASONRY / ASHLAR MASONRY
	GROUTED RUBBLE MASONRY
	GROUTED RUBBLE FILL
	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



SECTION E / S01 LOCK 46 - CHAMBER - WEST
1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

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Revision / Révision

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Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

LOCK 46 SECTION E

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number / Numéro du Dessin: GR-05
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1

PWGSC-B1 (1000 x 707) PLOTTED BY: SHAWN.LAPAIN DATE PLOTTED: May 01, 2018 LAYOUT NAME: Lock 46 Sect. E FILE NAME: 800 CROSS SECTIONS_LOCK 46rev1.dwg

LEGEND:

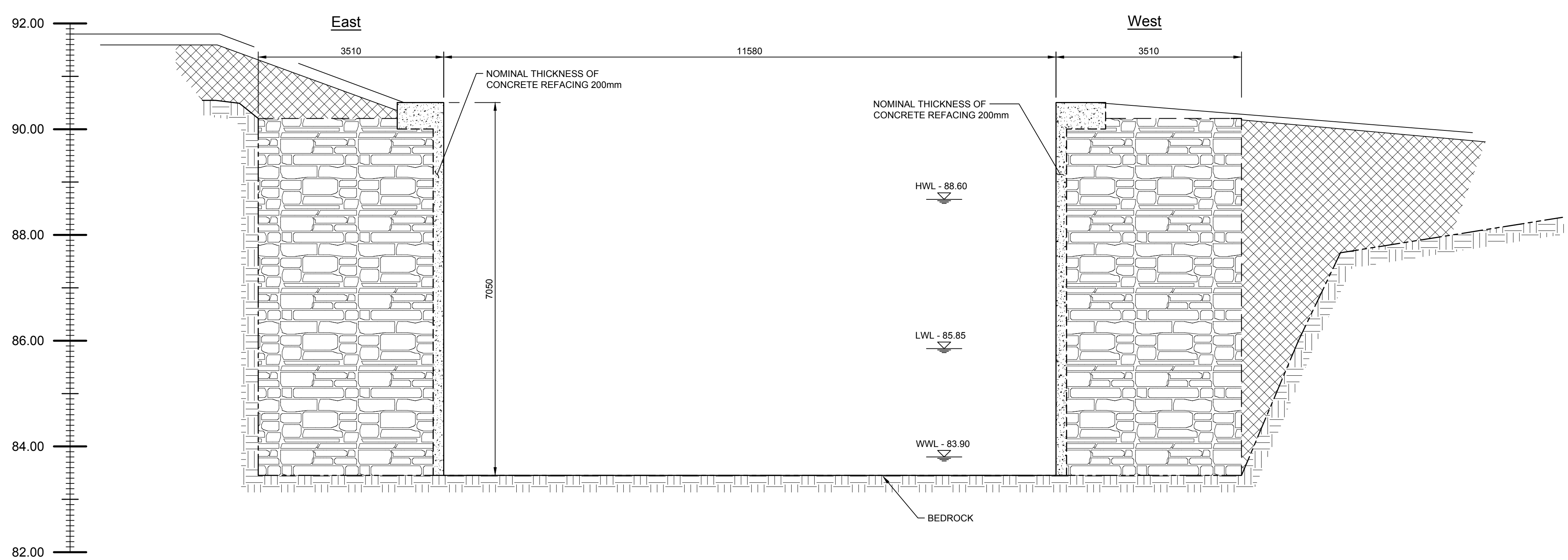
- ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
- BASED ON GEOTECHINCAL INVESTIGATION
- BASED ON VISUAL 2017 INVESTIGATION
- EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
- TOPSOIL
- SANDY SILT / SANDY CLAY FILL
- CONCRETE
- FINISHED MASONRY / ASHLAR MASONRY
- GROUTED RUBBLE MASONRY
- GROUTED RUBBLE FILL
- BEDROCK

NWL - XXX.XX NORMAL WATER LEVEL

WWL - XXX.XX WINTER WATER LEVEL

HWL - XXX.XX HIGH WATER LEVEL

LWL - XXX.XX LOW WATER LEVEL



SECTION F/S01 — 1:50 — LOCK 46 - LOWER GATE RECESS

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

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Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

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B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

LOCK 46 SECTION F

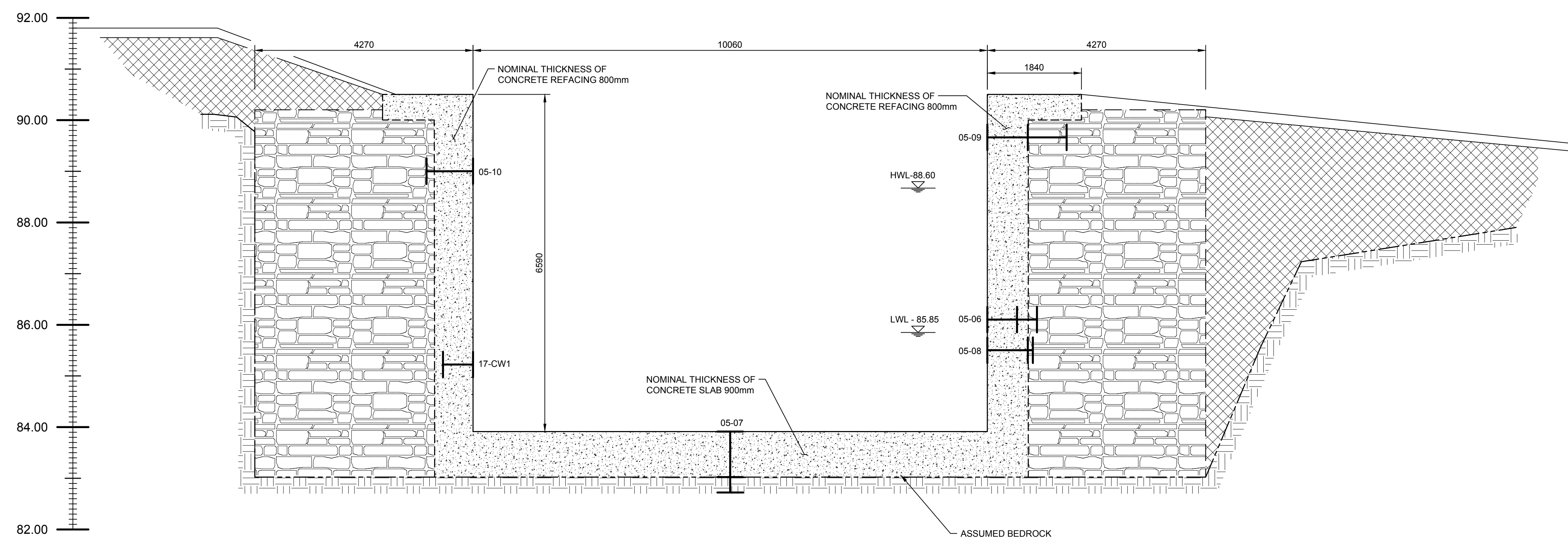
Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-06
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

- ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
- BASED ON GEOTECHINCAL INVESTIGATION
- BASED ON VISUAL 2017 INVESTIGATION
- EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
- TOPSOIL
- SANDY SILT / SANDY CLAY FILL
- CONCRETE
- FINISHED MASONRY / ASHLAR MASONRY
- GROUTED RUBBLE MASONRY
- GROUTED RUBBLE FILL
- BEDROCK

NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



SECTION G / S01 LOCK 46 - LOWER GATE SILL
1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

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A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

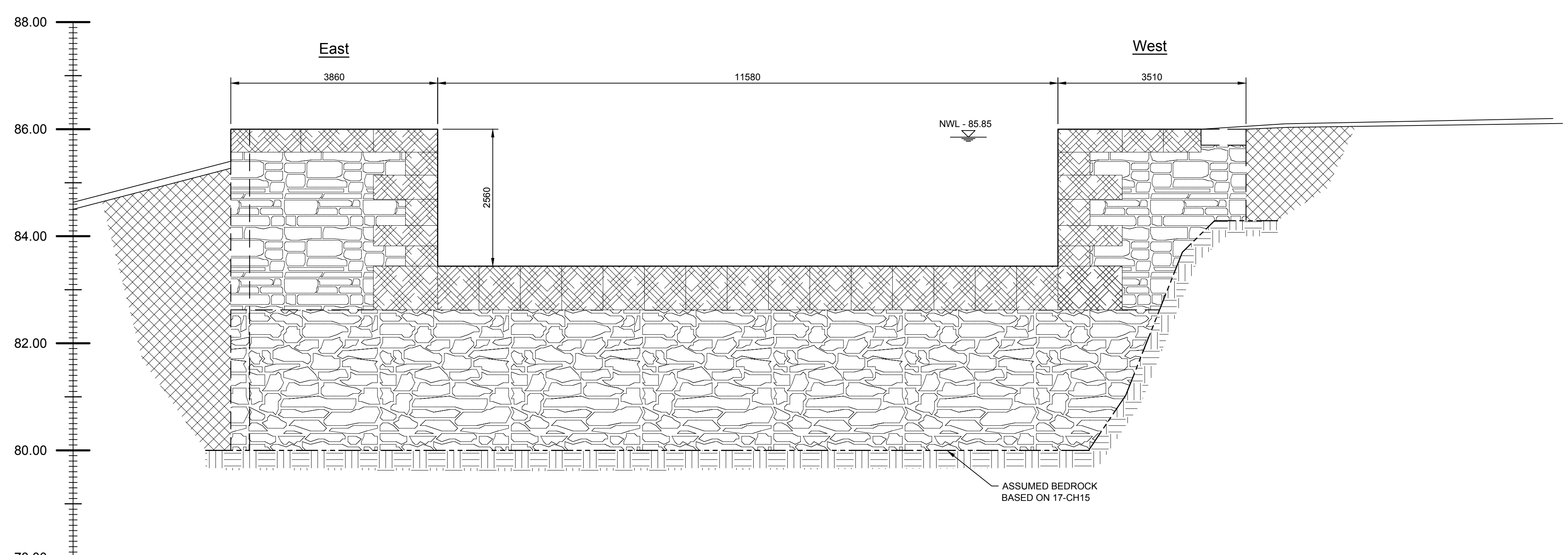
LOCK 46 SECTION G

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number / Numéro du Dessin: GR-07
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

- ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
- BASED ON GEOTECHINCAL INVESTIGATION
- BASED ON VISUAL 2017 INVESTIGATION
- EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
- TOPSOIL
- SANDY SILT / SANDY CLAY FILL
- CONCRETE
- FINISHED MASONRY / ASHLAR MASONRY
- GROUTED RUBBLE MASONRY
- GROUTED RUBBLE FILL
- BEDROCK
- NWL - XXX.XX NORMAL WATER LEVEL
- WWL - XXX.XX WINTER WATER LEVEL
- HWL - XXX.XX HIGH WATER LEVEL
- LWL - XXX.XX LOW WATER LEVEL



SECTION H / S01 — LOCK 47 - UPPER GATE RECESS

1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

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Revision / Révision

Do not scale drawings.
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A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

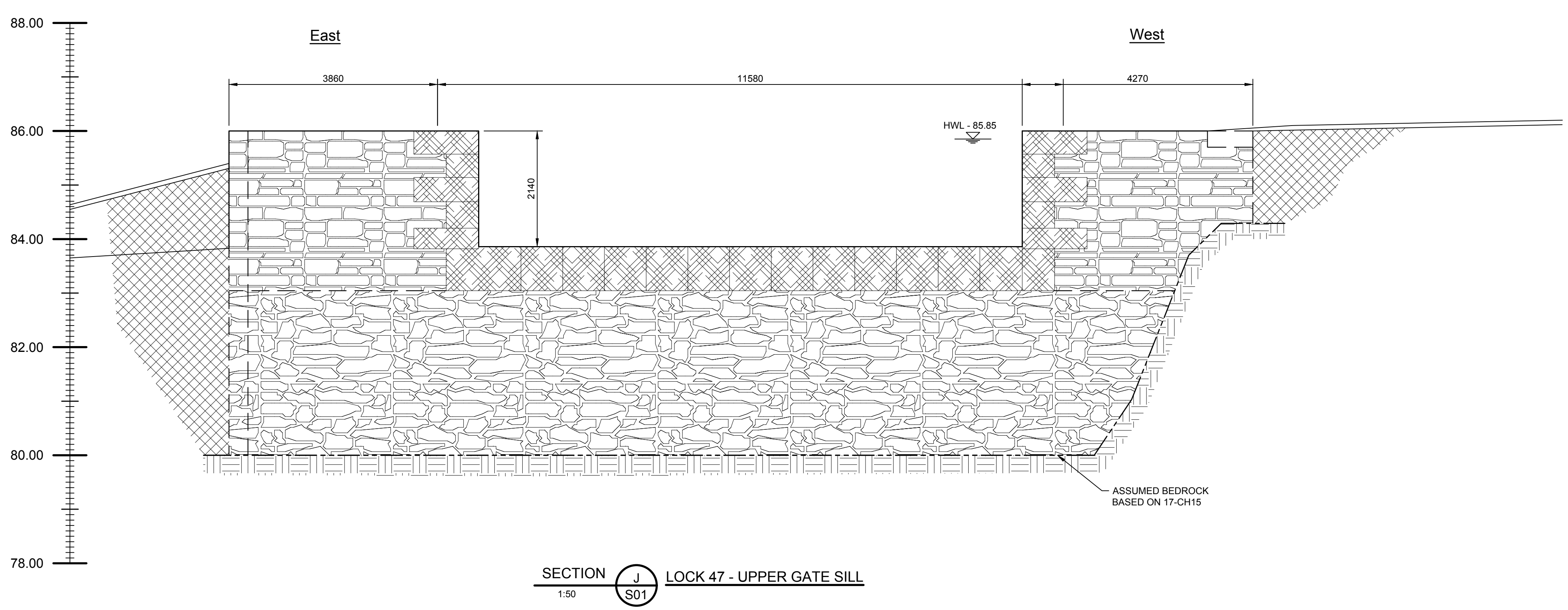
Drawing title / Titre du dessin

LOCK 47 SECTION H

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-08
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1

LEGEND:

- ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
- BASED ON GEOTECHINCAL INVESTIGATION
- BASED ON VISUAL 2017 INVESTIGATION
- EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
- TOPSOIL
- SANDY SILT / SANDY CLAY FILL
- CONCRETE
- FINISHED MASONRY / ASHLAR MASONRY
- GROUTED RUBBLE MASONRY
- GROUTED RUBBLE FILL
- BEDROCK
- NWL - XXX.XX NORMAL WATER LEVEL
- WWL - XXX.XX WINTER WATER LEVEL
- HWL - XXX.XX HIGH WATER LEVEL
- LWL - XXX.XX LOW WATER LEVEL



SECTION J / S01 LOCK 47 - UPPER GATE SILL
1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada
Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada
Parks Canada
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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

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A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

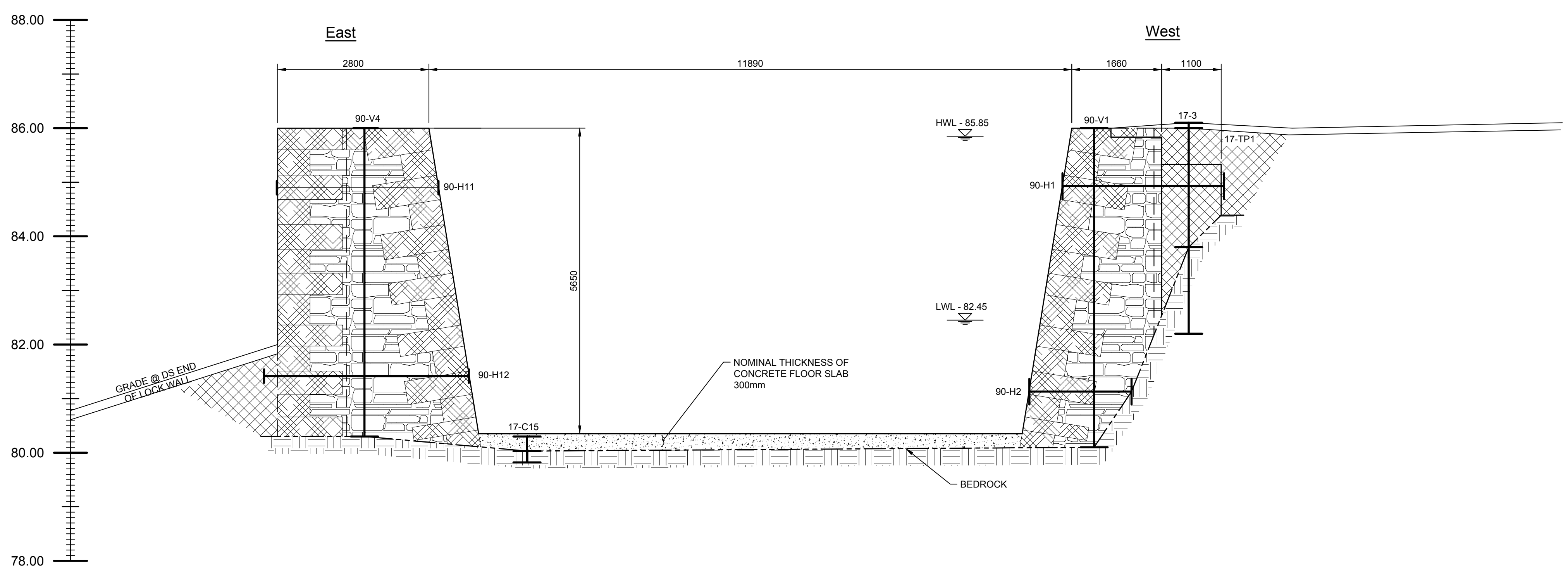
LOCK 47 SECTION J

Drawn by / Dessiné par: BN/AM/JH
Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON
Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION
Drawing Number / Numéro du Dessin: GR-09
Project Number / Numéro du projet: R.079796.009
Sheet / Feuille 1 of / du 1

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

——— ———	ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
— — — —	BASED ON GEOTECHINCAL INVESTIGATION
————	BASED ON VISUAL 2017 INVESTIGATION
— · — · —	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
	TOPSOIL
	SANDY SILT / SANDY CLAY FILL
	CONCRETE
	FINISHED MASONRY / ASHLAR MASONRY
	GROUTED RUBBLE MASONRY
	GROUTED RUBBLE FILL
	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



SECTION K / S01 LOCK 47 - CHAMBER

1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

LOCK 47 SECTION K

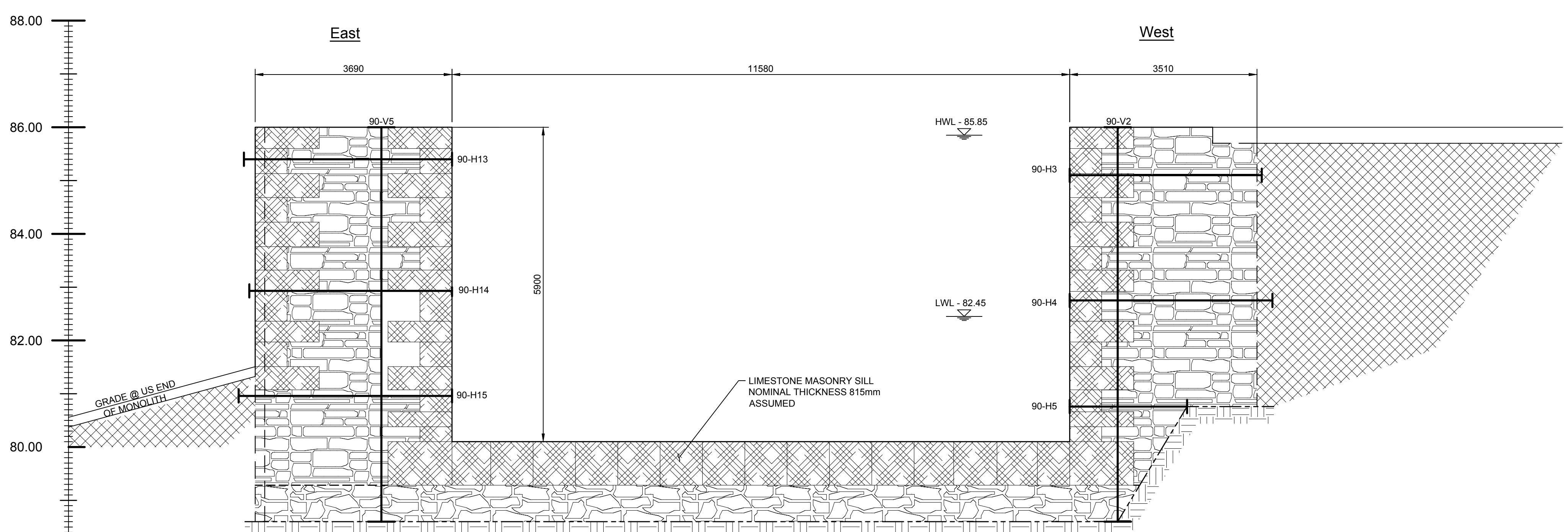
Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number / Numéro du Dessin: GR-10
Project Number / Numéro du projet: R.079796.009	Sheet 1 of 1 / Feuille 1 du 1

PWGSC–B1 (1000 x 707) PLOTTED BY: SHAWN.LAPAIN DATE PLOTTED: May 01, 2018 LAYOUT NAME: Lock 47 Section K FILE NAME: 800 CROSS SECTIONS_LOCK 47rev1.dwg

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

(long dashed line)	ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
(dashed line)	BASED ON GEOTECHINCAL INVESTIGATION
(solid line)	BASED ON VISUAL 2017 INVESTIGATION
(dash-dot line)	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
(hatch)	TOPSOIL
(hatch)	SANDY SILT / SANDY CLAY FILL
(hatch)	CONCRETE
(hatch)	FINISHED MASONRY / ASHLAR MASONRY
(hatch)	GROUTED RUBBLE MASONRY
(hatch)	GROUTED RUBBLE FILL
(hatch)	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



SECTION L / S01 — 1:50

LOCK 47 - LOWER GATE RECESS — 1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A — Detail number / Numéro du détail
B — Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

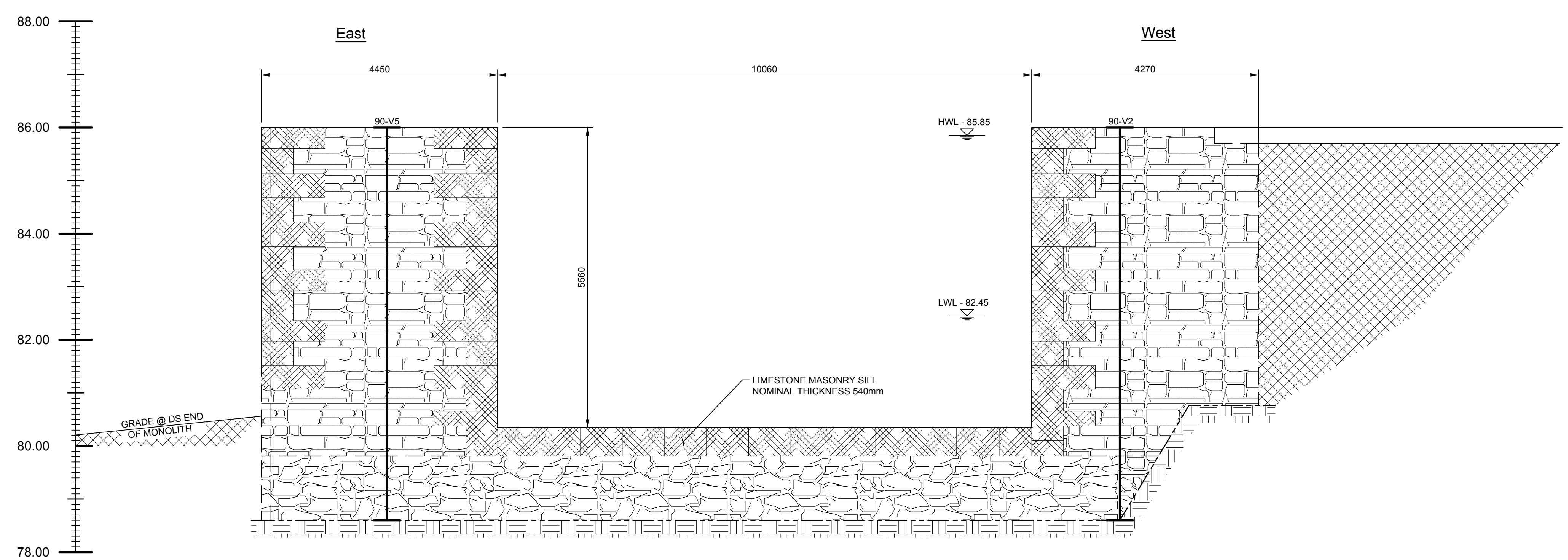
Drawing title / Titre du dessin

LOCK 47 SECTION L

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number / Numéro du Dessin: GR-11
Project Number / Numéro du projet: R.079796.009	Sheet 1 of 1 / Feuille 1 du 1

LEGEND:

	ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
	BASED ON GEOTECHINCAL INVESTIGATION
	BASED ON VISUAL 2017 INVESTIGATION
	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
	TOPSOIL
	SANDY SILT / SANDY CLAY FILL
	CONCRETE
	FINISHED MASONRY / ASHLAR MASONRY
	GROUTED RUBBLE MASONRY
	GROUTED RUBBLE FILL
	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



SECTION M / S01 — LOCK 47 - LOWER GATE SILL

1:50

Public Services and Procurement Canada / Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group / Parks Canada Infrastructure Directorate / Groupe Canaux historiques et travaux d'ingénierie / Direction de l'infrastructure de Parcs Canada

Parks Canada / Parcs Canada

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WWW.WSPGROUP.COM

No.	Description	Dwn.By Des.Par	Date

Revision / Révision

Do not scale drawings. Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

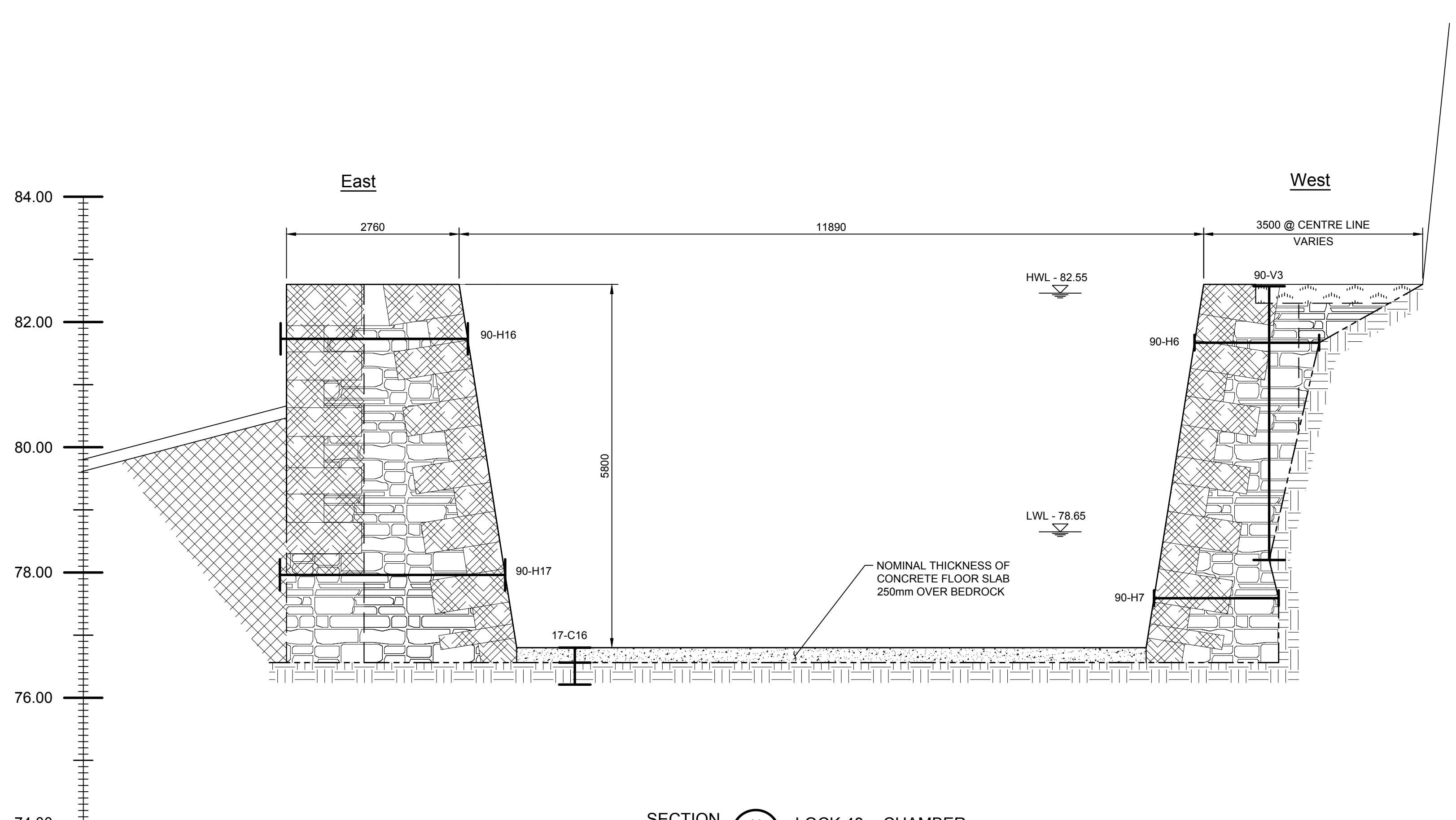
LOCK 47 SECTION M

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-12
Project Number / Numéro du projet: R.079796.009	Sheet 1 of 1 / Feuille 1 du 1

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

(long dash line)	ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
(dashed line)	BASED ON GEOTECHINCAL INVESTIGATION
(solid line)	BASED ON VISUAL 2017 INVESTIGATION
(dash-dot line)	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
(hatch)	TOPSOIL
(hatch)	SANDY SILT / SANDY CLAY FILL
(hatch)	CONCRETE
(hatch)	FINISHED MASONRY / ASHLAR MASONRY
(hatch)	GROUTED RUBBLE MASONRY
(hatch)	GROUTED RUBBLE FILL
(hatch)	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



SECTION N / S01 LOCK 48 - CHAMBER

1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

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Revision / Révision

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number
Numéro du détail
B Location dwg. number
Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

LOCK 48 SECTION N

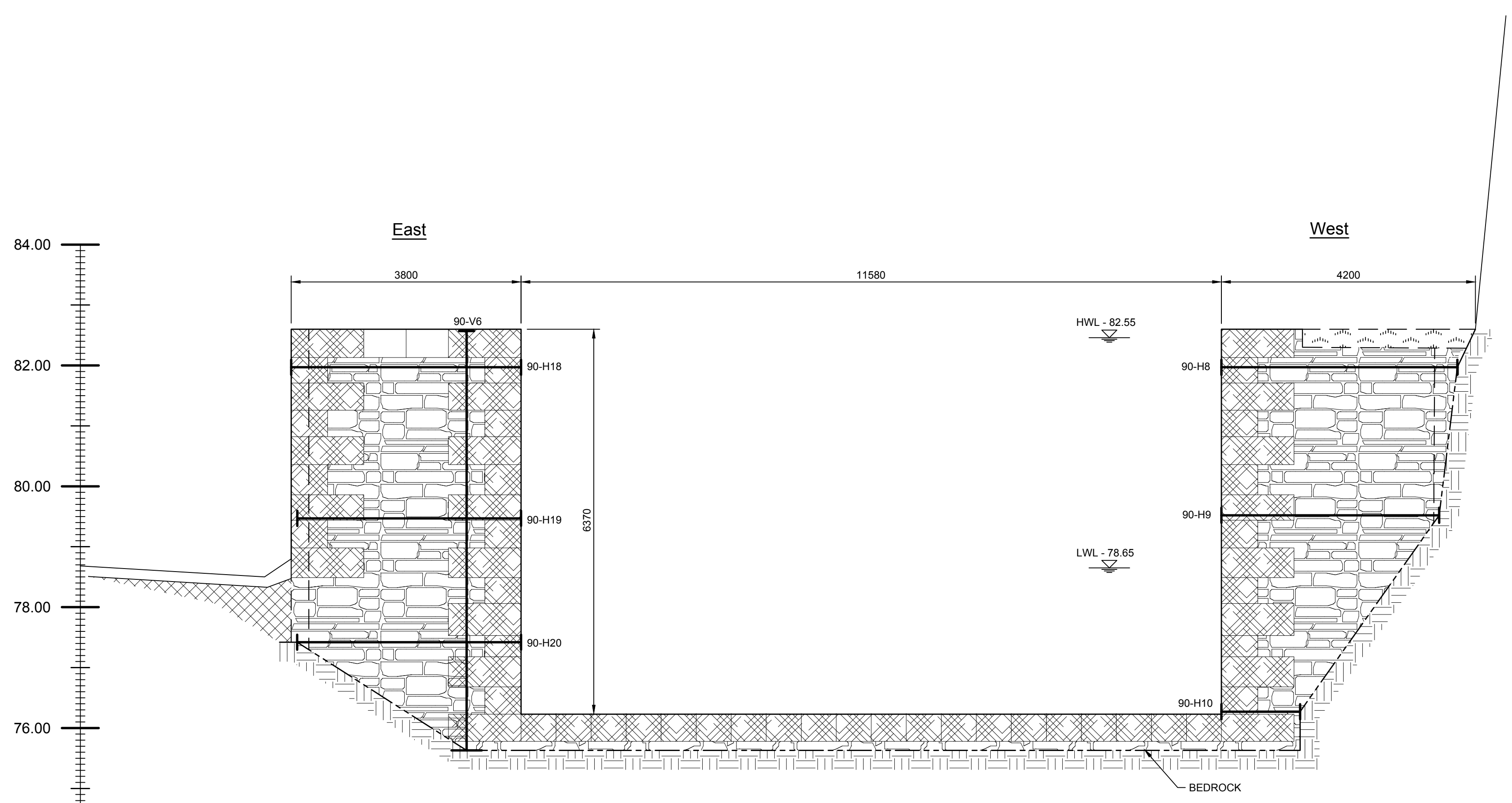
Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-13
Project Number / Numéro du projet: R.079796.009	Sheet 1 of 1 / Feuille 1 du 1

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

- ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
- BASED ON GEOTECHINCAL INVESTIGATION
- BASED ON VISUAL 2017 INVESTIGATION
- EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
- TOPSOIL
- SANDY SILT / SANDY CLAY FILL
- CONCRETE
- FINISHED MASONRY / ASHLAR MASONRY
- GROUTED RUBBLE MASONRY
- GROUTED RUBBLE FILL
- BEDROCK

NWL - XXX.XX NORMAL WATER LEVEL
WWL - XXX.XX WINTER WATER LEVEL
HWL - XXX.XX HIGH WATER LEVEL
LWL - XXX.XX LOW WATER LEVEL



SECTION P / S01 — 1:50 — LOCK 48 - GATE RECESS

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
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Revision / Révision

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A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

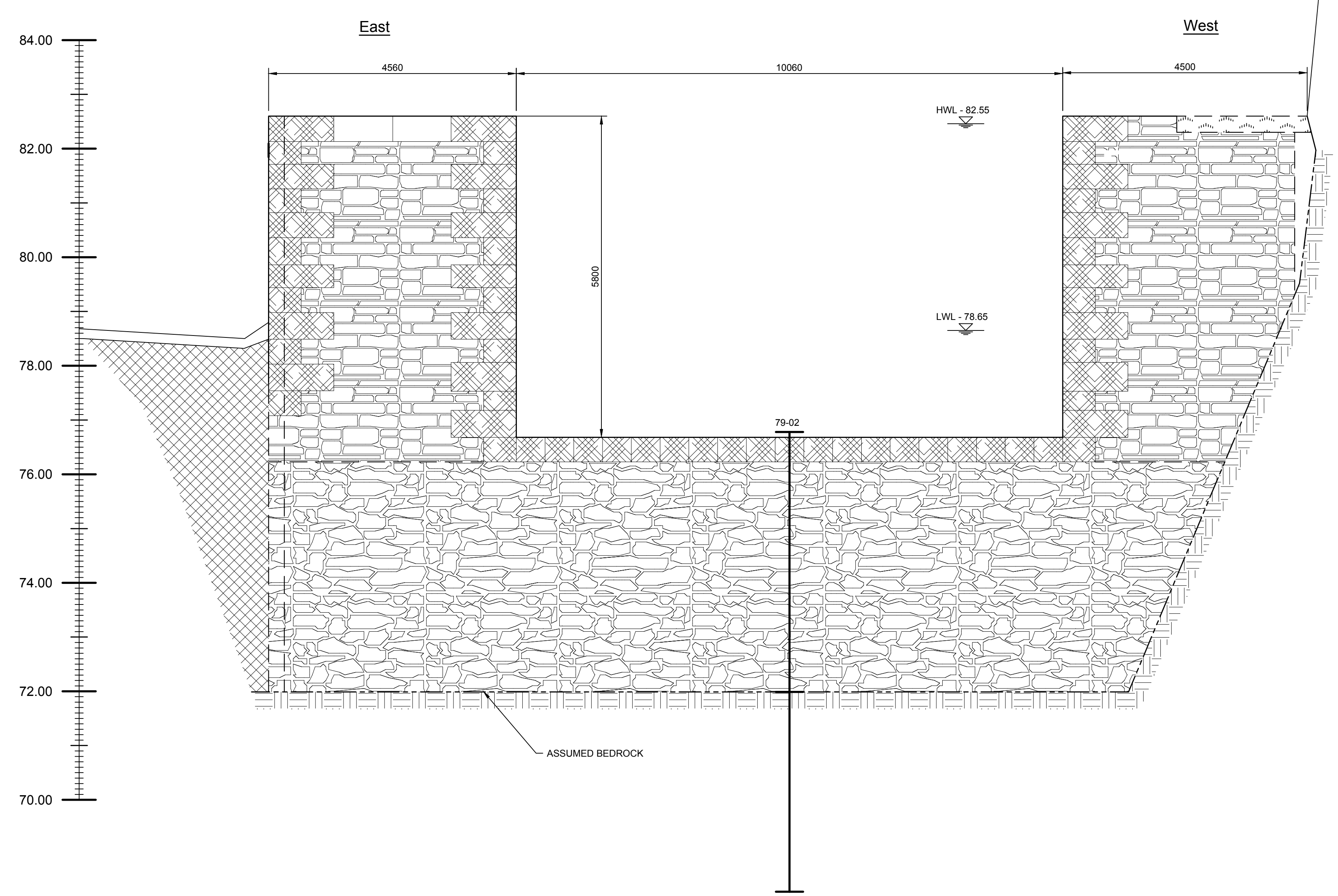
LOCK 48 SECTION P

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number / Numéro du Dessin: GR-14
Project Number / Numéro du projet: R.079796.009	Sheet 1 of 1 / Feuille 1 du 1

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

——— ———	ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
— — — —	BASED ON GEOTECHINCAL INVESTIGATION
————	BASED ON VISUAL 2017 INVESTIGATION
— - — - —	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
(hatch)	TOPSOIL
(hatch)	SANDY SILT / SANDY CLAY FILL
(hatch)	CONCRETE
(hatch)	FINISHED MASONRY / ASHLAR MASONRY
(hatch)	GROUTED RUBBLE MASONRY
(hatch)	GROUTED RUBBLE FILL
(hatch)	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



SECTION Q / S01 LOCK 48 - GATE SILL
1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada
Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

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A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

LOCK 48 SECTION Q

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-15
Project Number / Numéro du projet: R.079796.009	Sheet 1 of 1 / Feuille 1 du 1

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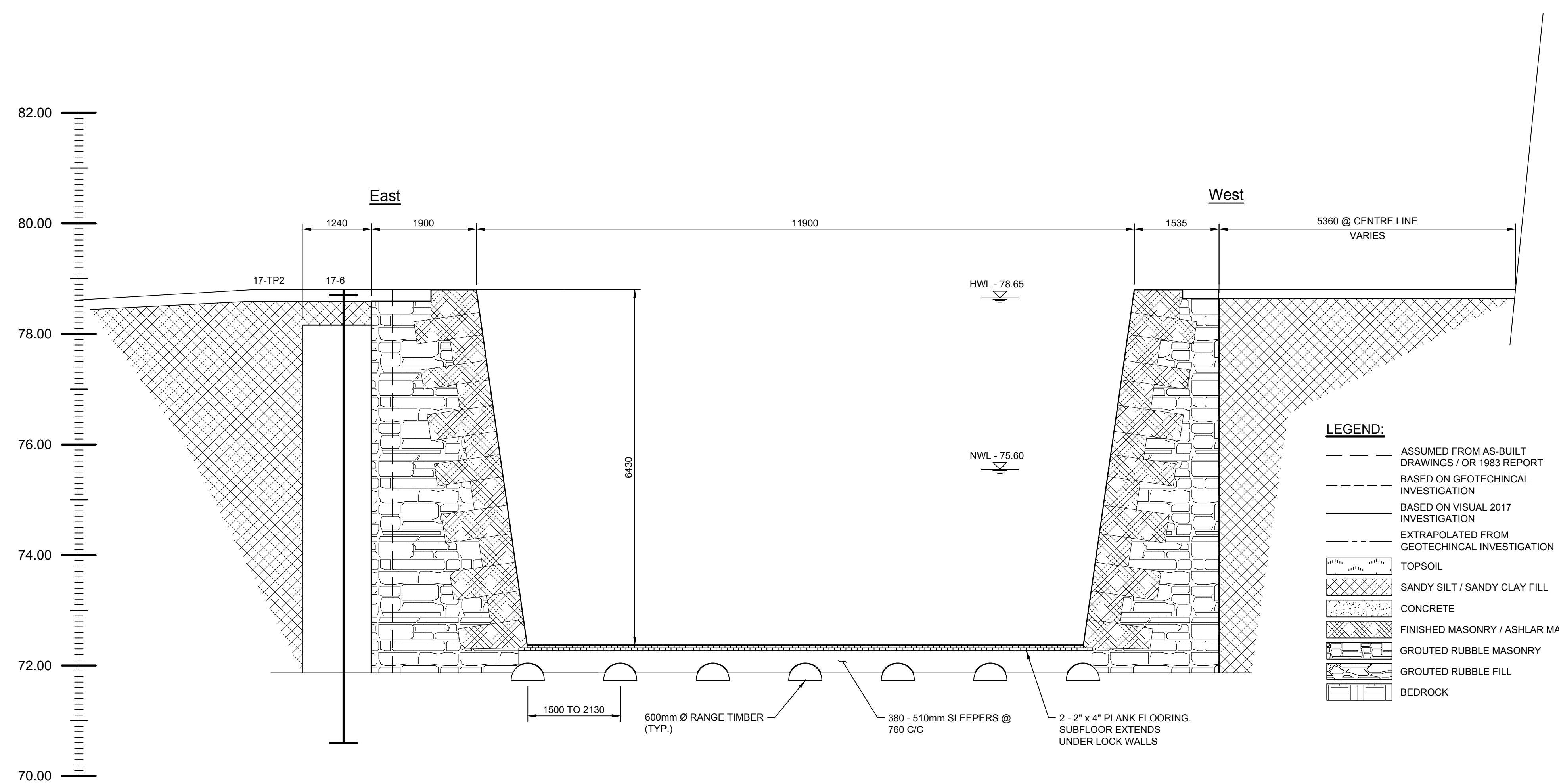
LEGEND:

— — (long dash)	ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
— — — — (dash)	BASED ON GEOTECHINCAL INVESTIGATION
——— (solid)	BASED ON VISUAL 2017 INVESTIGATION
—·—·— (dash-dot)	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
(hatch)	TOPSOIL
(hatch)	SANDY SILT / SANDY CLAY FILL
(hatch)	CONCRETE
(hatch)	FINISHED MASONRY / ASHLAR MASONRY
(hatch)	GROUTED RUBBLE MASONRY
(hatch)	GROUTED RUBBLE FILL
(hatch)	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



LEGEND:
ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
BASED ON GEOTECHINCAL INVESTIGATION
BASED ON VISUAL 2017 INVESTIGATION
EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
TOPSOIL
SANDY SILT / SANDY CLAY FILL
CONCRETE
FINISHED MASONRY / ASHLAR MASONRY
GROUTED RUBBLE MASONRY
GROUTED RUBBLE FILL
BEDROCK

SECTION (R / S01) LOCK 49 - CHAMBER
1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
Direction de l'infrastructure de Parcs Canada

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Revision / Révision

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A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet
RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION
ELGIN ONTARIO

Drawing title / Titre du dessin
LOCK 49 SECTION R

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-16
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1

PWGSC-B1 (1000 x 707) PLOTTED BY: SHAWN.LAPAIN DATE PLOTTED: May 01, 2018 LAYOUT NAME: Lock 49 Sect R FILE NAME: 800 CROSS SECTIONS_LOCK 49rev1.dwg

LEGEND:

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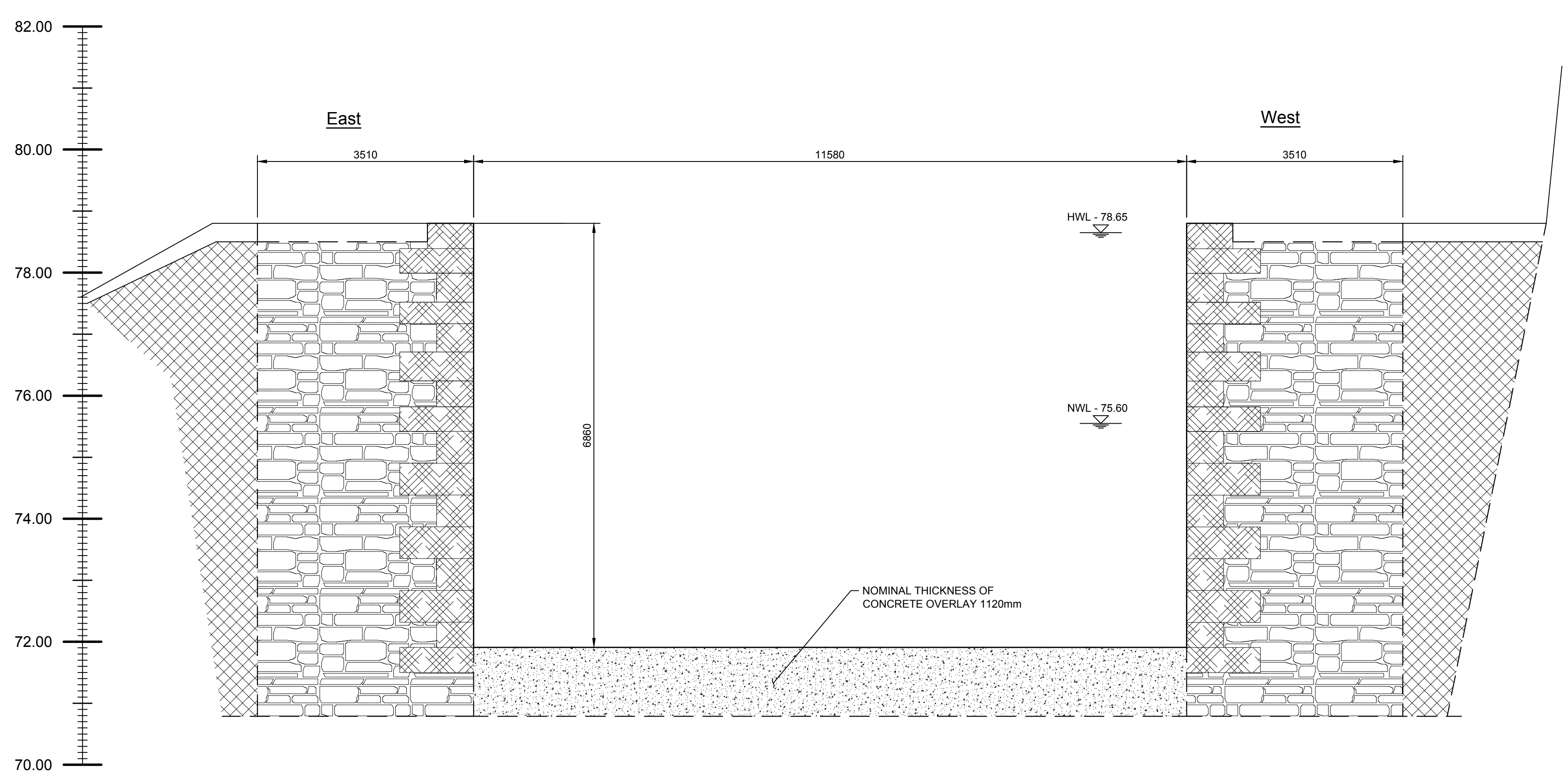
(short-dash line)	BASED ON GEOTECHINCAL INVESTIGATION
(solid line)	BASED ON VISUAL 2017 INVESTIGATION
(dash-dot line)	EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
(hatch)	TOPSOIL
(hatch)	SANDY SILT / SANDY CLAY FILL
(hatch)	CONCRETE
(hatch)	FINISHED MASONRY / ASHLAR MASONRY
(hatch)	GROUTED RUBBLE MASONRY
(hatch)	GROUTED RUBBLE FILL
(hatch)	BEDROCK
NWL - XXX.XX	NORMAL WATER LEVEL
WWL - XXX.XX	WINTER WATER LEVEL
HWL - XXX.XX	HIGH WATER LEVEL
LWL - XXX.XX	LOW WATER LEVEL



SECTION S / S01 LOCK 49 - GATE RECESS

1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the Departmental Representative of all discrepancies.

A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

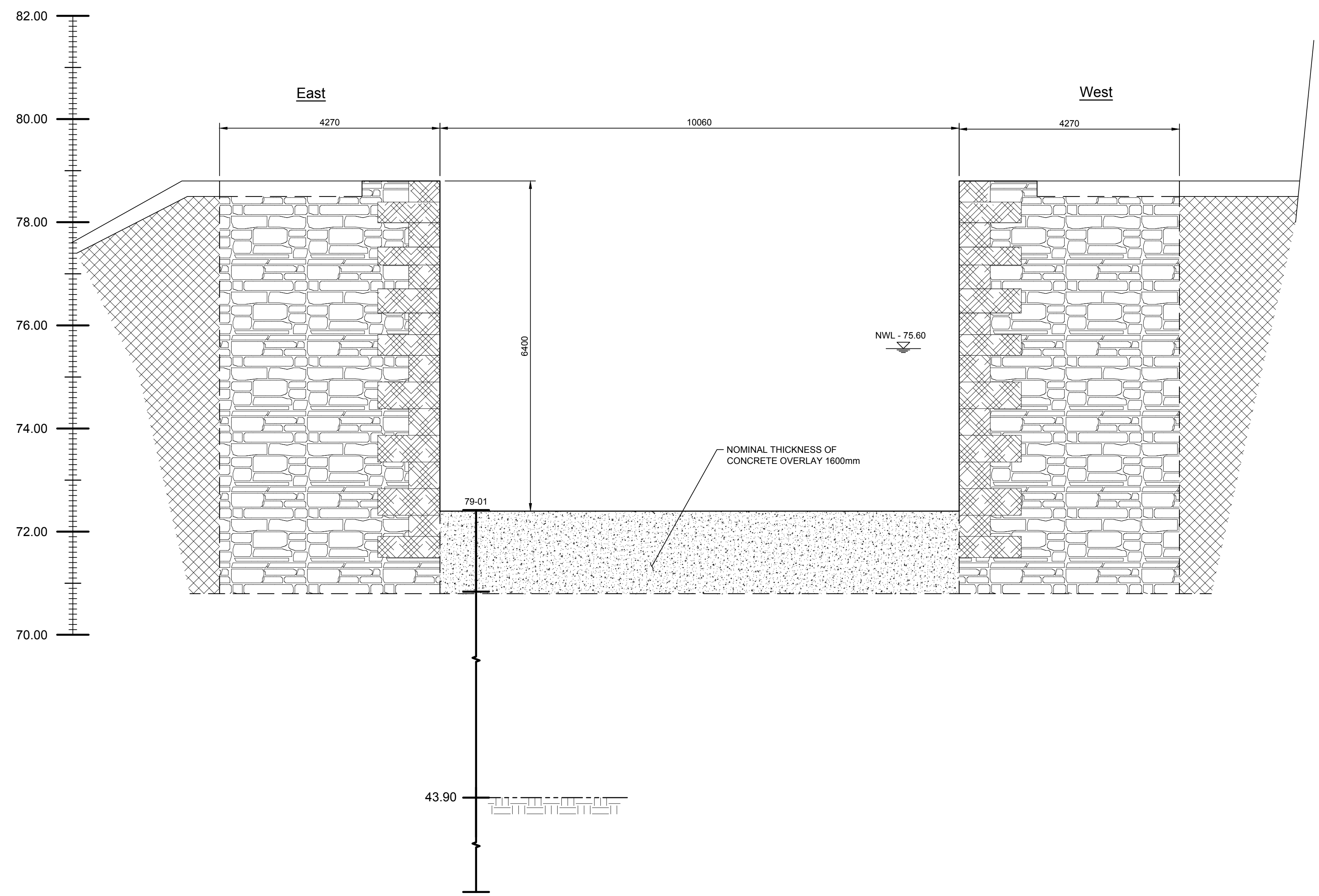
LOCK 49 SECTION S

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-17
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND:

- ASSUMED FROM AS-BUILT DRAWINGS / OR 1983 REPORT
- BASED ON GEOTECHINCAL INVESTIGATION
- BASED ON VISUAL 2017 INVESTIGATION
- EXTRAPOLATED FROM GEOTECHINCAL INVESTIGATION
- TOPSOIL
- SANDY SILT / SANDY CLAY FILL
- CONCRETE
- FINISHED MASONRY / ASHLAR MASONRY
- GROUTED RUBBLE MASONRY
- GROUTED RUBBLE FILL
- BEDROCK
- NWL - XXX.XX NORMAL WATER LEVEL
- WWL - XXX.XX WINTER WATER LEVEL
- HWL - XXX.XX HIGH WATER LEVEL
- LWL - XXX.XX LOW WATER LEVEL



SECTION T / S01 1:50 — LOCK 49 - GATE SILL 1:50

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
Groupe Canaux historiques et travaux d'ingénierie
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Revision / Révision

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A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

LOCK 49 SECTION T

Drawn by / Dessiné par: BN/AM/JH	Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON	Drawing Date / Date du dessin: MARCH 2018
Project manager / Administrateur de projet: SHAWN FILION	Drawing Number/ Numéro du Dessin: GR-18
Project Number / Numéro du projet: R.079796.009	Sheet / Feuille 1 of / du 1

APPENDIX

F

BEDROCK LOCATION

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

Historical Soils Information

1977 Site Investigation Services Vertical Borehole
2000 Quontacon Associates Boreholes
2005 Jacques Whitford Vertical Boreholes
2015 Golder Associates Vertical Boreholes and Test pits

2017 WSP Investigation

2017 WSP Vertical Borehole
2017 WSP Test Pit

ELEVATIONS LOCK 46:

UPSTREAM GATE RECESS	86.02
UPSTREAM SILL	86.56
CHAMBER INVERT	83.81
DOWNSTREAM GATE RECESS	83.45
DOWNSTREAM SILL	83.91
COPING ELEVATION EAST	90.50
COPING ELEVATION WEST	90.50

LEGEND:

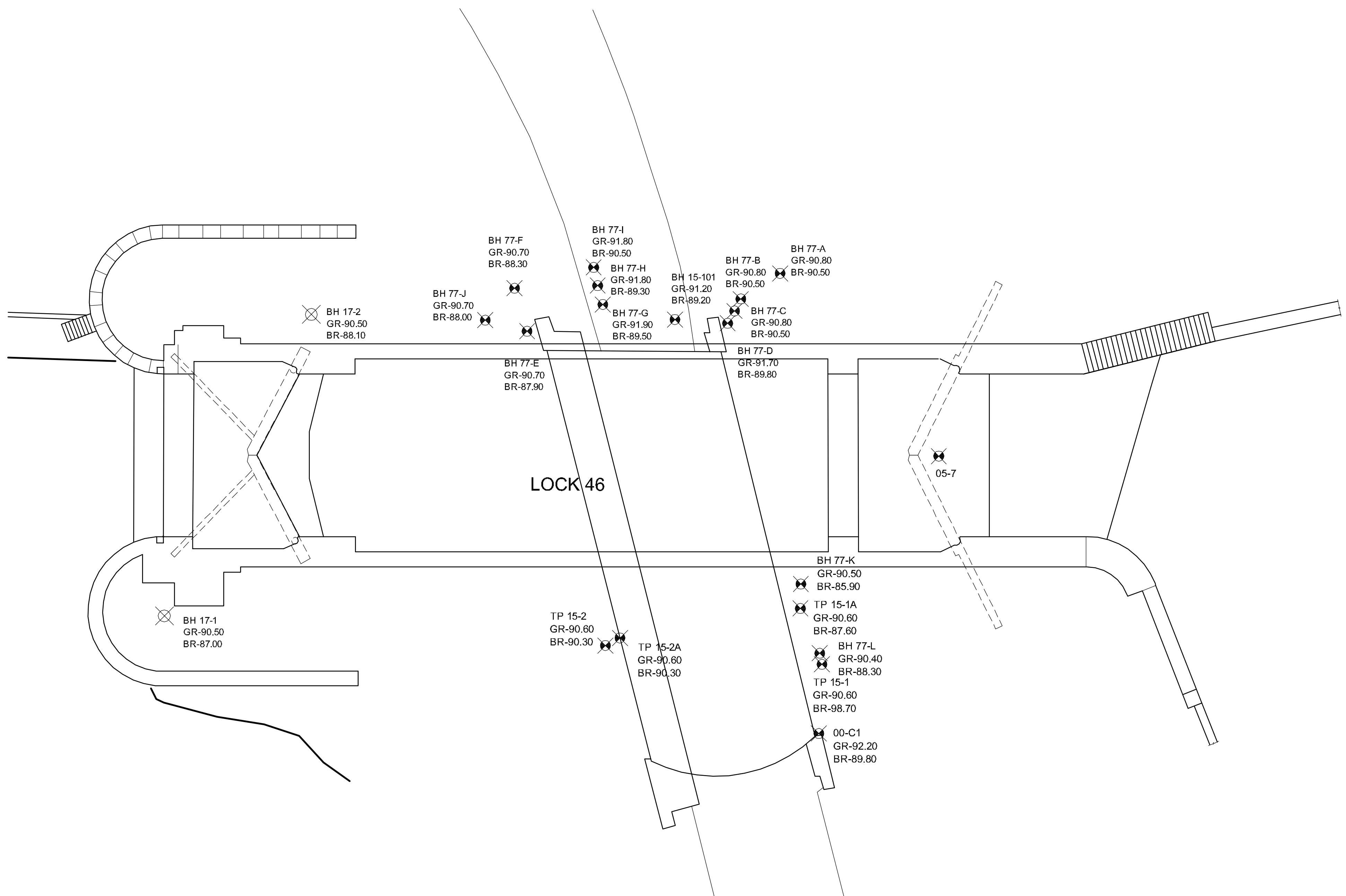
BH BOREHOLE
GR GRADE
BR BEDROCK

BH XX-X
BOREHOLE ID NUMBER
YEAR OF STUDY
BORE HOLE

TP XX-X
BOREHOLE ID NUMBER
YEAR OF STUDY
TEST PIT

NOTE:

ALL LOCATIONS ARE APPROXIMATE.



PLAN
1:150

Public Services and Procurement Canada
Services publics et Approvisionnement Canada
Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate
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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

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A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet
RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION
ELGIN ONTARIO

Drawing title / Titre du dessin
LOCK 46
BEDROCK ELEVATION
PLAN

Drawn by / Dessiné par: BN/AM/JH
Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON
Drawing Date / Date du dessin: DECEMBER 2017
Project manager / Administrateur de projet: SOMEONE NEW
Drawing Number / Numéro du Dessin: GR-02
Project Number / Numéro du projet: R.079796.006
Sheet / Feuille 2 of / du 3

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

Historical Soils Information

1979 Golder Associates Borehole
1990 Trow Associates Vertical Borehole

2017 WSP Investigation

2017 WSP Borehole and Vertical Core

ELEVATIONS LOCK 47:

UPSTREAM GATE RECESS	83.44
UPSTREAM SILL	83.86
CHAMBER INVERT	80.38
DOWNSTREAM GATE RECESS	80.10
DOWNSTREAM SILL	80.35
COPING	86.00

ELEVATIONS LOCK 48:

CHAMBER INVERT	74.02
DOWNSTREAM GATE RECESS	76.23
DOWNSTREAM SILL	76.80
COPING	82.60

ELEVATIONS LOCK 49

CHAMBER INVERT	72.35
DOWNSTREAM GATE RECESS	71.94
DOWNSTREAM SILL	72.40
COPING	78.80

LEGEND:

BH BOREHOLE
GR GRADE
BR BEDROCK

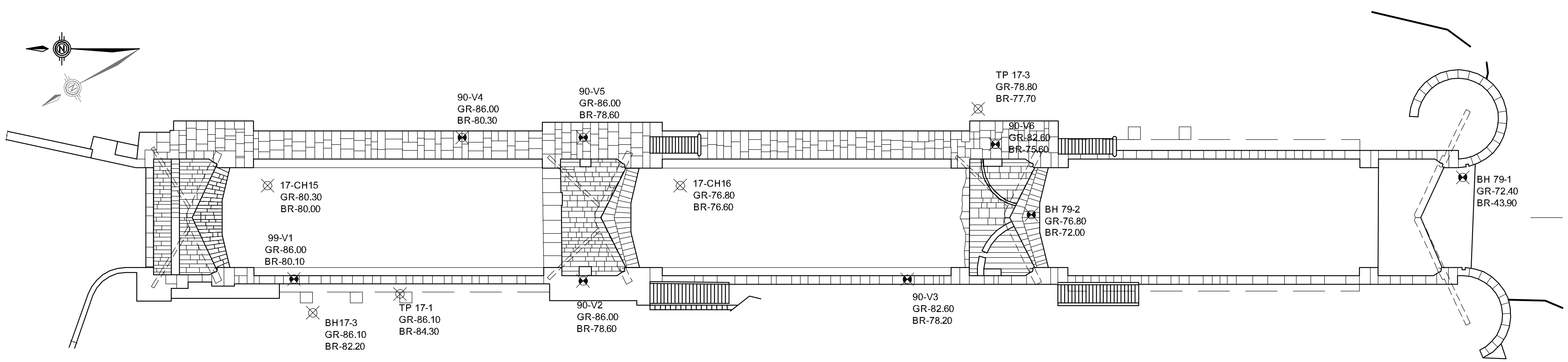
NOTE:

ALL LOCATIONS ARE APPROXIMATE.

BH XX-X
BOREHOLE ID NUMBER
YEAR OF STUDY
BORE HOLE

TP XX-X
BOREHOLE ID NUMBER
YEAR OF STUDY
TEST PIT

XX - XXX
CORE ID
YEAR OF STUDY



PLAN
1:250

Public Services and Procurement Canada
Services publics et Approvisionnement Canada

Heritage Canals and Engineering Works Group
Parks Canada Infrastructure Directorate

Groupe Canaux historiques et travaux d'ingénierie
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No.	Description	Dwn.By Des.Par	Date

Revision / Révision

Do not scale drawings.
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A Detail number / Numéro du détail
B Location dwg. number / Numéro sur dessin

Project title / Titre du projet

RIDEAU CANAL
NATIONAL HISTORIC SITE
KINGSTON MILLS LOCKSTATION
LOCKS 46 – 49
REHABILITATION

ELGIN ONTARIO

Drawing title / Titre du dessin

LOCK 47, 48, & 49
BEDROCK ELEVATION
PLAN

Drawn by / Dessiné par: BN/AM/JH
Designed by / Conçu par: JENNIFER HUNTLEY
Approved by / Approuvé par: JULIA MARSON
Drawing Date / Date du dessin: DECEMBER 2017
Project manager / Administrateur de projet: SOMEONE NEW
Drawing Number / Numéro du Dessin: GR-02
Project Number / Numéro du projet: R.079796.006
Sheet / Feuille 3 of / du 3

APPENDIX

G SUMMARY TABLE

Kingston Mills Lock Station Rehabilitation
Locks 46
Kingston Mills, Ontario

171-02359-00
Simplified Soil Strata

Borehole No.	Simplified Stratigraphy (Depth)										
(Elevation)	**Asphalt**	**Conc**	**Topsoil**	**Structure**	**Fill (Rubble)**	**Fill (Soil)**	**Fill (Rock)**	**Till**	**Auger Refusal**	**Bedrock (Cored)**	**Inferred Bedrock**
77-A (30.9)			0 - 250 mm						250 mm		250 mm
77-B (30.8)			0 - 330 mm						330 mm		330 mm
77-C (30.4)			0 - 300 mm			300 mm - 760 mm			760 mm		760 mm
77-D (30.1)			0 - 300 mm			300 mm - 910 mm			910 mm		910 mm
77-E (28.3)			0 - 250 mm					250 mm - 2.7 m	2.7 m		2.7 m
77-F (28.7)			0 - 230 mm					230 mm - 2.3 m	2.3 m		2.3 m
77-G (91.9)	0 - 100 mm					100 mm - 1.5 m	1.5 m - 2.4 m		2.4 m		
77-H (91.8)	0 - 100 mm					100 mm - 1.2 m	1.2 m - 1.4 m		1.4 m		
77-I (91.8)	0 - 100 mm					100 mm - 1.3 m					
77-J (90.7)			0 - 150 mm			150 mm - 1.8 m		1.8 m - 2.7 m		2.7 m - 4.3 m	
77-K (90.5)			0 - 100 mm			100 mm - 4.9 m			4.9 m		4.9 m
77-L (90.4)			0 - 100 mm			100 mm - 2.0 m			2.0 m		2.0 m
00-C1 (92.1)		0 - 1.5 m		1.5 m - 2.4 m						2.4 m - 3.0 m	
00-C2 (92.1)		0 - 3.8 m									
00-C3 (91.9)	0 - 50 mm	50 mm - 300 mm									
00-C4 (91.9)	0 - 38 mm	38 mm - 400 mm									
00-C5 (90.4)		0 - 1.7 m						1.7 m - 2.3 m			
00-C6 (90.4)		0 - 1.4 m						1.4 m - 1.9 m			
00-C7 (N/A)											
00-C8 (N/A)											
05-1 (85.9)		0 - 690 mm		*690 mm - 1.7 m*[1]							
05-2 (N/A)											
05-3 (86.8)		0 - 760 mm		*760 mm - 1.7 m*[1]							
05-4 (N/A)											
05-5 (N/A)											
05-6 (N/A)											
05-7 (N/A)											
05-8 (N/A)											
05-9 (N/A)											
05-10 (N/A)											
05-11 (N/A)											
05-9 (89.7)		0 - 790 mm		*790 mm - 1.6 m*[1]							

Kingston Mills Lock Station Rehabilitation
Locks 46
Kingston Mills, Ontario

171-02359-00
Simplified Soil Strata

Borehole No.	Simplified Stratigraphy (Depth)										
(Elevation)	Asphalt	Conc	Topsoil	Structure	Fill (Rubble)	Fill (Soil)	Fill (Rock)	Till	Auger Refusal	Bedrock (Cored)	Inferred Bedrock
05-11 (86.0)				0 - 410 mm	410 mm - 860 mm[1]						
15-TP1 (N/A)											
15-TP1A (90.6)			0 - 30 mm			30 mm - 3.0 m			3.0 m		
15-TP2 (N/A)											
15-TP2A (90.6)			0 - 30 mm			30 mm - 1.4 m			1.4 m		
15-101 (91.9)	0 - 100 mm					100 mm - 2.7 m				2.7 m - 5.1 m	
15-202 (86.6)				0 - 660 mm	660 mm - 2.1 m					2.1 m - 3.4 m	
15-203 (86.6)				0 - 310 mm	310 mm - 1.6 m					1.6 m - 2.6 m	
15-204 (86.6)				0 - 510 mm	510 mm - 2.8 m					2.8 m - 4.0 m	
17- CH1				0 - 490 mm[1]							
17- CH2				0 - 470 mm[1]							
17- CH3 (Turning Basin)				0 - 410 mm[1]							
BH 17-1 (89.8)			0 - 70 mm			70 mm - 3.5 m				3.5 m -3.6 m	
BH 17-2 (89.8)			0 - 90 mm			90 mm - 2.4 m				2.4 m - 4.0 m	

1 - Did not penetrate wall

Kingston Mills Lock Station Rehabilitation
Locks 47
Kingston Mills, Ontario

171-02359-00
Simplified Soil Strata

Borehole No. (Elevation)	Simplified Stratigraphy (Depth)			
	Topsoil	Structure	Fill (Soil)	Bedrock (Cored)
90-H1 (84.9)		0 - 3.0 m	3.0 m - 3.2 m	
90-H2 (81.3)		0 - 1.9 m		1.9 m - 3.0 m
90-H3 (85.2)		0 - 3.6 m	3.6 m - 3.7 m	
90-H4 (82.8)		0 - 3.8 m	3.8 m - 3.9 m	
90-H5 (80.5)		0 - 2.2 m	2.2 m - 3.8 m	
90-H13 (85.3)		0 - 3.9 m		
90-H14 (82.9)		0 - 3.8 m		
90-H15 (79.9)		0 - 3.8 m	3.8 - 4.2 m	
90-V1 (86.0)		0 - 5.9 m		5.9 m - 7.5 m
90-V4 (86.0)		0 - 5.7 m		5.7 m - 7.1 m
90-V5 (86.0)		0 - 7.4 m		7.4 m - 8.9 m
17- CH4		0 - 470 mm[1]		
17- CH5		0 - 480 mm[1]		
17- CH6		0 - 500 mm[1]		
17- CH13		0 - 480 mm[1]		
17- CH15		0 - 370 mm[1]		
17- CH17		0 - 370 mm[1]		
17- CH19		0 - 440 mm[1]		
17- CH20		0 - 480 mm[1]		
17- CH23		0 - 460 mm[1]		
17- CH24		0 - 410 mm[1]		
17- CH32		0 - 430 mm[1]		
BH 17-3 (86.1)	0 - 80 mm		80 mm - 2.3 m	2.3 m - 3.3 m

1 - Did not penetrate wall

Kingston Mills Lock Station Rehabilitation
Locks 48
Kingston Mills, Ontario

171-02359-00
Simplified Soil Strata

Borehole No.	Simplified Stratigraphy (Depth)				
(Elevation)	Topsoil	Structure	Fill (Soil)	Auger Refusal	Bedrock (Cored)
90-H6 (81.7)		0 - 2.0 m			
90-H7 (77.6)		0 - 2.0 m			2.0 m - 3.0 m
90-H8 (82.01)		0 - 3.9 m			3.9 m - 4.2 m
90-H9 (79.6)		0 - 3.6 m			3.6 m - 4.2 m
90-H10 (76.3)		0 - 1.3 m			
90-H16 (81.7)		0 - 3.0 m			
90-H17 (72.6)		0 - 3.6 m			3.6 m - 3.8 m
90-H18 (82.0)		0 - 3.8 m			
90-H19 (79.4)		0 - 3.8 m			
90-H20 (77.0)		0 - 3.7 m			
90-V2 (N/A)					
90-V3 (82.6)		0 - 4.4 m			4.4 m - 6.7 m
90-V5 (N/A)					
90-V6 (82.6)		0 - 7.0 m			7.0 m - 8.6 m
17- CH7		0 - 480 mm[1]			
17- CH8		0 - 480 mm[1]			
17- CH9		0 - 480 mm[1]			
17- CH10		0 - 480 mm[1]			
17- CH11		0 - 440 mm[1]			
17- CH12		0 - 480 mm[1]			
17- CH14	0 - 240 mm	240 - 590 mm[1]			
17- CH16		0 - 540 mm[1]			
17- CH18		0 - 540 mm[1]			
17- CH21		0 - 470 mm[1]			
17- CH22		0 - 480 mm[1]			
17- CH25		0 - 400 mm[1]			
17- CH26		0 - 430 mm[1]			
17- CH27		0 - 500 mm[1]			
HA 17-4 (82.6)			0 - 160 mm	160 mm	
TP 17-3 (78.8)	0 - 160 mm		160 mm - 1.6 m		

1 - Did not penetrate wall

Kingston Mills Lock Station Rehabilitation
Locks 49
Kingston Mills, Ontario

171-02359-00
Simplified Soil Strata

Borehole No. (Elevation)	Simplified Stratigraphy (Depth)									
	Conc	Topsoil	Fill (Rubble)	Fill (Soil)	Clay	Silt	Sand	Till	Auger Refusal	Bedrock (Cored)
79-01 (72.4)	0 - 1.6 m		1.6 m - 1.9 m	1.9 m - 3.6 m	3.6 m - 6.4 m	6.4 m - 10.4 m	10.4 m - 28.8 m			28.8 m - 32.8 m
79-02 (76.8)				0 - 4.8 m	4.8 m - 8.5 m					
17- CH28 (N/A)				0 - 460 mm[1]						
17- CH29 (N/A)				0 - 460 mm[1]						
17- CH30 (N/A)				0 - 460 mm[1]						
17- CH31 (N/A)				0 - 460 mm[1]						
17- CH33 (N/A)				0 - 460 mm[1]						
HA 17-5 (78.8)				0 - 100 mm					100 mm	
BH 17-6 (78.8)			0 - 100 mm	100 mm - 6.1 m				6.1 - 8.2 m		
TP 17-2 (78.7)		0 - 170 mm		170 mm - 1.1 m						
TP 17-3 (78.8)			0 - 160 mm				160 mm - 1.6 m			
TP 17-4 (N/A) - Structural Hole										
TP 17-5 (N/A) - Structural Hole										

1 - Did not penetrate wall

APPENDIX

H LIMITATIONS AND CONDITIONS

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to SPL Consultants Limited at the time of preparation. Unless otherwise agreed in writing by SPL Consultants Limited, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. SPL Consultants Limited accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.