APPENDIX B

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED BRIDGE REPLACEMENT
HASTINGS SWING BRIDGE
TRENT – SEVERN CANAL
TORONTO, ONTARIO

Prepared for:

ASSOCIATED ENGINEERING

By:

SPL CONSULTANTS LIMITED

Project: 1842-910
October 1, 2014

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

351 Steelcase Road West, Unit 10-12
Markham, Ontario L3R 3H9
Tel: 905.475.0065 Fax: 905.475.0045

Table of Contents

1. INTRODUCTION

SPL Consultants Limited (SPL) was retained by Associated Engineering (AE) to undertake a geotechnical investigation for the proposed replacement of the existing swing bridge located on Trent-Severn Canal in Hastings, Ontario.

Based on the conceptual design information provided by AE, it is understood that the existing superstructure of the existing swing bridge will be removed and replaced with a new bridge superstructure; the existing pier and abutments will remain in place.

The purpose of the geotechnical investigation was to obtain subsurface soil and groundwater information at the site by means of a limited number of exploratory coreholes. Based on our interpretation of the corehole data, this report presents the findings of the investigation and provides comments and recommendations related to the design of the proposed bridge replacement.

This report deals with geotechnical issues only. The Terms of Reference (TOR) for this investigation are outlined in SPL's Proposal No. P-13.05.120 dated June 5, 2013 and the subsequent project correspondence.

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 the applicable codes and standards. Once the detail design is available, or if there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the design. It may then be necessary to carry out additional borings and reporting before the recommendations of this office can be relied upon.

The site investigation and recommendations follow generally accepted practice for geotechnical consultants in Ontario. The format and contents are guided by client specific needs and economics and do not conform to generalized standards for services. Laboratory testing for most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report has been prepared for AE. Third party use of this report without SPL consent is prohibited. The limitation conditions presented in Section "General Comments and Limitations of Report" of this report form an integral part of the report and they must be considered in conjunction with this report.

2. BACKGROUND INFORMATION

Hastings Swing Bridge is located at Lock 18 of Trent-Severn Canal, in Hastings, Ontario. The swing bridge was constructed in 1952 and is now classified as "Other" under the category of Cultural Resources, requiring no specific historical rehabilitation. The swing bridge is a deck plate girder bridge with a combined steel grate and asphalt covered concrete deck as shown in Figures 6A to 6N of Golder's report attached in Appendix E. The bridge has an overall length of approximately 25 metres and an overall width of approximately 8 metres (from centre line to centre line of the outer most girders). Hastings Swing Bridge is supported by an off centre concrete pivot pier and concrete abutments.

The swing span is an unequal arm type pivoting above a centre pintle with balance wheels. The swing bridge is supported by an off-center concrete pivot pier and concrete abutments. The off-centre pivot pier is located on the north side of the Trent-Severn Canal. The bridge is not currently posted for maximum load. As noted above, the two-lane bridge is located on Bridge Street South and taking a very high traffic volume as part of Highway 45. The deck serves vehicular as well as bicycle traffic. A steel grate pedestrian walkway is mounted on the south side of the bridge, outside the south girder.

The Trent-Severn Canal is operational from April to October and open to the public for navigational purposes from the Friday before the May long weekend until the Wednesday following the Canadian Thanksgiving long weekend. It is understood that the swing bridge is swung away from canal five to eight times a day during the operational season to allow boats to pass the lock.

It is understood that the dead load of the super-structure of the swing bridge is solely supported by the combination of the central pintle, the pillow block rests and balance wheels. The swing bridge is swung by two hydraulic jacks built underneath the bridge deck within the circular track on top of the central concrete pier. The south end of the swing bridge deck is supported by two wheels sitting on two steel plates. The north end of the swing bridge deck is supported by two hydraulic jacks. During the non-operational seasons, the north and south ends of the swing bridge deck are supported by steel posts placed underneath the deck between two hydraulic jacks at the north end and two wheels at the south end. It is anticipated that the loading at the south and north ends are generally light (i.e. primarily live loading from traffic).

It is further understood that the last emergency repair was carried in December of 2010. The bridge deck and slab repair and the hydraulic system upgrades were carried out in 1996 to 1997. The pillow block rests have had shims removed to accommodate the settlement of the bridge (i.e. the settlement of the central pintle was assumed); the last adjustment was carried out in 1992.

Localized repairs to the concrete surface of the existing pier and abutments were also noted, however, the history of these repairs were not available at the time of preparing the report.

3. REVIOUS INVESTIGATIONS

A previous geotechnical investigation report entitled “Geotechnical Evaluation, Hastings Swing Bridge, Trent-Severn Canal, Hastings, Ontario, Report Number 11-1184-0022” dated November 21, 2011 carried out by Golder Associates Ltd. (Golder) was provided by AE to SPL and attached in Appendix E of this report.

The results of the previous Golder’s report have been reviewed and referenced in this report.

4. INVESTIGATION PROCEDURE

The field work for this investigation was carried out on October 1 and 2, 2013, during which time 4 coreholes were advanced at the locations shown on the Corehole Location Plan, Drawing 1A. Coreholes CH13-01, 13-2 and CH13-4 were cored through existing pier concrete, highly weathered rock and continuously cored to depths ranging from 2.9 m to 3.5 m below pier concrete surface into slightly weathered to fresh limestone bedrock using diamond coring equipment. Corehole 13-3 was cored though the existing concrete slab between the pier and abutment footing to a depth of 2.4 m below slab surface.

The field work for this investigation was observed by members of our engineering staff who arranged underground service locates, logged the subsurface conditions encountered in the coreholes and cared for the samples obtained.

All of the soil samples and concrete and rock core samples from coreholes were visually examined in the laboratory by project engineer. Selected one soil sample was subjected to grain size analyses and the results of which are presented in Drawing 7.

Unconfined Compressive Strength testing (UCS) was carried out on two concrete core samples and the results are presented in Appendix C, attached to this report.

Selected one rock core sample was shipped to Queen's University for Unconfined Compressive Strength testing (UCS) and the results are presented in Appendix D, attached to this report.

Shallow groundwater conditions were noted in the open coreholes during drilling. All of the coreholes were backfilled and sealed with pre-mix concrete upon completion of drilling.

5. REGIONAL GEOLOGY

The site is located within the physiographic region known as the Peterborough Drumlin Field (Chapman, L.J. and Putnam, D.F. "The Physiography of Southern Ontario", 3rd Edition, 1984). This region is lying north of the Oak Ridges Moraines with a rolling till plains with numerous drumlins. For most of the part, the bedrock underlying this region is limestone of the Lindsay and Verulam Formations which are somewhat softer and less massive formations than the Gull River formation. They are also highly fossiliferous and disintegrate easily. The beds slope slightly towards the southwest and the edges overlapping strata face north. Based on the findings in this geotechnical investigation, the soil and bedrock conditions are generally consistent with the Regional Geology.

6. SITE AND SUBSURFACE CONDITIONS

The swing bridge is located on Bridge Street South in Hastings, Ontario and as part of Highway 45, experiencing high traffic volumes.

It is understood that the replacement of the existing bridge has been considered based on previous investigation and evaluation.

The corehole locations are shown on Drawing 1A. The subsurface conditions in coreholes (CH13-01 to CH13-04) are presented in the individual borehole logs (Drawing Nos. 3 to 6 inclusive). The generalized sub-surface profile is presented on Drawings 1B and 1C. The following is a summarized account of the subsurface conditions encountered in the coreholes drilled during this investigation, followed by more detailed descriptions of the major soil strata and shallow groundwater conditions.

Pier Concrete (CH 13-1, 13-2 and 13-4) and slab Concrete (CH 13-3)

Concrete was encountered in all boreholes. The thicknesses of the concrete of the existing pier ranged from approximately 970 mm to 1850 mm at the corehole locations as measured in the Coreholes CH 13-1, 13-2 and 13-4. The thickness of the concrete of the concrete slab south of the existing abutment was approximately 230 mm at the corehole location as measured in the Corehole CH 13-3.

The condition of the concrete was observed at core locations. The inside of the coreholes were examined carefully and photographed for cracks and the condition of the concrete. A review of concrete cores did not reveal any defects on concrete cores. Refer to the photos taken inside of the coreholes attached in Appendix A and photos of concrete and rock cores attached in Appendix B.

Full depth cores identified 970 mm to 1850 mm thickness for the concrete within the circular concrete pier and 230 mm thickness of the concrete slab north of the concrete pier. No rebar was found in the concrete cores.

Cores from corehole CH13-1 at a depth of 0.15 m to 0.35 below ground surface and corehole 13-2 at a depth of 0.5 m to 0.8 m below ground surface were tested for compressive strength of the hardened concrete in accordance with CSA A23.2-09-14C. The compressive strengths of the hardened concrete for these cores were 37 MPa and 35 Mpa, with an average compressive strength of 36 MPa. The results of the compressive strength testing are attached in Appendix C.

Fill Materials

Fill materials were encountered below the concrete slab in Corehole CH 13-3 and extended to a depth of 2.1 m below ground surface. The fill materials generally consisted of rock fragments with clayey silt.

Clayey Silt

Thin layers/zones of clayey silt soil were encountered within the weathered limestone bedrock.

Grain size analyses of one sample (CH13-4/SA1) were conducted and the results are presented in Drawing 7 as well as shown on the corehole log with the following fractions:

Grain Size Distribution

Borehole No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
CH13-04	1	5	10	51	34

Bedrock

Based on the results of rock coring, the bedrock at the site generally consists of highly weathered to fresh, grey to dark grey, fine grained fossiliferous limestone of the Lindsay and Verulam Formations. This bedrock was confirmed by coring in Coreholes CH 13-1 to 13-4. The surface elevation of the bedrock is variable at the corehole locations, as shown on the cross section drawing, Drawings 1B and 1C.

The Total Core Recovery (TCR) of the core samples ranged from 58 percent to 100 percent; the Solid Core Recovery (SCR) ranged from 15 percent to 93 percent; and the Rock Quality Designation (RQD) ranged from 0 percent to 100 percent. The RQD values for the bedrock cores sampled immediately below the concrete structures were generally 0 percent. Based on these results and on our visual examination of the core samples, the rock quality of the limestone encountered is generally considered to be very poor immediately below the concrete structures becoming poor to excellent with depth.

One rock core sample from Corehole CH 13-1 was prepared and subjected to compressive strength testing. This testing was carried out in general accordance with ASTM Standard Test Method D 7012-07, entitled “Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens”. This testing gave an unconfined compressive strength value of 42 MPa, indicating that the strength of this bedrock is classified as medium strong (Canadian Foundation Engineering Manual, 2006, 4th Edition, Table 3.5).

Shallow Groundwater

The water levels encountered upon completion of coring were at depths ranging from 1.0 m to 2.2 m below ground surface in the coreholes. The measured groundwater tables in the coreholes upon completion of coring and one day after the coring are summarized in the following table:

Groundwater Levels Observed in Coreholes

Corehole	Date of Observation	Water Level Depth (m) below ground surface	Note
CH13-01	October 1, 2013	1.0	The water level in the canal was approximately at the same elevation.
	October 2, 2013	1.0	
CH13-02	October 1, 2013	1.2	The water levels measured may be affected by the water used for coring and/or the water in the canal
	October 2, 2013	1.2	
CH13-03	October 1, 2013	1.0	
	October 2, 2013	1.0	
CH13-04	October 1, 2013	-	
	October 2, 2013	2.2	

It is considered that the stable groundwater levels at the bridge site would be affected by the water level in the canal and the local prevailing water levels. It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to major weather events.

7. GEOTECHNICAL INTERPRETATION AND RECOMMENDATIONS

In this section, the subsurface conditions are interpreted as they relate to the design and construction of the proposed bridge replacement. Comments relating to construction methods are intended for the guidance of the designer (AE) to establish constructability only.

The construction methods described in this report must not be misconstrued as being specifications or direct recommendations to the contractors, or as being the only suitable methods. Prospective contractors should evaluate all of the factual information, obtain additional subsurface information as they might deem necessary and should select their construction methods, sequence and equipment based on their own experience in similar ground and groundwater conditions. Readers of this report are also reminded that the conditions are known only at the borehole locations and in view of the generally wide spacing of the coreholes, conditions may vary significantly between boreholes.

7.1 Summary of Previous Geotechnical Evaluation carried out by Golder

Based on the results of the previous geotechnical investigation carried out by Golder, no voids or fractures of the concrete were observed at two corehole locations (Coreholes 2 and 3) on the existing pier; no cracks are visible on the exposed vertical face on the west side of the central pier. No obvious voids were detected during the coring based on coring reaction within the bedrock at two coreholes within the pier; however, the quality of the upper portion of the limestone, immediately below the concrete pier, was very poor with R.Q.D. measurements of 0 percent at both locations. Relative higher quality bedrock was encountered at a depth of about 2.0 m below the ground surface, which is generally consistent with the sound bedrock encountered in the test pit (i.e. 1.8 m below ground surface). The very poor quality of the bedrock at the founding level of the central pier could be a result of the weathering and deterioration of bedrock after the completion of the construction, or it could indicate that the weathered/fractured bedrock was not removed at the time of the construction.

As referenced to Golder's previous geotechnical report, it is understood that no noticeable lateral movement of the abutment wall towards the bridge deck and no noticeable settlement of the abutment footing have been reported; The structural loading at the north abutment is relatively light and no significant cracks were observed at the support jacks and at the locations of the steel posts; The global stability analysis of the north abutment indicates a factor of safety greater than the typical minimum requirement of 1.5; The horizontal cracks at northeast corner of the north abutment noted in previous Golder's report were recommended be adequately sealed to prevent further deterioration of the concrete from exposure.

7.2 Discussion and Recommendations

It is understood that the existing superstructure of the swing bridge will be replaced and the existing central pier and both abutments will remain in place. The proposed new superstructure of the swing bridge will be constructed off the bridge site and be shipped to the site upon completion. The superstructure of the existing bridge will be lifted and removed. The new superstructure will be craned and placed on the existing pier after appropriate repair/rehabilitation being carried out to the existing pier and abutments. It is understood that the new superstructure would be the same weight or slightly heavier than the existing superstructure. The recommendations provided in the report must be further reviewed by this office should the new superstructure is greatly heavier than the existing one.

The construction history, design drawings and as-built drawings of the existing swing bridge are not currently available. The design details of the central pintle and the existing pier are not available however, it is understood that, the dead load of the bridge structure is solely supported by the central pier.

It is understood that shims for the pillow block rests were removed about 22 years ago to accommodate the settlement which was reported to occur at the central pintle location. No record of the above noted settlement and repair has been provided for review and the nature and cause of the settlement reported at the central pintle location is unknown. No further settlement has been reported and no further adjustment of the pillow block rests have been carried out since that time. The design and as-built information for the pintle, the loading distribution for the pintle and the as-built steel reinforcement of the exiting pier are not available. Based on the visual observation and measurements, the pintle is a round steel plate with an approximate diameter of 0.8 m bolted on top of a hexagon shape concrete platform which is slightly elevated above the concrete pier. No cracking was observed on the concrete surrounding the steel plate and in the hexagon shaped concrete platform. However, this concrete appeared to have been placed during relatively recent repair works and cracks that may be present in the original concrete would be obscured by this newer concrete.

Based on the results of the current investigation carried out by SPL and previous investigation carried by Golder, there is no noticeable cracking or facture being observed inside of the coreholes within the existing concrete pier, which is generally consistent with the observation of the concrete cores. The quality of the bedrock immediately below the existing concrete pier is very poor with noticeable voids and layers of clayey silt. The very poor quality of the bedrock at the founding levels could be a result of the weathering and deterioration of bedrock after the completion of the construction, or it could indicate that the weathered/fractured bedrock was not removed at the time of the construction. Based on the results of current investigation, the groundwater flow within the weathered rock zones could be another important factor of the ongoing erosion, which is reducing the rock quality continuously.

The water levels observed in the coreholes are generally at the same depth of the water in the canal immediately south of the concrete pier. Based on the recharge rate of the water in the coreholes when the water was pumped out of holes upon completion of the coring, the water in the coreholes is very likely hydraulically connected to the water in the canal. It should be noted that the water in the canal was lowered to the bottom of the canal during the previous investigation and there was no water was observed in the previous coreholes and boreholes and only minor water seepage was noted in the previous test pit.

The thickness of the highly weathered rock zone varied significantly from one location to another with an approximately range of 0.3 m to 1.0 m as shown on the Drawings 1B and 1C. It should be noted that the thick weathered zone is encountered at the upstream direction of the river (west portion of the concrete pier), which may be an indication of weathering and deterioration caused by the groundwater flow.

The voids in the weathered rock zones are considered to be obvious as shown in the photos taken inside the coreholes. However, it should be noted the voids observed inside of the coreholes were affected by the coring operation and may appear to be more severe. As noted above, the thickness of the weathered rock zones varied significantly between corehole locations. The weathered rock zone at the east portion of the existing concrete pier is very thin (i.e. approximately 0.3 m) while the weathered rock zone is up to a 1.0 m at the west portion of the existing pier. In consideration of the observed voids and potential ongoing erosion caused by the groundwater flow, the potential of excessive settlement of the existing pier should be considered as part of the bridge replacement design. Due to the variation of the weathered rock zone between the east portion and west portion of the concrete pier, the settlement could occur in a form of differential settlement. Therefore, it is recommended that measures such as grouting be considered for the existing pier to minimize/reduce the potential of further erosion of the existing founding materials.

It should be noted that the purposes of the grouting should be only for filling/sealing the voids. High pressure grouting is not recommended at this site due to the potential concern of adverse impaction to the existing wall of the canal and existing concrete pier. Low pressure close spacing grouting (such as polyurethane foam injection grouting or the equivalents) may be considered. Quick cure grouting should be considered in order to reduce the time of the bridge closure. Should it be practical, the grouting may be carried out prior to the bridge replacement to reduce the time of the bridge closure. In addition, consideration should be given to the following items for the grouting work:

- The water table in the canal should be lowered to the canal bottom or the water in the canal be diverted to lower the groundwater table in the grouting zones prior to the grouting.
- The concrete pier should be monitored for any movement by a geotechnical engineer from SPL, especially upward movement during and after the grouting; A detailed settlement monitoring plan would be provided once the grouting measures are determined;
- Additional coring should be considered after the grouting as part of the quality control/assurance;
- The selected grouting contractor should submit a work plan for review by the project engineer and geotechnical engineer prior to grouting;
- Sufficient protection should be provided to the canal in case the grout materials may come through the canal wall into the water in the canal;
- The zone of the grouting should be from the bottom of the concrete pier to the sound limestone bedrock.

The vertical loading of the existing abutment is considered to be light (primarily live traffic loading in addition to the weight of the abutment wall and backfill soils). Complete grouting of the weather rock zones below the abutment footing may not be considered to be necessary. The founding soil on the north side of the abutment wall may not be reachable by grouting equipment from the south side of the abutment wall. However, as reported in Golder's report, void seems to be present at the rock surface immediately below the abutment footing in the vicinity of Golder's Corehole 1. Should it be required, consideration may be given to grouting/filling the voids immediately below the abutment concrete footing on the south side of the abutment wall. Extending the grout to the sound bedrock is considered to be not necessary. As noted in Golder's previous investigation, only one drainage hole was noted on the existing abutment all. Complete grouting/sealing of the weathered rock zone may prevent the potential drainage through bottom of the abutment footing, which would potentially lead to an increased hydrostatic pressure behind the abutment wall. Should grouting be applied to the abutment footing, the relevant items listed above should be applied accordingly.

7.4 Existing South Abutment and Canal Concrete Walls

The geotechnical investigation and evaluation of the south abutment and canal walls are not within the scope of the work for this current assignment. However, from visual observations, the abutment footing and canal walls appear to be severely deteriorated and should be repaired as required.

8. GENERAL COMMENTS AND LIMITATIONS OF REPORT

SPL Consultants Limited 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 the privilege of making this review, SPL Consultants Limited will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the guidance of design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole and test pit results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

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.

9. CLOSURE

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

Yours Truly,

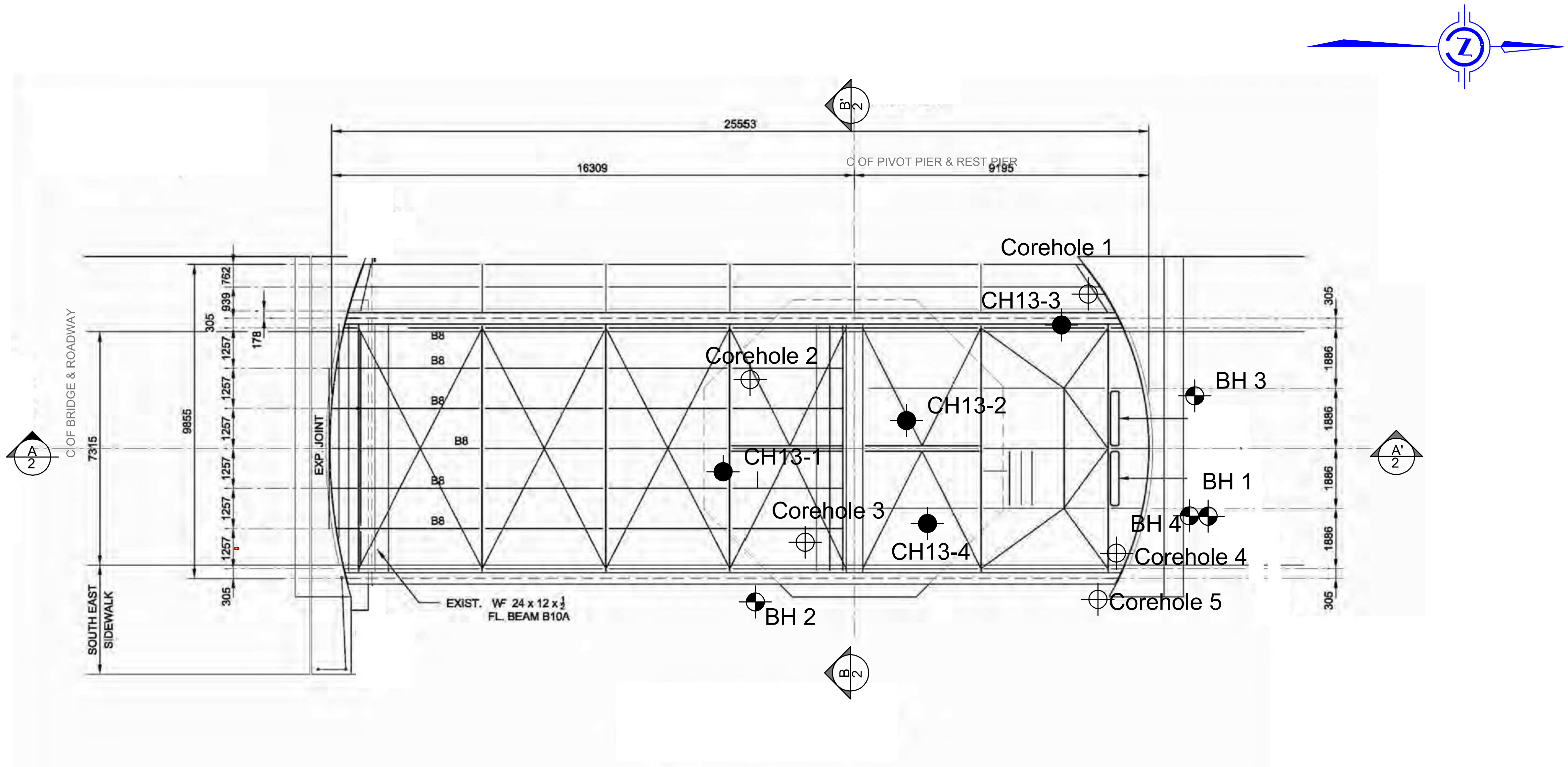
SPL CONSULTANTS LIMITED

LICENSED PROFESSIONAL ENGINEER
D. B. LIU
100107874
Oct 1/2014
PROVINCE OF ONTARIO

David B. Liu, P.Eng.
Senior Geotechnical Engineer

for

Fanyu Zhu, Ph.D., P.Eng.
Principal Engineer



C:\SPL\1842-910\Drawing2-Loc.dwg

CH13-1 Current Corehole locations carried out by SPL

Corehole 1 Previous Corehole locations carried out by Golder

BH1 Previous Borehole locations carried out by Golder

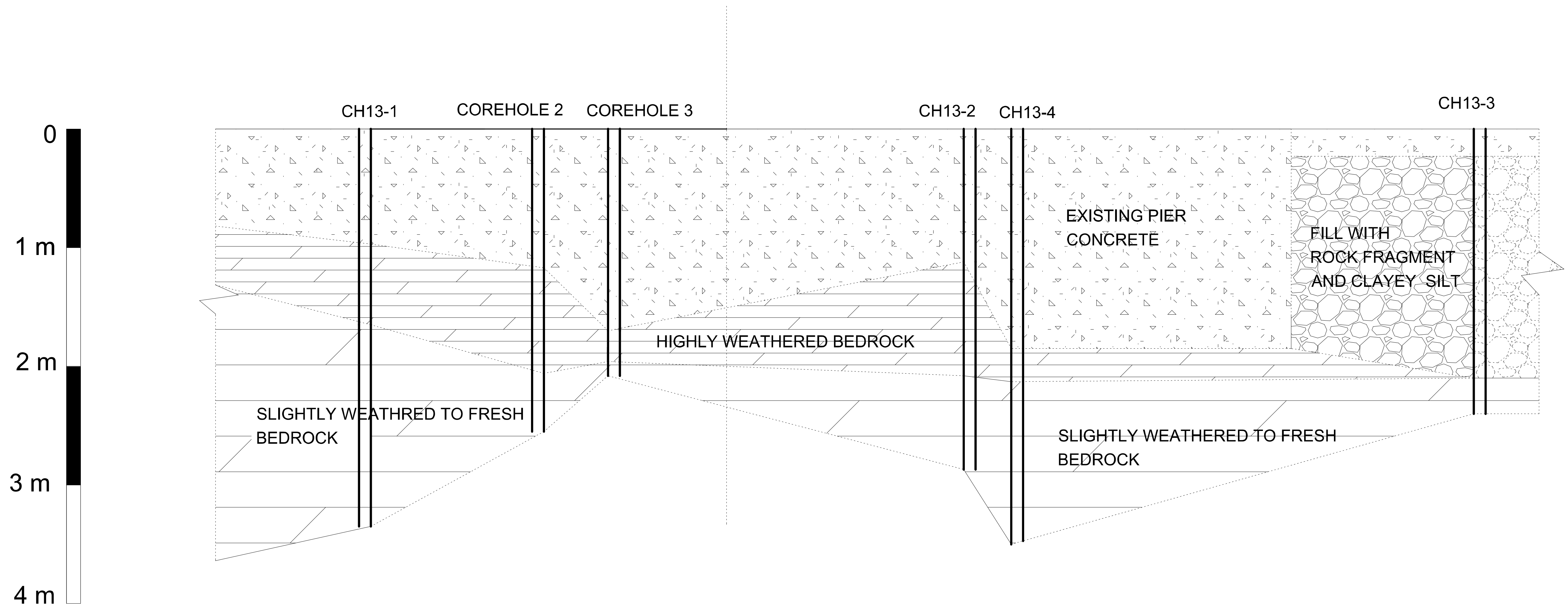
NOT TO SCALE

All locations are Approximate

Client:	Associated Engineering			Project No.: 1842-910	Drawing No.: 1A
Drawn:	LY	Approved:	DL	Title:	COREHOLE LOCATION PLAN
Date:	October 2013	Scale:	N/A	Project:	Geotechnical Investigation - Bridge Replacement Hastings Swing Bridge, Hastings, ONTARIO
Original Size:	11x17	Rev:	N/A	SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	

CROSS SECTION A-A

C/L



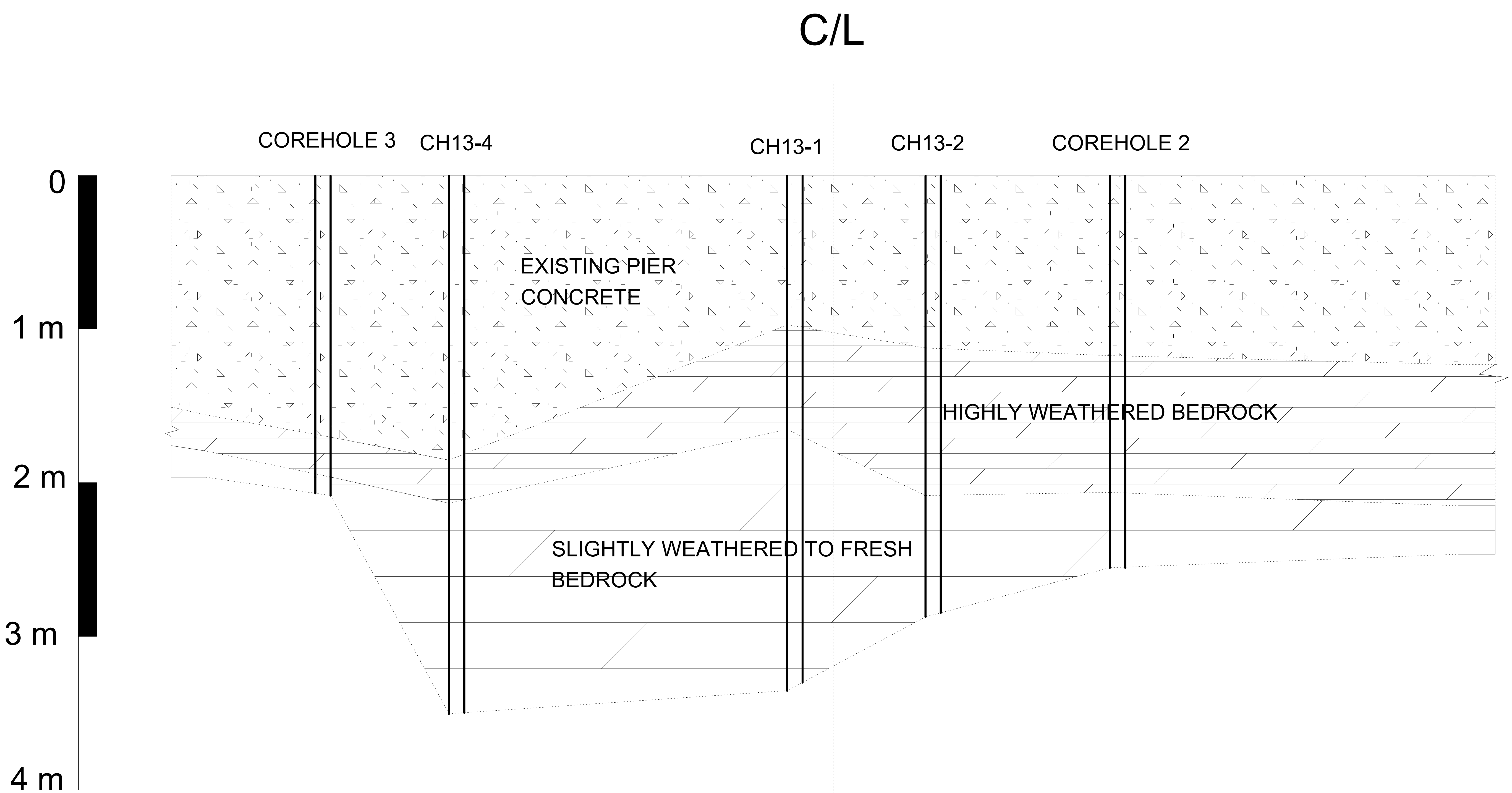
Notes:

1. Not to scale, all locations are approximate
2. The assumed boundary of the concrete footings & soil strada are based on limited corehole data and should be considered approximate

Client:	Associated Engineering			Project No.: 1842-910	Drawing No.: 1B
Drawn:	DW	Approved:	DL	Title:	CROSS SECTION A-A'
Date:	Ocotber 2013	Scale:	As shown	Project:	Geotechnical Investigation - Bridge Replacement, Hastings Swing Bridge, Hastings, Ontario
Original Size:	11X17	Rev:	N/A	SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	

C:_SPL\Hasting Bridge P13-05-120\Reporting\Haisting Bridge.dwg

CROSS SECTION B-B



Notes:

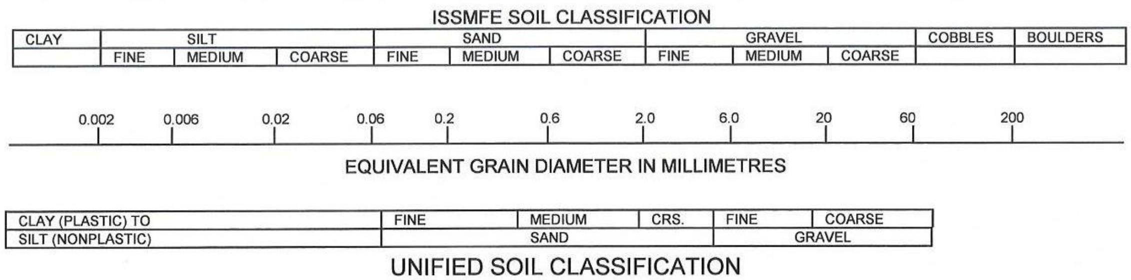
1. Not to scale, all locations are approximate
2. The assumed boundary of the concrete footings & soil strada are based on limited corehole data and should be considered pproximate

Client:	Associated Engineering		Project No.: 1842-910	Drawing No.: 1C	
Drawn: DW		Approved: DL	Title:	CROSS SECTION B-B'	
Date: Ocotber 2013		Scale: As shown	Project:	Geotechnical Investigation - Bridge Replacement, Hastings Swing Bridge, Hastings, Ontario	
Original Size: 11X17		Rev: N/A	SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology		

C:_SPL\Hasting Bridge P13-05-120\Reporting\Haisting Bridge.dwg

Drawing 2: Notes on Soil Sample Descriptions

1. All sample descriptions included in this report generally follow the Unified Soil Classification. Laboratory grain size analyses provided by SPL also follow the same system. Different classification systems may be used by others, such as the system by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Please note that, with the exception of those samples where a grain size analysis and/or Atterberg Limits testing have been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional preliminary geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

LOG OF BOREHOLE CH 13-1

PROJECT: Geotechnical Investigation - Bridge Replacement
CLIENT: Associated Engineering
PROJECT LOCATION: Hastings Swing Bridge, Hastings, Ontario
DATUM: N/A
BH LOCATION: See Corehole Location Plan

DRILLING DATA
Method: Coring
Diameter: 107 mm
Date: Oct/01/2013

REF. NO.: 1842-910
ENCL NO.: 3

(m) ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	SAMPLE TYPE	"N" BLOWS/0.3 m	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**CONCRETE:** 970mm						
1.0	**LIMESTONE:** Highly weathered, grey, highly fractured fossiliferous, layers of clayey silt	1	RC		W. L. 1.0 mBGL Oct 01, 2013		
		2	RC				
1.7	**LIMESTONE:** Slightly weathered to fresh, grey to dark grey, fossiliferous.	3	RC				
		4	RC				
		5	RC				
3.4	**END OF BOREHOLE** Note: 1) Water level at 1.0m upon completion. 2) Refer to rock core log.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED; + FIELD VANE & Sensitivity; ● QUICK TRIAXIAL; × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L
WATER CONTENT (%): 10 20 30
POCKET PEN. (Cu) (kPa)
NATURAL UNIT WT (kN/m^3)

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

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

SPL

LOG OF ROCK CORE CH 13-1

PROJECT: Geotechnical Investigation - Bridge Replacement
CLIENT: Associated Engineering
LOCATION: Hastings Swing Bridge, Hastings, Ontario
DATUM: N/A
BH LOCATION: See Corehole Location Plan

DRILLING DATA
Method: Coring
Diameter: 107 mm
Date: Oct/01/2013

REF. NO.: 1842-910
ENCL NO.: 3

(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE NUMBER	CORE SAMPLE SIZE	TOTAL CORE RECOVERY (%)	SOLID CORE RECOVERY (%)	HARD LAYER (%)	RQD (%)	FRACTURE INDEX (per 0.3 m)	DISCONTINUITIES	Weathering Index	HYDRAULIC CONDUCTIVITY (cm/sec)	POINT LOAD TEST UCS AXIAL (MPa)*	POINT LOAD TEST UCS DIAMETRAL (MPa)*	UNIAXIAL COMPRESSION (MPa)	DENSITY (g/cm³) E (GPa)
	Rock Surface															
1.0	**LIMESTONE:** Highly weathered, grey, highly fractured fossiliferous, layers of clayey silt	W. L. 1.0 mBGL Oct 01, 2013			100	54		0	14	Weathered Fracture: 0.81m-1.12m						
1.3			2	93mm	100	67		50	11	Highly weathered Soft layer (highly weathered): 1.30m-1.40m						
1.7	**LIMESTONE:** Slightly weathered to fresh, grey to dark grey, fossiliferous.															
1.8			3	93mm	80	41		23	3	Fresh						
2.3			4	93mm	100	32		32	1 12	Fresh Fracture: 3.07m-3.20m						
2.8			5	93mm	100	45		40	1	Fresh						
3.4	End of Corehole															

Weathering Index: W1-Fresh, W2-Slightly weathered, W3-Moderately weathered, W4-Highly weathered, W5-Completely weathered θ = angle to the core axis

E = Modulus of Elasticity
*: UCS [Mpa] ≈ 24 $I_{S(50)}$

LOG OF BOREHOLE CH 13-2

PROJECT: Geotechnical Investigation - Bridge Replacement	**DRILLING DATA**
CLIENT: Associated Engineering	Method: Coring
PROJECT LOCATION: Hastings Swing Bridge, Hastings, Ontario	Diameter: 107 mm — REF. NO.: 1842-910
DATUM: N/A	Date: Oct/01/2013 — ENCL NO.: 4
BH LOCATION: See Corehole Location Plan	

(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 20 40 60 80 100	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m^3)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**CONCRETE:** 1120mm											
1.1	**LIMESTONE:** Highly weathered, grey to dark grey, highly fractured fossiliferous.		1	RC		∇	W. L. 1.2 mBGL Oct 01, 2013					
			2	RC								
2.1	**LIMESTONE:** Slightly weathered to fresh, grey to dark grey, fossiliferous.		3	RC								
			4	RC								
2.9	**END OF BOREHOLE** Note: 1) Water level at 1.2m upon completion. 2) Refer to rock core log.											

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

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

LOG OF ROCK CORE CH 13-2

PROJECT: Geotechnical Investigation - Bridge Replacement	**DRILLING DATA**	
CLIENT: Associated Engineering	Method: Coring	REF. NO.: 1842-910
LOCATION: Hastings Swing Bridge, Hastings, Ontario	Diameter: 107 mm	ENCL NO.: 4
DATUM: N/A	Date: Oct/01/2013	
BH LOCATION: See Corehole Location Plan		

(m) ELEV / DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE NUMBER	CORE SAMPLE SIZE	TOTAL CORE RECOVERY (%)	SOLID CORE RECOVERY (%)	HARD LAYER (%)	RQD (%)	FRACTURE INDEX (per 0.3 m)	DISCONTINUITIES	Weathering Index	HYDRAULIC CONDUCTIVITY (cm/sec)	POINT LOAD TEST UCS AXIAL (MPa)*	POINT LOAD TEST UCS DIAMETRAL (MPa)*	UNIAXIAL COMPRESSION (MPa)	DENSITY (g/cm^3) E (GPa)
	Rock Surface															
1.1	**LIMESTONE:** Highly weathered, grey, highly fractured fossiliferous, layers of clayey silt	W. L. 1.2 mBGL Oct 01, 2013	1	93mm	58	15		0	34 / 30	Weathered Soft layer (highly weathered): 1.55m-1.74m Hard layer: 20mm						
1.7			2	93mm	100	59		33	10	Slightly weathered Soft layers (highly weathered): 1.85mm-1.855mm; 1.91mm-2.01mm; 2.08m-2.58m Hard layer: 10mm						
2.1	**LIMESTONE:** Slightly weathered to fresh, grey to dark grey, fossiliferous.		3	93mm	100	76		50	3	Fresh Hard layer: 20mm						
2.5			4	93mm	100	100		100	1	Fresh Hard layer: 60mm						
2.9	End of Corehole															

Weathering Index: W1-Fresh, W2-Slightly weathered, W3-Moderately weathered, W4-Highly weathered, W5-Completely weathered θ = angle to the core axis

E = Modulus of Elasticity
*: UCS [Mpa] ≈ 24 $I_{S(50)}$

LOG OF BOREHOLE CH 13-3

PROJECT: Geotechnical Investigation - Bridge Replacement
CLIENT: Associated Engineering
PROJECT LOCATION: Hastings Swing Bridge, Hastings, Ontario
DATUM: N/A
BH LOCATION: See Corehole Location Plan

DRILLING DATA
Method: Coring
Diameter: 107 mm
Date: Oct/01/2013

REF. NO.: 1842-910
ENCL NO.: 5

(m) ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRATA PLOT	SAMPLES: NUMBER	SAMPLES: 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 (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**CONCRETE:** 230mm											
0.2	**FILL:** rock fragments with clayey silt					▽	W. L. 1.0 mBGL Oct 01, 2013					
2.1	Highly weathered, grey to dark grey, highly fractured fossiliferous Limestone, layers of clayey silt, T.C.R.=70%, SCR = 30% R.O.D. = 0%											
2.4	**END OF BOREHOLE** Notes: 1) Water level at 1.0m upon completion.											

GROUNDWATER ELEVATIONS
Measurement 1st ▽ 2nd ▼ 3rd ▽ 4th ▼

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

SPL

LOG OF BOREHOLE CH 13-4

PROJECT: Geotechnical Investigation - Bridge Replacement
CLIENT: Associated Engineering
PROJECT LOCATION: Hastings Swing Bridge, Hastings, Ontario
DATUM: N/A
BH LOCATION: See Corehole Location Plan

DRILLING DATA
Method: Coring
Diameter: 107 mm
Date: Oct/02/2013

REF. NO.: 1842-910
ENCL NO.: 6

SOIL PROFILE (m) ELEV DEPTH	DESCRIPTION	SAMPLES 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 (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (10 20 30)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0.0	**CONCRETE:** 1850mm										
1.9	**LIMESTONE:** Highly weathered, grey, highly fractured fossiliferous, porkets of clayey silt	1	RC								
2.1	100mm clay layer at 2.0m **LIMESTONE:** Slightly weathered to fresh, grey to dark grey, fossiliferous.	2	RC		∇	W. L. 2.2 mBGL Oct 02, 2013					
		3	RC								
		4	RC								
3.5	**END OF BOREHOLE** Notes: 1) Water level at 2.2m upon completion. 2) Refer to rock core log.										

GROUNDWATER ELEVATIONS
Measurement 1st ∇ 2nd ▼ 3rd ∇ 4th ∇

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

LOG OF ROCK CORE CH 13-4

PROJECT: Geotechnical Investigation - Bridge Replacement
CLIENT: Associated Engineering
LOCATION: Hastings Swing Bridge, Hastings, Ontario
DATUM: N/A
BH LOCATION: See Corehole Location Plan

DRILLING DATA
Method: Coring
Diameter: 107 mm
Date: Oct/02/2013

REF. NO.: 1842-910
ENCL NO.: 6

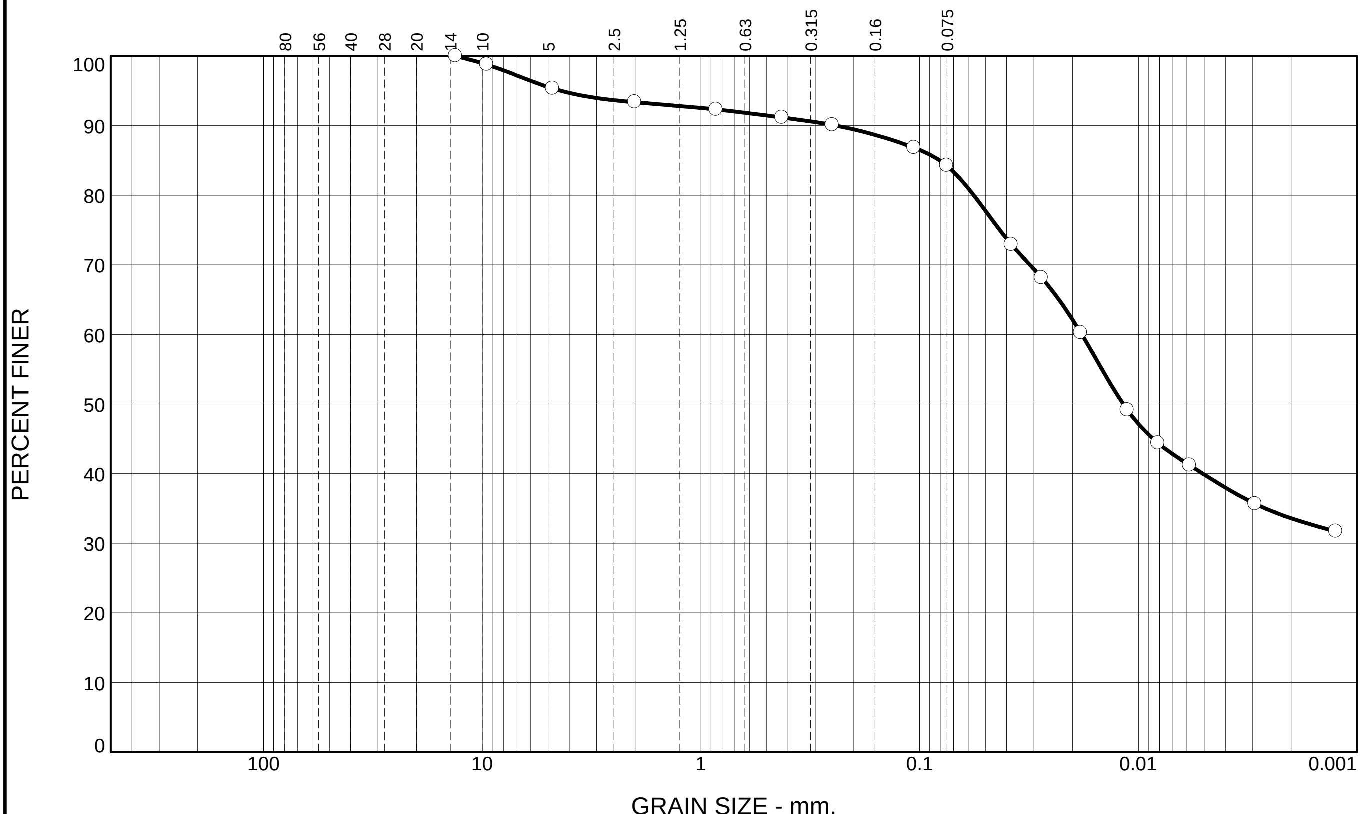
(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE NUMBER	CORE SAMPLE SIZE	TOTAL CORE RECOVERY (%)	SOLID CORE RECOVERY (%)	HARD LAYER (%)	RQD (%)	FRACTURE INDEX (per 0.3 m)	DISCONTINUITIES	Weathering Index	HYDRAULIC CONDUCTIVITY (cm/sec)	POINT LOAD TEST UCS AXIAL (MPa)*	POINT LOAD TEST UCS DIAMETRAL (MPa)*	UNIAXIAL COMPRESSION (MPa)	DENSITY (g/cm^3) E (GPa)
	Rock Surface															
1.9	**LIMESTONE:** Highly weathered, grey, highly fractured fossiliferous, pockets of clayey silt *(continued)*		1	93mm	90	20		0	12	Slightly weathered 100mm Clay layer at 2.03m						
2.1	**LIMESTONE:** Slightly weathered to fresh, grey to dark grey, fossiliferous.	∇ W. L. 2.2 mBGL Oct 02, 2013	2	93mm	92	73		35	3							
2.8			3	93mm	84	53		26	4							
3.3			4	93mm	100	100		100								
3.5	End of Corehole															

Weathering Index: W1-Fresh, W2-Slightly weathered, W3-Moderately weathered, W4-Highly weathered, W5-Completely weathered θ = angle to the core axis

E = Modulus of Elasticity
*: UCS [Mpa] ≈ 24 $I_{S(50)}$

SPL ROCK CORE-2014 1882-910 '2013'10'28.GPJ SPL.GDT 10/2/14

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	4.7	1.9	2.2	7.0	50.6	33.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
13.2mm	100.0		
9.5mm	98.8		
4.75mm	95.3		
2.00mm	93.4		
0.850mm	92.3		
0.425mm	91.2		
0.250mm	90.1		
0.106mm	86.8		
0.075mm	84.2		
0.0380 mm.	72.9		
0.0277 mm.	68.1		
0.0183 mm.	60.2		
0.0112 mm.	49.1		
0.0081 mm.	44.4		
0.0058 mm.	41.2		
0.0029 mm.	35.7		
0.0012 mm.	31.7		

* (no specification provided)

Soil Description

Clayey silt,trace gravel,some sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2414 D_{85}= 0.0807 D_{60}= 0.0181
D_{50}= 0.0117 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sampled by Andy on Oct 2, 2013

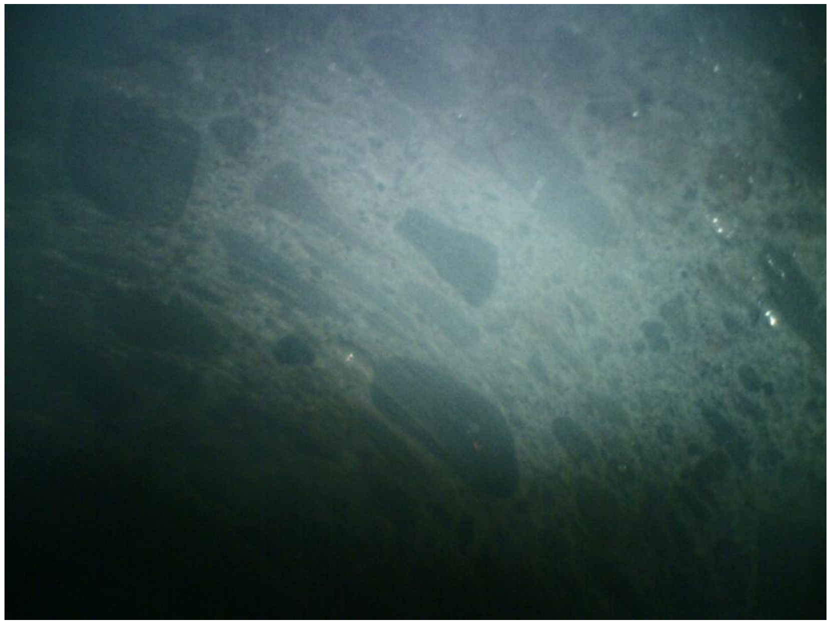
Location: CH4 (SA1)
Sample Number: MM-0216 **Depth:** 78" - 84" **Date:**

SPL Consultants Limited
Geotechnical Environmental Materials Hydrogeology

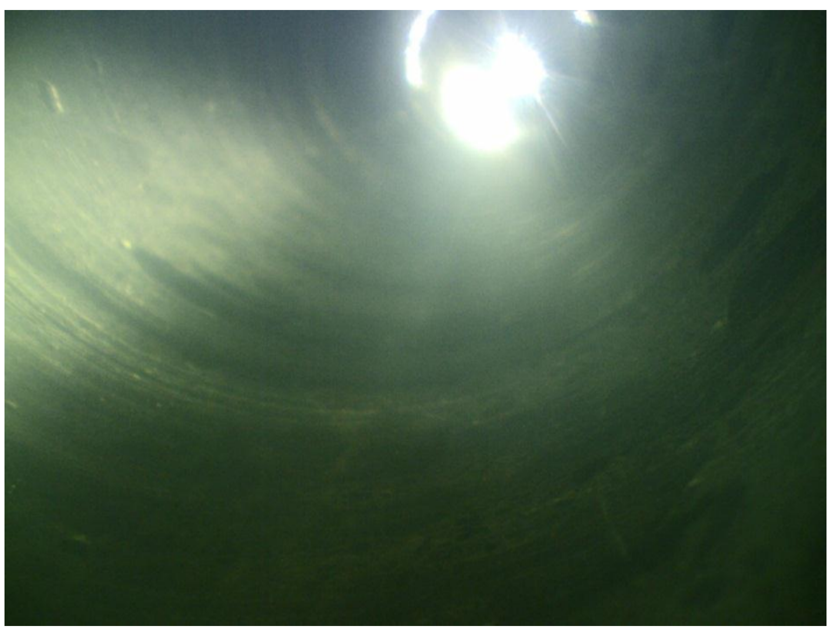
Client: Associated Engineering
Project: Geotechnical Investigation,Hasting Swing Bridge,Hastings.
Project No: 1842-910 **Figure** 7

APPPENDIX A

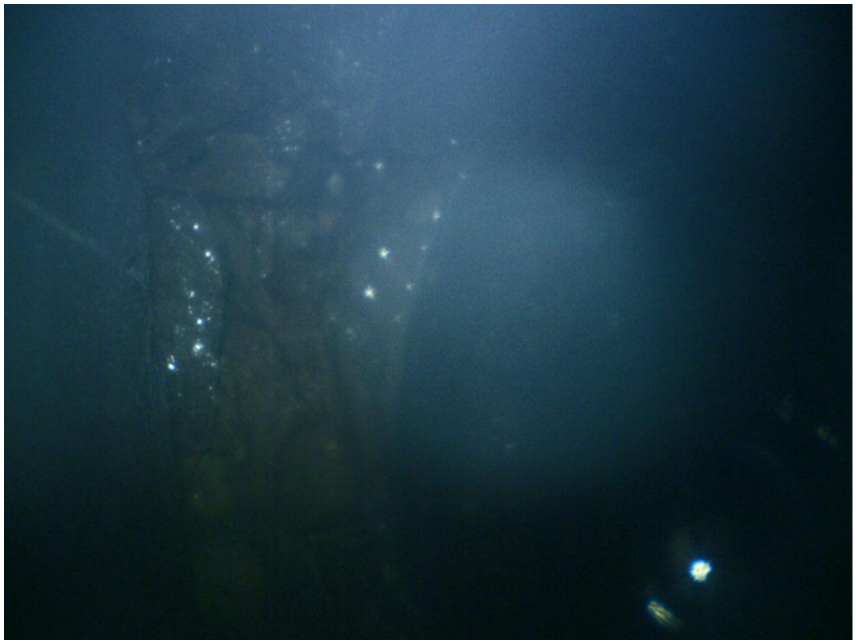
PHOTOGRAPHS INSIDE COREHOLES



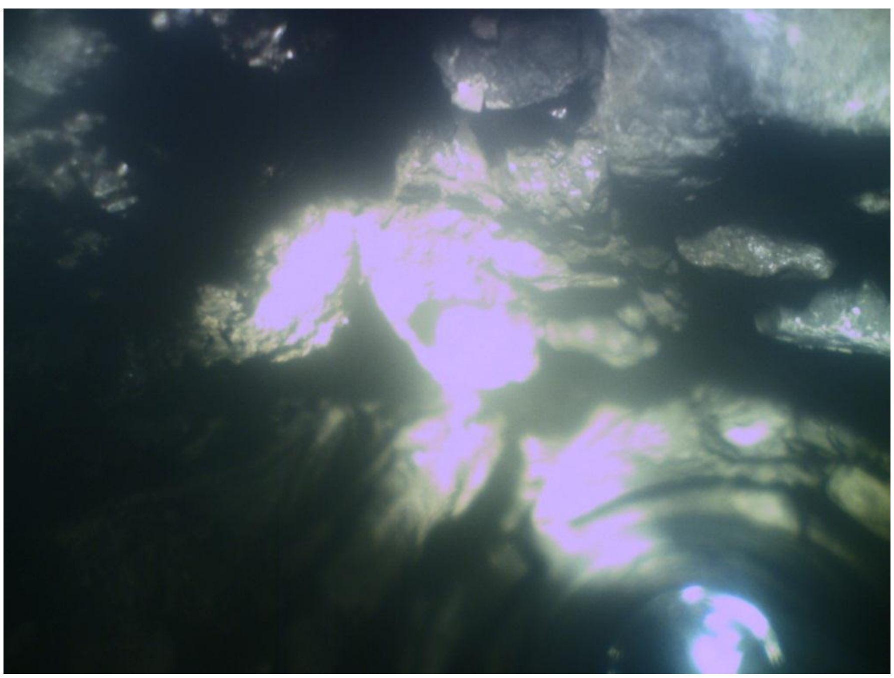
Photograph 1: no cracking or fracture of the pier concrete noted inside the corehole



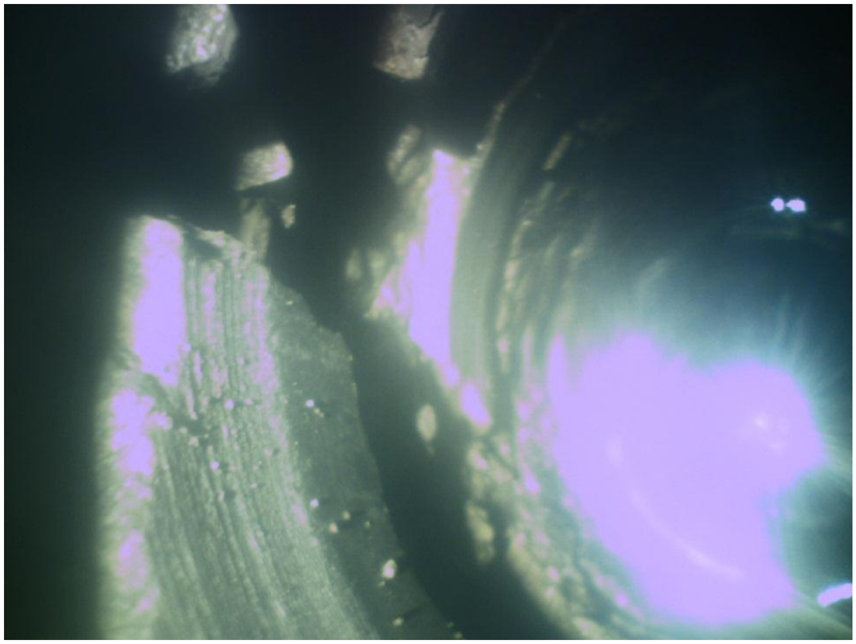
Photograph 2: no cracking or fracture of the pier concrete noted inside the corehole



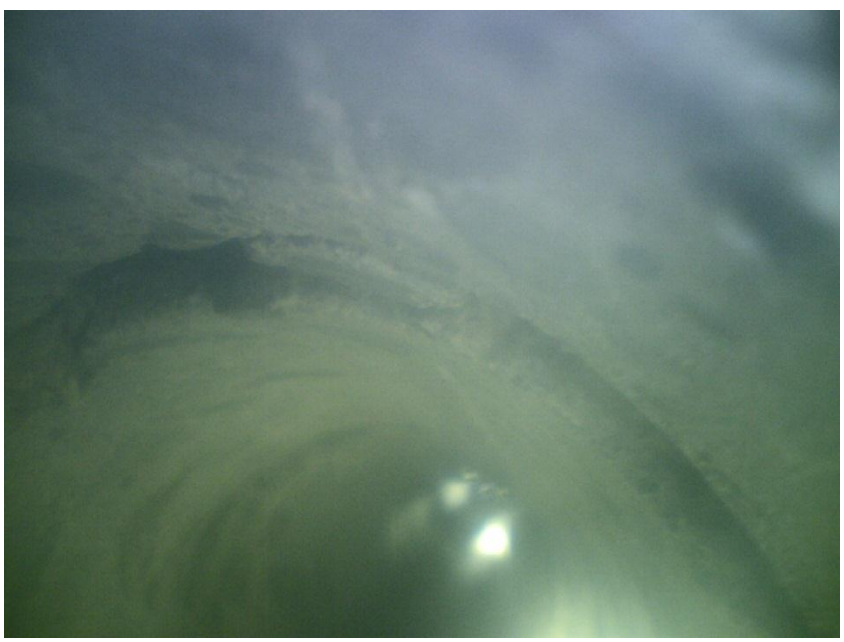
Photograph 3: highly weathered rock between pier concrete and bedrock



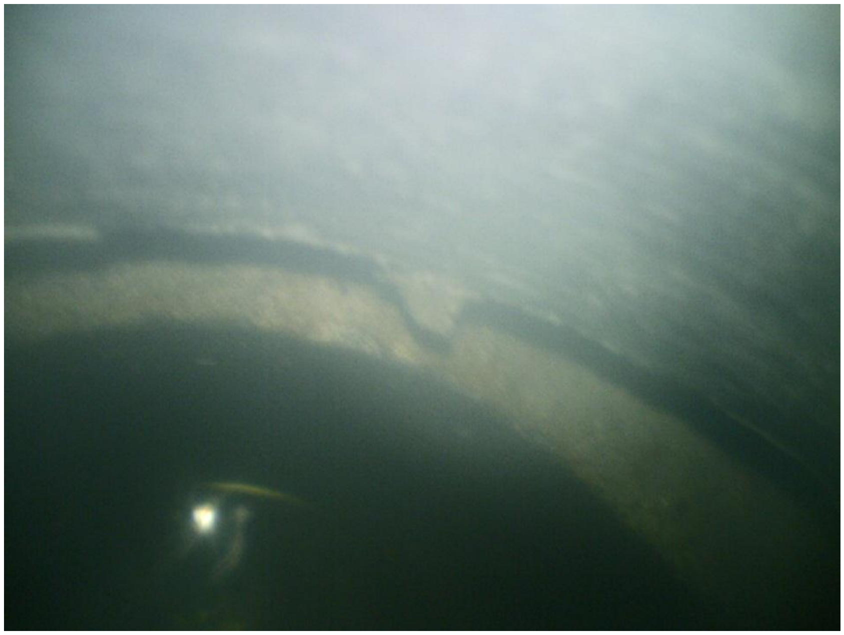
Photograph 4: highly weathered rock below pier concrete; note the clayey silt soil within the voids between rock pieces



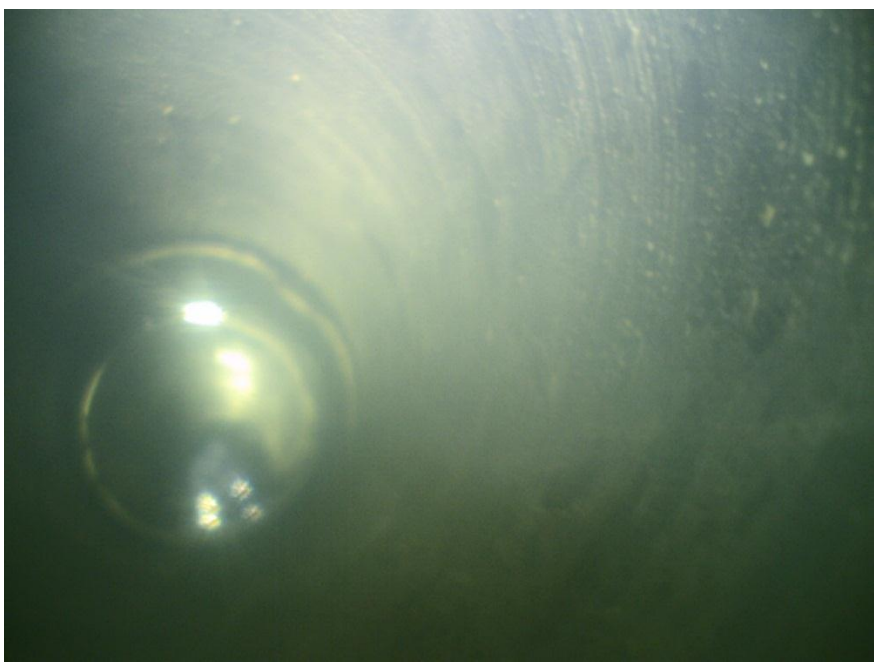
Photograph 5: void between pier concrete and bedrock and highly weathered rock pieces



Photograph 6: approximately 1 inch void between the pier concrete and bedrock



Photograph 7: approximately 2 inches void between the pier concrete and bedrock



Photograph 8: the void between the pier concrete and bedrock

APPPENDIX B

PHOTOGRAPHS OF CONCRETE AND ROCK CORES

Photograph 1: Corehole CH 13-1 Concrete Cores and partial Rock Cores

Photograph 2: Corehole CH 13-1 Concrete Cores and full depth Rock Cores

Photograph 3: Corehole CH 13-2 Concrete Cores and full depth Rock Cores

Photograph 4: Corehole CH 13-2 Concrete Cores and zone of clayey silt within the limestone rock

Photograph 5: Corehole CH 13-2 Layers of clayey silt within the limestone rock

Photograph 6: Corehole CH 13-3 Concrete Cores and Rock Fills or Highly Weathered Rock pieces

Photograph 7: Corehole CH 13-3 Rock Fills or Highly Weathered Rock Pieces

Photograph 8: Corehole CH 13-4 Rock Cores

Photograph 9: Corehole CH 13-4 Concrete Cores and Rock Cores

Photograph 10: Layers of clayey silt within the limestone rock

Appendix C

Concrete Core Strength Testing Results

SPL Consultants Limited
351 Steelcase Road West, Unit 10-12
Markham, Ontario, L3R 4H9
Phone: (905) 475-0065
Fax: (905) 475-0045

Concrete Strength Test Report
100mm X 200mm Cylinder

Project Name: Hasting Swing Bridge, Hastings.
Client: Associated Engineering
Project Number: 1842-910
Lab Number: MC-0134
Mix Reference: Concrete Cores

Specified Day Strength MPa

Sample No.	Date Cast	Date Tested	Curing	Age (Days)	T.O.F.*	Unit Mass (Kg/m3)	Strength (MPa)
CH 13-1	01-Oct-13	02-Oct-13	Lab	1	D	2423	36.9
CH 13-2	01-Oct-13	02-Oct-13	Lab	1	D	2317	35

Contractor:
Location of Structure: CH 13-1 and CH 13-2
Concrete Supplier:
Cylinder Cast By: A.Z.
Time Mixer Charged:
Specified Slump (mm) **Min:** **Max:**
Concrete Temp. (C):
Specified Air (%): **Min:** **Max:**
Truck No: **Ticket No:**
Water Added on Job: Not Observed
Nom. Size Aggregate (mm):
Type of Admixtures: Not Available
Date Received in Lab: Tuesday, October 01, 2013

Report Number:
Representing: SPL
Time Cylinders Cast:
Measured Slump (mm):
Air Temp. (C):
Measured Air (%):
Load No./ Cum. Total (m3):
By What Authority: Not Applicable
Type of Mould: Concrete Cores
Initial 24 Hr. Curing Temp. (C):
Maximum: **Minimum:**

Remarks: CH 13-1 Depth:0.15 - 0.35 m CH 13-2 Depth:0.50 - 0.80 m

CCIL Certified Concrete Testing Laboratory

C	CV	V	D	E	O
Cone	Cone-Split	Columnar	Shear	End	Other

*Type of Fracture

Reviewed By:

Appendix D

Rock Core Strength Testing Results

DEPARTMENT OF
MINING ENGINEERING

Goodwin Hall
Queen's University
Kingston, Ontario, Canada K7L 3N6
Tel 613 533-2230
Fax 613 533-6597

October 25, 2013

Mr. David Liu
SPL Consultants Limited
351 Steelcase Road W, Unit 10-12
Markham, ON L3R 4H9

Re: Core sample testing (Project #1842-910)

Mr. Liu:

One core sample for Project #1842-910 was prepared and tested for determination of unconfined compressive strength. The unconfined compression specimen was subjected to a process of preparation that included:

-diamond lathing (where feasible) to prepare sample faces parallel to within $\pm$ 0.025 mm
-testing to unconfined failure within a servo-controlled compression frame; all tests were performed under axial strain control at rates approximating 10^{-5} s^{-1}, and simultaneous recording of axial force and axial deformation was conducted, from which determination of the sample Unconfined Compressive Strength (UCS) and other parameters were obtained

A summary of strength test results and sample photographs of the pre- and post-test specimen is also attached. Should you also require any additional information concerning work that has been performed, please do not hesitate to contact me by telephone at (613)-545-2198 or by FAX at (613)-545-6597.

Yours sincerely,

J. F. Archibald, Ph.D., P. Eng., FCIM

Failure Test Results
(SPL Consultants Limited Project #1842-910) – October 2013)

Sample (depths indicated)	Bulk Density (g/cm^3)	UCS (MPa)	Young's Modulus (GPa)	Poisson's ratio
BH1, Run-2 (4'9"-5'6")	2.72	41.9	41.860	---

Pre-Test Samples

Post-Test Sample

Appendix E

Report of the Previous Geotechnical Investigation carried out by Golder

November 21, 2011

GEOTECHNICAL EVALUATION

Hastings Swing Bridge
Trent-Severn Canal
Hastings, Ontario

Submitted to:
Mr. Jonathan Werner, P.Eng.
Delcan Corporation
625 Cochrane Drive, Suite 500
Toronto Ontario
L3R 9R9

REPORT

Report Number: 11-1184-0022

Distribution:

4 Copies - Delcan Corporation

2 Copies - Golder Associates Ltd.

A world of capabilities delivered locally

Golder Associates

Table of Contents

1.0 INTRODUCTION

This report presents the results of a geotechnical evaluation carried out for the existing Hastings Swing Bridge located in Hastings, Ontario on the Trent-Severn Canal, as shown on the Key Plan, Figure 1. The swing bridge is located immediately north of the existing fixed bridge.

The purpose of the investigation was to investigate the subsurface conditions and shallow groundwater conditions at the site of the swing bridge by means of a limited number of shallow boreholes/coreholes and one test pit. Based on our interpretation of the borehole /corehole / test pit data and our review of the laboratory test results, this report provides the following geotechnical evaluations:

- Geotechnical evaluation of the conditions of the existing north abutment;
- Geotechnical evaluation of the condition of the existing pier supporting the swing bridge;
- Geotechnical evaluation of the stability of the north abutment and central pier;

The results for a bridge condition survey carried out by Golder are reported under a separate cover.

Authorization to proceed with this investigation was given by Mr. Jonathan Werner of Delcan Corporation (Delcan) in an email dated March 28th, 2011.

The factual data, interpretations and recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location. If the project is modified in concept, location or elevation, or if the project is not initiated within eighteen months of the date of the report, Golder Associates Ltd. (Golder) should be given an opportunity to confirm that the recommendations are still valid. In addition, this report should be read in conjunction with the attached "Important Information and Limitations of This Report" included in Appendix A. The reader’s attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report.

2.0 BACKGROUND INFORMATION AND SITE DESCRIPTION

Hastings Swing Bridge is located at Lock 18 of the Trent-Severn Canal, in Hastings, Ontario. The present bridge was constructed by the Central Bridge Company in 1952, and is now classified by Parks Canada as ‘Other’ under Cultural Resources, requiring no specific historical rehabilitation. Some deck grating and slab repairs occurred in 1996 and electrical upgrades in 1997, gate arms and hydraulics were installed to replacing the electric drive system. Emergency repairs recently took place in December, 2010 including temporary reinforcing of the southern most transverse floor beam and one stub stringer, partial steel plate replacement on the counter weight of the north bridge nosing, abutment repair at the south bridge nosing plate and concrete curb/sidewalk repair at the southeast end of the bridge.

Based on the information provided by Parks Canada as outlined in the Request For Proposal (RFP) dated February, 2011, the general swing bridge structure descriptions are listed below:

1) The Swing Bridge is a deck plate girder construction with a combination steel grate and asphalt covered concrete deck.

2) The overall length of the bridge is 25.68 metres (84’-3”) long from panel end to panel end and has a width of 8.15 metres (26’-9”) from center line to center line of the outer most girders.

3) The swing span is an unequal arm type pivoting above a centre pintle with balance wheels.

4) The swing bridge is supported by an off center concrete pivot pier and concrete abutments. The off centre pivot pier is located on the north side of the Trent Canal.

5) The bridge is not currently posted for maximum load.

6) The two-lane bridge is a high traffic volume crossing.

7) The deck serves vehicular as well as bicycle traffic. A steel grate pedestrian walkway is mounted on the south side of the bridge, outside the south girder.

Based on the information provided in the RFP, the current bridge conditions and the history of the rehabilitation and repairs are listed below:

1) The southern most transverse floor beam and one stub stringer has had temporary strengthening support added in December 2010.

2) Some corrosion has been observed on steel members.

3) Some surface deterioration and paint peeling is evident on steelwork.

4) Some vertical stiffeners on the plate girders are bent and/or corroded

5) Extensive deterioration of the north end concrete deck and ballast. Concrete has spalled off, exposing reinforcing steel which has severe corrosion. Nosing plate has had previous repairs but is questionable.

6) Substantial deterioration of both vertical and top faces of the south canal wall includes surface crack formation, exposed reinforcing steel and surface spalling.

7) Second pour concrete support pad for bridge wheels has deterioration.

8) Vertical face of curved section of abutment has spalling and cracking. The surface of the curved abutment behind the nosing plate received repair in December 2010.

9) The concrete sidewalk sections that abut the wing walls on the south side have substantial cracking and spalling.

10) The north canal wall has substantial deterioration of the vertical and top faces spalling and deterioration includes spalling, and deep surface cracking.

11) East and west guardrail posts are corroded and have broken welds at their base plates, some connections at posts and rails have corroded through.

12) Splash plates that form the steel curbing along the plate girders have severe corrosion

13) Deteriorated concrete under the balance wheel rail track has eliminated continuous support of the rail, affecting vertical alignment.

14) Dam service electrical conduit under the north end of the bridge is corroded

15) Cylinder mounting bolts and boots broken and or missing. Cylinders need to have rust removed and a form of protection incorporated

16) Pillow block rests have had shims removed to accept bridge, indicating pintle has had settlement. Last adjusted 1992.

17) Jack cylinders are 25 years old.

3.0 INVESTIGATION PROCEDURE

The field work for this investigation was carried out on May 3, 6 and 7, 2011, during which time 4 boreholes, 5 coreholes and one test pit were advanced at the locations shown on the Borehole, Corehole and Test Pit Location Plan, Figure 2. The boreholes were drilled using a truck-mounted drillrig supplied and operated by a drilling specialist, under our supervision. Standard penetration testing and sampling were carried out at regular intervals of depth in the boreholes using conventional 35 mm internal diameter split spoon sampling equipment. The test pits were carried out using a backhoe supplied and operated by an excavation subcontractor, under our supervision. The coreholes were carried out using a coring machine supplied and operated by a coring specialist, under our supervision.

Shallow groundwater conditions were noted in the open boreholes during drilling. All of the boreholes and test pits were loosely backfilled and sealed at the surface upon completion of drilling and test pitting.

All of the soil samples, concrete cores and rock core samples obtained during this investigation were brought to our Whitby laboratory for further examination, natural water content testing, selected classification testing and compressive strength testing.

The field work for this investigation was directed by members of our engineering staff who also determined the borehole/corehole/test pit locations in the field, logged the boreholes/corehole/test pit, and cared for the samples obtained.

4.0 REGIONAL GEOLOGY

The site is located within the physiographic region known as the Peterborough Drumlin Field (Chapman, L.J. and Putnam, D.F. “The Physiography of Southern Ontario”, 3rd Edition, 1984). This region is lying north of the Oak Ridges Moraines with a rolling till plains with numerous drumlins. For most of the part, the bedrock underlying this region is limestone of the Lindsay and Verulam Formations which are somewhat softer and less massive formations than the Gull River formation. They are also highly fossiliferous and disintegrate easily. The beds slope slightly towards the southwest and the edges overlapping strata face north. Based on the findings in this geotechnical investigation, the soil and bedrock conditions are generally consistent with the Regional Geology.

5.0 SUBSURFACE CONDITIONS

The existing subgrade soils and shallow groundwater conditions encountered in the coreholes/boreholes and test pits, as well as the results of the field and laboratory testing, are shown in detail on the Record of Borehole/Corehole and Record of Test Pit sheets, following the text of this report. Lists of abbreviations and symbols are provided to assist in the interpretation of the borehole logs. Profiles of the structure and the subsurface stratigraphy below the structure are presented on cross section drawings, Figures 3A to 3C. The results of soil laboratory gradation analyses are provided on Figures 4 and 5.

It should be noted that the boundaries between the strata shown on the borehole/corehole logs have been inferred from drilling/coring observations and non-continuous samples. They generally represent a transition from one soil type to another and should not be inferred to represent an exact plane of geological change. Further, conditions will vary between and beyond the boreholes. The following is a summarized account of the subsurface conditions encountered in the boreholes drilled at the site, followed by more detailed descriptions of the existing fill and native soil strata, and shallow groundwater conditions.

The subsurface soil conditions generally consisted of granular fill containing some rock fragments, overlying limestone bedrock.

5.1 Pavement Structures

Pavement structure was encountered surficially in Boreholes 1, 3 and 4 along the road behind the north abutment wall. The pavement structure consisted of 120 mm of asphalt overlying about 400 mm of granular base.

5.2 Topsoil

Topsoil was encountered surficially in Borehole 2. The thickness of the topsoil was 130 mm.

5.3 Fill Materials

Fill materials were encountered in all of the boreholes and in the test pit. The fill extended to depths ranging from 1.2 m to 2.7 m below ground surface. The fill materials are associated with previous backfilling behind the abutment walls and surrounding the central pier. The fills are variable in composition but generally consist of silty sand, gravelly sand and sandy gravel with variable sized rock fragments encountered at all depths. Standard penetration tests carried out within the various fill materials gave variable N values ranging widely from 3 blows to 53 blows per 0.3 m penetration, indicating a very loose to very dense relative density, although the higher N values could be influenced by the presence of the large sized rock fragments. The in-situ water content of the fill samples tested ranged widely from 2 percent to 9 percent. Grain size distribution curves for samples of the sandy gravel and gravelly sand fills are shown on Figures 4 and 5.

5.4 Bedrock

Based on the results of rock coring, the bedrock at the site generally consists of slightly weathered to weathered, grey to dark grey, fine grained fossiliferous limestone of the Lindsay and Verulam Formations. This bedrock was confirmed by coring in Coreholes 1 to 4 and in Test Pit 1 to depths of 1.8 m to 3.0 m below the existing ground surface. The surface elevation of the bedrock is variable at the test hole locations, as shown on the cross section drawings, Figures 3A to 3C.

The Total Core Recovery (TCR) of the core samples ranged from 39 percent to 100 percent; the Solid Core Recovery (SCR) ranged from 18 percent to 93 percent; and the Rock Quality Designation (RQD) ranged from 0 percent to 93 percent. The RQD values for the bedrock cores sampled immediately below the concrete structures were generally 0 percent. Based on these results and on our visual examination of the core samples, the rock quality of the limestone encountered is generally considered to be very poor immediately below the concrete structures becoming good to excellent with depth.

Two samples of the rock core from Coreholes 2 and 4 were prepared and subjected to compressive strength testing. This testing was carried out in general accordance with ASTM Standard Test Method D 7012-07, entitled "Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens". This testing gave unconfined compressive strength values of 151.1 MPa and 90.3 MPa, indicating that the strength of this bedrock is classified as very strong and strong (Canadian Foundation Engineering Manual, 2006, 4th Edition, Table 3.5).

It should be noted that a portable coring rid and small diameter coring bits were used for the coring, due to the very restricted work area underneath the swing bridge, Core breakage associated with the use of the smaller coring bits may have resulted in lower rock quality measurements than may have been recorded for larger diameter cores.

5.5 Shallow Groundwater

Details of our groundwater level observations are shown on the Record of Borehole and Test Pit sheets, which follow the text of this report. The water levels encountered upon completion of drilling and test pit excavation were at depths of 1.7 m and 1.8 m below ground surface in the borehole and test pit carried out immediately adjacent to the central pier. The boreholes located along the road behind the north abutment were dry upon completion of drilling. It is considered that the stable groundwater levels at the bridge site would be affected by the water level in the canal and the local prevailing water level and some seasonal fluctuations should be anticipated.

6.0 DISCUSSION

This section of the report provides engineering information for the geotechnical design aspects of the project, based on our interpretation of the borehole data and on our understanding of the project requirements. The information in this portion of the report is provided for the guidance of the design professionals. Where comments are made on construction, they are provided only in order to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking any work at the site should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, equipment capabilities, costs, sequencing and the like.

Our professional services for this assignment address only the geotechnical (physical) aspects of the subsurface conditions at this site. The geo-environmental (chemical) aspects, including the consequences 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 report and have not been investigated or addressed.

6.1 Project Description

The Trent-Severn Canal is operational from mid April until the end of October and open to the public for navigational purposes from the Friday before the May long weekend until the Wednesday following the Canadian Thanksgiving long weekend. It is understood that the swing bridge is swung away from canal five to eight times a day during the operational season to allow boats to pass the lock. The swing bridge is located on Bridge Street South in Hastings, Ontario and as part of Highway 45, experiences high traffic volumes. It is understood that the purpose of the investigation and evaluation is to determine the appropriate rehabilitations for the swing bridge only.

The existing Hastings Swing Bridge is a deck plate girder construction with a combination steel grate and asphalt covered concrete deck. The swing span is an unequal arm type pivoting above a centre pintle with balance wheels. The swing bridge is supported by an off-center concrete pivot pier and concrete abutments. The off-centre pivot pier is located on the north side of the Trent-Severn Canal. The bridge is not currently posted for maximum load. As noted above, the two-lane bridge is a high traffic volume crossing. The deck serves vehicular as well as bicycle traffic. A steel grate pedestrian walkway is mounted on the south side of the bridge, outside the south girder.

It is understood that the loading of the super-structure of the swing bridge is generally supported by the combination of the central pintle, the pillow block rests and balance wheels. The swing bridge is swung by two hydraulic jacks built underneath the bridge deck within the circular track on top of the central concrete pier. The south end of the swing bridge deck is supported by two wheels sitting on two steel plates. The north end of the swing bridge deck is supported by two hydraulic jacks. During the non-operational seasons, the north and south ends of the swing bridge deck are supported by steel posts placed underneath the deck between two hydraulic jacks at the north end and two wheels at the south end. It is anticipated that the loading at the south and north ends are generally light (i.e. primarily live loading from traffic).

It is further understood that the last emergency repair was carried in December of 2010. The bridge deck and slab repair and the hydraulic system upgrades were carried out in 1996 to 1997. The pillow block rests have had shims removed to accommodate the settlement of the bridge (i.e. the settlement of the central pintle was assumed); the last adjustment was carried out in 1992.

The concrete of the west portion of the face wall of the north abutment (as shown in Photo No. 22 on Figure 6K and No.23 on Figure 6L) appears to be poured in different years. The concrete on the top portion of the retaining wall northwest of the bridge also appears to be poured in different years (as shown in Photo No. 25 on Figure 6M). Localized repairs to the concrete surface of the existing pier and abutments in addition to those described in the RFP were also noted, however, the history of these repairs were not available at the time of preparing the report.

The Borehole, Corehole and Test Pit Location Plan, Figure 2, was developed based on the previous drawing provided by Delcan and on measurements made during our field investigation. The cross-sections of the existing structures below the water and ground, as shown on Figures 3A to 3D, are based on the interpretation of our borehole/corehole/test pit data, and therefore should be considered as approximate only and are only suitable for illustrative purposes. Site Photographs are provided on Figures 6A to 9R.

6.2 Geotechnical Evaluations

6.2.1 Existing Central Pier

The construction history, design drawings and as-built drawings for the existing swing bridge are not currently available.

Two coreholes were drilled on top of the existing central pier within the balance wheel track, one borehole was drilled on the east side of the existing central pier and one test pit was excavated on the west side of the exiting central pier. Based on the results of the coring, the existing central pier is founded on fractured limestone at a depth of 1.2 m and 1.7 m below the top surface of the central pier at the locations of Coreholes 2 and 3, respectively, as shown on Figure 2, 3A and 3B. Based on the observations from the test pit on the west side of the existing central pier as shown on Figure 3B and Photos No. 35 and 36 on Figure 6R, the depth of the bedrock is about 1.8 m below the ground surface (i.e. 1.8 m below the top of the concrete pier), which indicates elevation of the bedrock surface varies over a short distance in this area. A concrete mud slab with an approximate thickness of 200 mm was encountered in the test pit at a depth of about 1.2 m below the top elevation of the central pier, which is generally consistent with the rock depth encountered in Corehole 2. Weathered and fractured limestone bedrock was encountered below the mud slab. Based on the borehole data from Borehole 2 located east of the central pier, the bedrock is present at a depth of approximate 1.7 m below the ground surface, which is generally consistent with the bedrock depth encountered in Corehole 3.

Based on the our observations during the concrete coring and our visual inspection of the concrete cores recovered from Coreholes 2 and 3, no voids or fractures were observed at two corehole locations. Based on the observations from the test pit, there no cracks are visible on the exposed vertical face on the west side of the central pier.

No significant voids were noted within the bedrock during the advance of two coreholes at the top of the pier; however, the quality of the upper portion of the limestone, immediately below the concrete pier, was very poor with R.Q.D. measurements of 0 percent at both locations. Relative higher quality bedrock was encountered at a depth of about 2.0 m below the ground surface, which is generally consistent with the sound bedrock encountered in the test pit (i.e. 1.8 m below ground surface). The poor quality of the bedrock at the founding level of the central pier could be a result of the weathering and deterioration of bedrock after the completion of the construction, or it could indicate that the weathered/fractured bedrock was not removed at the time of the construction.

The design details of the central pintle and the existing pier are not available however, it is understood that, the structural loading of the bridge structure is mainly supported by the central pier. Due to the significant variability of the rock quality at the founding level, estimation of further settlement of the central pier under future structural loading is not feasible and it is recommended that future structural loads should not exceed the original design loads unless a more detailed geotechnical study is carried out to fully evaluate the engineering properties of the bedrock below the existing foundation.

It is understood that shims for the pillow block rests were removed about 20 years ago to accommodate the settlement that had occurred at the central pintle location. However, no further settlement has been reported and no further adjustment of the pillow block rests have been carried out since that time. The design and as-built information for the pintle, the loading distribution for the pintle and the as-built steel reinforcement of the exiting pier are not available. Based on our visual observation and measurements, the pintle is a round steel plate with an approximate diameter of 0.8 m bolted on top of a hexagon shape concrete platform which is slightly elevated above the concrete pier as shown in Photo No.21. No cracking was observed in the concrete surrounding the steel plate and in the hexagon shaped concrete platform. However, this concrete appeared to have been placed during relatively recent repair works and cracks that may be present in the original concrete would be obscured by this newer concrete.

It is recommended that the structural engineer carry out a detailed structural stability analysis to evaluate the potential for "punching" failure to occur within the small area below the pintle base. . The following parameters are provided for the analysis of the structural stability analysis purposes:

- Unit weight of fractured limestone = γ = 23 kN/m³
- Unit weight of water = γ_w = 9.8 kN/m³
- "Unfactored coefficient of friction between Concrete and fractured limestone bedrock = μ = 0.3

The thickness of the concrete could be assumed to be between 1.2 m to 1.7 m and the concrete pier should be assumed to be non-reinforced concrete unless the as-built reinforcement of the pier can be confirmed.

Four samples of the concrete core from Coreholes 2 and 3 were prepared and subjected to compressive strength testing. This testing was carried out in general accordance with CSA A23.2-14C and the results are attached in Appendix B of the report and are summarized in the following table:

Corehole No.	Sample No.	Sample Depth (m)	Density (Mg/m³)	Compressive Strength (MPa)
2	1	0.1 -0.25	2.413	49.6
2	2	0.75 -1.03	2.399	55.1
3	1	0-0.15	2.260	30.8
3	2	1.4-1.67	2.459	48.5

It is recommended that the uneven/unlevel balance wheel track and the void below the track should be repaired.

The surficial clear stone fill on the west side of the pier should be removed and grass should be placed to shed surface runoff water away from the pier. The water levels in the canal are higher than the founding elevation of the central pier. The surface cracking and deterioration of the canal concrete wall should be adequately repaired to minimize water infiltration into and below the pier

6.2.2 Existing South Abutment and Canal Concrete Walls

The geotechnical investigation and evaluation of the south abutment and canal walls are not within the scope of the work for this current assignment. However, from visual observations, the abutment footing and canal walls appear to be severely deteriorated and should be repaired as required.

6.2.3 Existing North Abutment

Coreholes 1 and 4 were drilled on top of the existing abutment footing close to the support jacks as shown on Figure 2 and 3C and in Photos No. 26 and 28. Corehole 5 was drilled horizontally in the existing crack on the vertical wall east of the support jack (refer to Photo No. 28). Three boreholes (Boreholes 1, 3 and 4) were located north of the abutment wall.

Based on the results of the coring carried out on top of the abutment footing, the existing abutment slab was founded on limestone bedrock at a depth of 1.2 m and 2.4 m below the top surface of the abutment footing at the locations of Coreholes 1 and 4, respectively, which indicates at the elevation of the bedrock surface varies over a short distance. Based on the inferred bedrock depths from the borehole data from Boreholes 1, 3 and 4, the existing concrete footings of the abutment wall are founded on limestone bedrock at a depth of about 4 m below ground surface.

Based on the our observations during the concrete coring and our visual inspection of the concrete cores recovered from Coreholes 1 and 4, no voids or fractures were observed at two corehole located at the top of the abutment footing. Corehole 5 was drilled horizontally in the existing crack on the abutment wall face and extended into the abutment wall for a distance of 1.7 m where it terminated in the concrete of the wing wall. The crack extended at least 0.7 m beyond the face of the abutment wall (i.e. north direction). The crack is extends approximately 1.5 m from the east edge of the abutment wall towards the west as shown in Photos 24 and 28. A similar straight line crack was observed on the west portion of the abutment wall as shown in Photos No.22 and 23. The crack appeared to have formed at a construction joint, as evidenced by the surface treatment of the concrete and the generally straight nature of the crack. The width of the existing crack was measured to range from 0 mm to 30 mm. It is recommended that this crack should be adequately sealed to prevent further deterioration of the concrete from exposure.

No other significant cracks, which may be evidence of excessive foundation settlement, were observed, however, previous concrete repairs may have obscured other existing cracks.

Only one drainage hole was observed along the abutment wall, as shown on Photo 23.

The quality of the upper portion of the limestone, immediately below the concrete pier, was very poor with R.Q.D. measurements of 0 percent at both locations. Further, based on the rate of coring advance, it seems that a void is present at the bedrock surface below the abutment footing at the location of Corehole 1. The poor quality of the bedrock at the founding level of the north abutment could be a result of the weathering and deterioration of bedrock after the completion of the construction, or it could indicate that the weathered/fractured bedrock was not removed at the time of the construction.

It is understood that no significant lateral movement of the abutment wall towards the bridge deck and no settlement of the abutment footing have been recorded. As previously noted, the structural loading at the north abutment is relatively light and no significant cracks were observed at the support jacks and at the locations of the steel posts.

Due to the significant variability of the rock quality at the founding level, estimation of further settlement of the north abutment under future structural loading is not feasible and it is recommended that future structural loads should not exceed the original design loads unless a more detailed geotechnical study is carried out to fully evaluate the engineering properties of the bedrock below the existing foundation.

A global stability analysis has been carried out as shown on Figure 7. Based on the analysis, global stability factor of safety for the north abutment is greater than the typical minimum requirement of 1.5. However, a more detailed structural stability analysis may be necessary, especially at the location of the existing crack at the east portion of the abutment wall. The following parameters are provided for structural stability analysis purposes:

- Unit weight of existing granular backfill = γ = 21 kN/m³
- Unit weight of fractured limestone = γ = 23 kN/m³
- Unit weight of abutment wall = γ = 24 kN/m³
- Unit weight of water = γ_w = 9.8 kN/m³
- "Active" lateral earth pressure coefficient = K_a = 0.3
- "At Rest" lateral earth pressure coefficient = K_o = 0.5
- Unfactored coefficient of friction between Concrete and fractured limestone bedrock = μ = 0.3

The groundwater level could be assumed at the road surface behind the abutment wall due to the poor drainage.

6.2.4 Additional Comments

Proper repairs to the existing structures are recommended to reduce the rate of future deterioration of the structural concrete and of the foundation bedrock. It is recommended that the condition of the swing bridge be periodically monitored and photographed by a geotechnical engineer to document the state of the deterioration so that appropriate remedial actions may be taken in the future.

7.0 MONITORING AND TESTING

Once the rehabilitation /repair design is finalized, this report should be reviewed by the geotechnical engineer to confirm that the subsurface information obtained and geotechnical recommendations provided are sufficient. An additional investigation may be required, if deemed necessary.

In addition, the geotechnical aspects of the final design drawings and specifications should be reviewed by this office prior to tendering and construction, to confirm that the intent of this report has been met.

During construction, sufficient subgrade inspections and in-situ materials testing should be carried out to confirm that the conditions exposed are consistent with those encountered in the boreholes and to monitor conformance to the pertinent project specifications. Asphalt and concrete testing should be carried out in CCIL and CSA certified laboratories, respectively.

We trust that this report provides sufficient geotechnical engineering information to facilitate the detailed design of this project. If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

We trust that this report provides sufficient geotechnical engineering information to facilitate the designs of the proposed rehabilitation. As noted above, additional inspection and monitoring by a structural and geotechnical engineer should be required for your present requirements. If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

Yours truly,

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
D. B. LIU
100107874
Nov 21/2011
PROVINCE OF ONTARIO

1111840022 2011 11 21

David B. Liu, P. Eng.
Geotechnical Engineer

Ty J. Garde, P. Eng., Principal
Senior Geotechnical Engineer

DBL/TJG/ms/sv/leb

n:\active_2011\1184 pavement materials\11-1184-0022 delcan swing bridge hastings\reports\geo report\11-1184-0022 final rep 2011'11'21 geotechinvestigation hastingsbridge.docx

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. 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.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Penetration Resistance; 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

Piezo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected 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 (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
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, s_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
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, G_S)
DS	direct shear test
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	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

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
F factor of safety
V volume
W weight

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
ν Poisson's ratio
σ total stress
σ' effective stress ($\sigma' = \sigma - u$)
σ'_{vo} initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$ principal stresses (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

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

w water content
w_l liquid limit
w_p plastic limit
I_P 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)

(c) 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

(d) Consolidation (one-dimensional)

C_c compression index (normally consolidated range)
C_r recompression index (overconsolidated range)
C_S swelling index
C_α coefficient of secondary consolidation
m_v coefficient of volume change
c_v coefficient of consolidation
T_v time factor (vertical direction)
U degree of consolidation
σ'_p pre-consolidation pressure
OCR Overconsolidation ratio $= \sigma'_p/\sigma'_{vo}$

(e) 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 ($\phi = 0$ 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

Notes: 1. $\tau = c' + \sigma' \tan\phi'$
2. Shear strength = (Compressive strength)/2

RECORD OF COREHOLE 1

PROJECT: 11-1184-0022

LOCATION: SEE FIGURE 2

BORING DATE: May 6, 2011

SHEET 1 OF 1

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES – NUMBER	SAMPLES – TYPE	ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (25, 50, 75, 100) nat V. + Q - ● rem V. ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢ W ⊣ Wl (5, 10, 15, 20)	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		GROUND SURFACE									
0	50 mm Diameter Coring Machine	CONCRETE									
1		Weathered, grey to dark grey, highly fractured fossilferous Limestone, pockets of clayey silt T.C.R. = 70% S.C.R. = 35% R.Q.D. = 0%		1.19							
2											
		END OF COREHOLE		2.62							
3											
4											
5											
6											
7											
8											
9		T.C.R. - TOTAL CORE RECOVERY S.C.R. - SOLID CORE RECOVERY (% CYLINDRICAL) R.Q.D. - ROCK QUALITY DESIGNATION (% CORE RUN > 0.1 m LONG)									
10											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED:

LDN_BHS 11-1184-0022.GPJ GLDR_LDN.GDT 5/30/11 DATA INPUT: MK MAY 2011

PROJECT: 11-1184-0022

LOCATION: SEE FIGURE 2

RECORD OF COREHOLE 2

BORING DATE: May 6, 2011

SHEET 1 OF 1

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE - DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE		ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (25, 50, 75, 100) nat V. + Q - ● rem V. ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp —○W— Wl (5, 10, 15, 20)	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		GROUND SURFACE										
0	50 mm Diameter Coring Machine	CONCRETE										
1		Weathered, grey to dark grey, highly fractured fossilferous Limestone, pockets of clayey silt T.C.R. = 39% S.C.R. = 18% R.O.D. = 0%		1.17								
2		Slightly weathered grey to dark grey fine grained fossilferous Limestone BEDROCK T.C.R. = 84% S.C.R. = 63% R.O.D. = 50%		2.03								
		END OF BOREHOLE		2.52								
3												
4												
5												
6												
7												
8												
9		T.C.R. - TOTAL CORE RECOVERY S.C.R. - SOLID CORE RECOVERY (% CYLINDRICAL) R.O.D. - ROCK QUALITY DESIGNATION (% CORE RUN > 0.1 m LONG)										
10												

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: AZ

CHECKED:

RECORD OF COREHOLE 3

PROJECT: 11-1184-0022

LOCATION: SEE FIGURE 2

BORING DATE: May 6, 2011

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (25, 50, 75, 100)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ├──○W──┤ Wl (5, 10, 15, 20)	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		GROUND SURFACE									
0	50 mm Diameter Coring Machine	CONCRETE									
1											
		Weathered, grey to dark grey, highly fractured fossilferous Limestone, pockets of clayey silt T.C.R. = 90% S.C.R. = 80% R.O.D. = 0%		1.70							
2		Slightly weathered grey to dark grey fine grained fossilferous Limestone BEDROCK T.C.R. = 100% S.C.R. = 90% R.O.D. = 80%		1.96							
		END OF COREHOLE		2.08							
3											
4											
5											
6											
7											
8											
9		T.C.R. - TOTAL CORE RECOVERY S.C.R. - SOLID CORE RECOVERY (% CYLINDRICAL) R.O.D. - ROCK QUALITY DESIGNATION (% CORE RUN > 0.1 m LONG)									
10											

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: AZ

CHECKED:

RECORD OF COREHOLE 4

PROJECT: 11-1184-0022

LOCATION: SEE FIGURE 2

BORING DATE: May 7, 2011

SHEET 1 OF 1

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES – NUMBER	SAMPLES – TYPE	ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (25, 50, 75, 100)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (5, 10, 15, 20)	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		GROUND SURFACE									
0	50 mm Diameter Coring Machine	CONCRETE									
2		Weathered, grey to dark grey, highly fractured fossilferous Limestone, pockets of clayey silt T.C.R. = 80% S.C.R. = 80% R.O.D. = 0%		2.36							
				2.79							
3		Slightly weathered grey to dark grey fine grained fossilferous Limestone BEDROCK T.C.R. = 100% S.C.R. = 93% R.O.D. = 93%		3.00							
		END OF COREHOLE									

T.C.R. - TOTAL CORE RECOVERY

S.C.R. - SOLID CORE RECOVERY (% CYLINDRICAL)

R.O.D. - ROCK QUALITY DESIGNATION (% CORE RUN > 0.1 m LONG)

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED:

PROJECT: 11-1184-0022

RECORD OF COREHOLE 5

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

BORING DATE: May 7, 2011

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	SAMPLES: TYPE		ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (25, 50, 75, 100) nat V. + Q - ● rem V. ⊕ U - ○	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (5, 10, 15, 20)	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		GROUND SURFACE										
0	50 mm Diameter Coring Machine	CONCRETE										
1												
		END OF COREHOLE		1.73								
2												
3												
4												
5												
6												
7												
8												
9												
10												

DEPTH SCALE

1 : 50

LOGGED: AZ

CHECKED:

PROJECT: 11-1184-0022

RECORD OF BOREHOLE BH-1

LOCATION: SEE FIGURE 2

BORING DATE: May 7, 2011

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: 150 mm Diameter SOLID STEM AUGERS, AUTOMATIC HAMMER; Track Mounted Power Auger

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT PERCENT (W)	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
0	GROUND SURFACE							
	ASPHALT (120 mm)							
	GRANULAR BASE	0.12	1	24 DO	40	~6.5		
	Dense to compact brown gravelly sand, trace silt (FILL)	0.52	2	50 DO	20	~2.5		
	Loose to compact brown sandy gravel, trace silt, containing rock fragments (FILL)	1.37	3	50 DO	8	~4		
			4	50 DO	29	~5	M	
	Probable CONCRETE (Existing Footing)	2.74						
4	Probably weathered Limestone BEDROCK	3.96						
	END OF BOREHOLE DUE TO AUGER REFUSAL TO FURTHER AUGERING ON PROBABLE BEDROCK	4.22						Borehole open and dry upon completion of drilling, May 7, 2011

Chart headings: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (25, 50, 75, 100), nat V. + Q - ●, rem V. ⊕ U - ○; HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT Wp — W — Wl (5, 10, 15, 20); ELEVATION; STRATA PLOT.

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: AZ

CHECKED:

PROJECT: 11-1184-0022

RECORD OF BOREHOLE BH-2

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

BORING DATE: May 7, 2011

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (25 50 75 100)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl (5 10 15 20)	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		GROUND SURFACE										
0	150 mm Diameter SOLID STEM AUGERS, Track Mounted Power Auger	TOPSOIL										
		Compact to very dense brown gravelly sand, trace silt, containing rock fragments (FILL)		0.13	1	AS	-			○		
1					2	50 DO	30			○	M	
					3A	50 DO	53			○		
		Probably weathered Limestone BEDROCK		1.73	3B					○		
2		END OF BOREHOLE DUE TO AUGER REFUSAL TO FURTHER AUGERING ON PROBABLE BEDROCK		2.00								Water encountered during drilling at a depth of 1.52 m bgs, May 7, 2011. Water level in open portion of borehole at a depth of 1.83 m, upon completion of drilling, May 7, 2011 Borehole caved to a depth of 1.83 m, upon completion of drilling, May 7, 2011
3												
4												
5												
6												
7												
8												
9												
10												

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED:

RECORD OF BOREHOLE BH-3

PROJECT: 11-1184-0022

LOCATION: SEE FIGURE 2

BORING DATE: May 7, 2011

SHEET 1 OF 1

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: 150 mm Diameter SOLID STEM AUGERS, Track Mounted Power Auger

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT PERCENT (W)	INSTALLATION AND GROUNDWATER OBSERVATIONS
GROUND SURFACE	0					
ASPHALT (120 mm)						
GRANULAR BASE	0.12	1A	50 DO	32	○	
Dense to compact sandy gravel, trace silt, containing rock fragments (FILL)	0.50	1B	50 DO	32	○	
		2	50 DO	14		
Very loose to loose brown silty sand, trace to some gravel, containing rock fragments (FILL)	1.37	3	50 DO	3	○	
		4	50 DO	5	○	
Probable CONCRETE (Existing Footing)	2.74					
Probably weathered Limestone BEDROCK	3.96					
END OF BOREHOLE DUE TO AUGER REFUSAL TO FURTHER AUGERING ON PROBABLE BEDROCK	4.11					Borehole open and dry upon completion of drilling, May 7, 2011

Column headings: DEPTH SCALE METRES; BORING METHOD; SOIL PROFILE (DESCRIPTION, STRATA PLOT, ELEV. DEPTH (m)); SAMPLES (NUMBER, TYPE); ELEVATION; DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat V. + Q - ●, rem V. ⊕ U - ○) (25, 50, 75, 100); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT (Wp —○W— Wl) (5, 10, 15, 20); ADDITIONAL LAB. TESTING; INSTALLATION AND GROUNDWATER OBSERVATIONS

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

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED:

PROJECT: 11-1184-0022

RECORD OF BOREHOLE BH-4

LOCATION: SEE FIGURE 2

BORING DATE: May 7, 2011

SAMPLER HAMMER, 63.5kg; DROP, 760mm

PENETRATION TEST HAMMER, 63.5kg; DROP, 760mm

BORING METHOD: 150 mm Diameter SOLID STEM AUGERS, Track Mounted Power Auger

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	INSTALLATION AND GROUNDWATER OBSERVATIONS
0	GROUND SURFACE					
	ASPHALT (120 mm)					
	GRANULAR BASE	0.12	1	AS	-	
	Brown gravelly sand, trace silty (FILL)	0.52				
1			2	AS	-	
	END OF BOREHOLE REFUSAL TO FURTHER AUGERING ON PROBABLE CONCRETE	1.22				Borehole open and dry upon completion of drilling, May 7, 2011

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 25, 50, 75, 100

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}

WATER CONTENT PERCENT: Wp, W, Wl : 5, 10, 15, 20

ADDITIONAL LAB. TESTING

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

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED:

RECORD OF TEST PIT TP-1

PROJECT: 11-1184-0022

LOCATION: SEE FIGURE 2

EXCAVATION DATE: May 3, 2011

SHEET 1 OF 1

DEPTH SCALE METRES	METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80) / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (25, 50, 75, 100)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}) / WATER CONTENT PERCENT Wp —○W— Wl (5, 10, 15, 20)	ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		GROUND SURFACE									
0	BACKHOE	Brown crushed clear stone (FILL)									
		Brown sandy gravel, trace silt, containing rock fragments, clayey silt pockets, organic inclusions (FILL)		0.10							
1											
2		Weathered Limestone BEDROCK		1.80							Water encountered at a depth of 1.7 m below ground surface, May 3, 2011.
		END OF TEST PIT ON LIMESTONE BEDROCK									
3											
4											
5											
6											
7											
8											
9											
10											

DEPTH SCALE

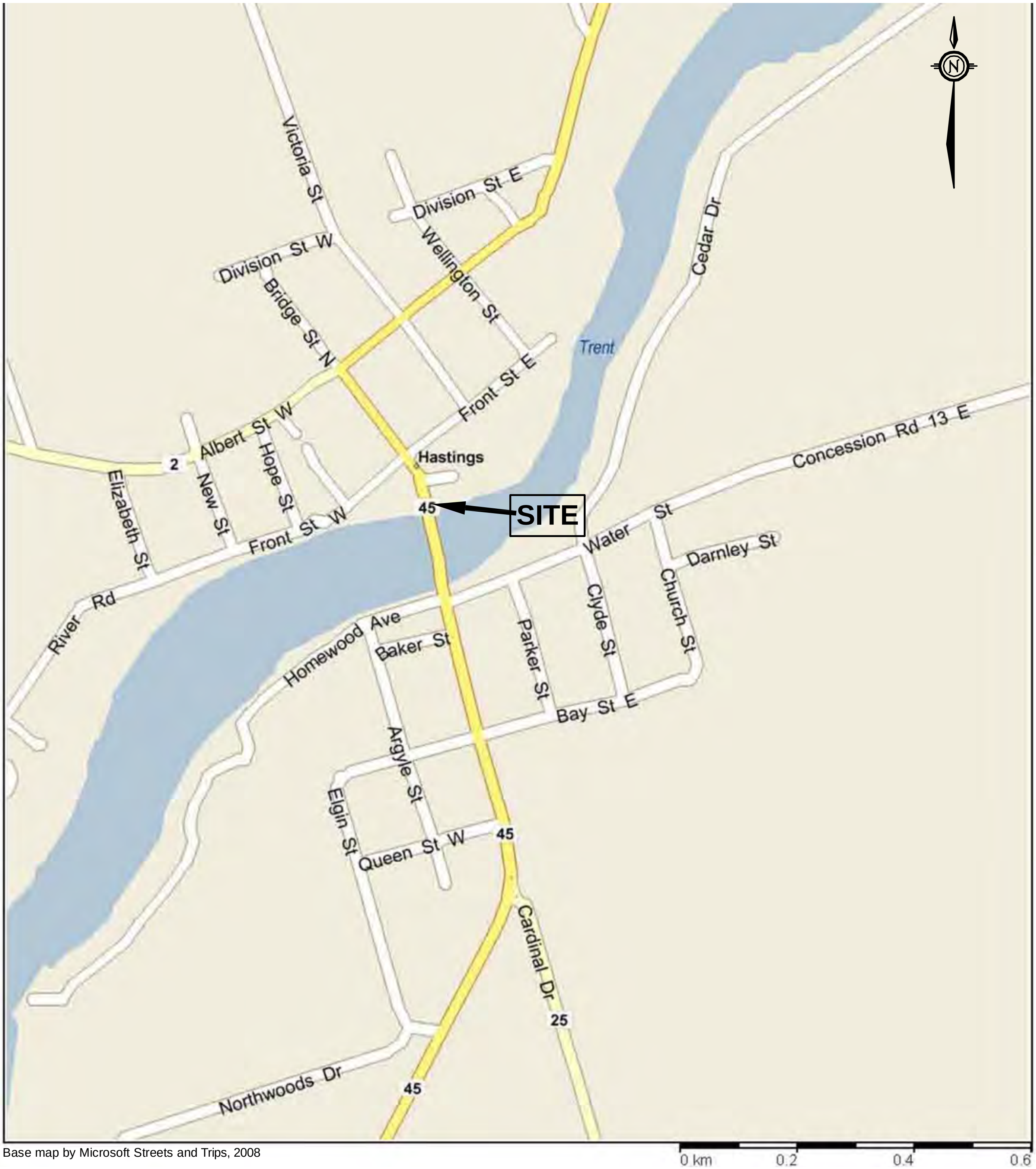
1 : 50

Golder Associates

LOGGED: AZ

CHECKED:

LDN_BHS 11-1184-0022.GPJ GLDR_LDN.GDT 5/30/11 DATA INPUT: MK MAY 2011



Base map by Microsoft Streets and Trips, 2008

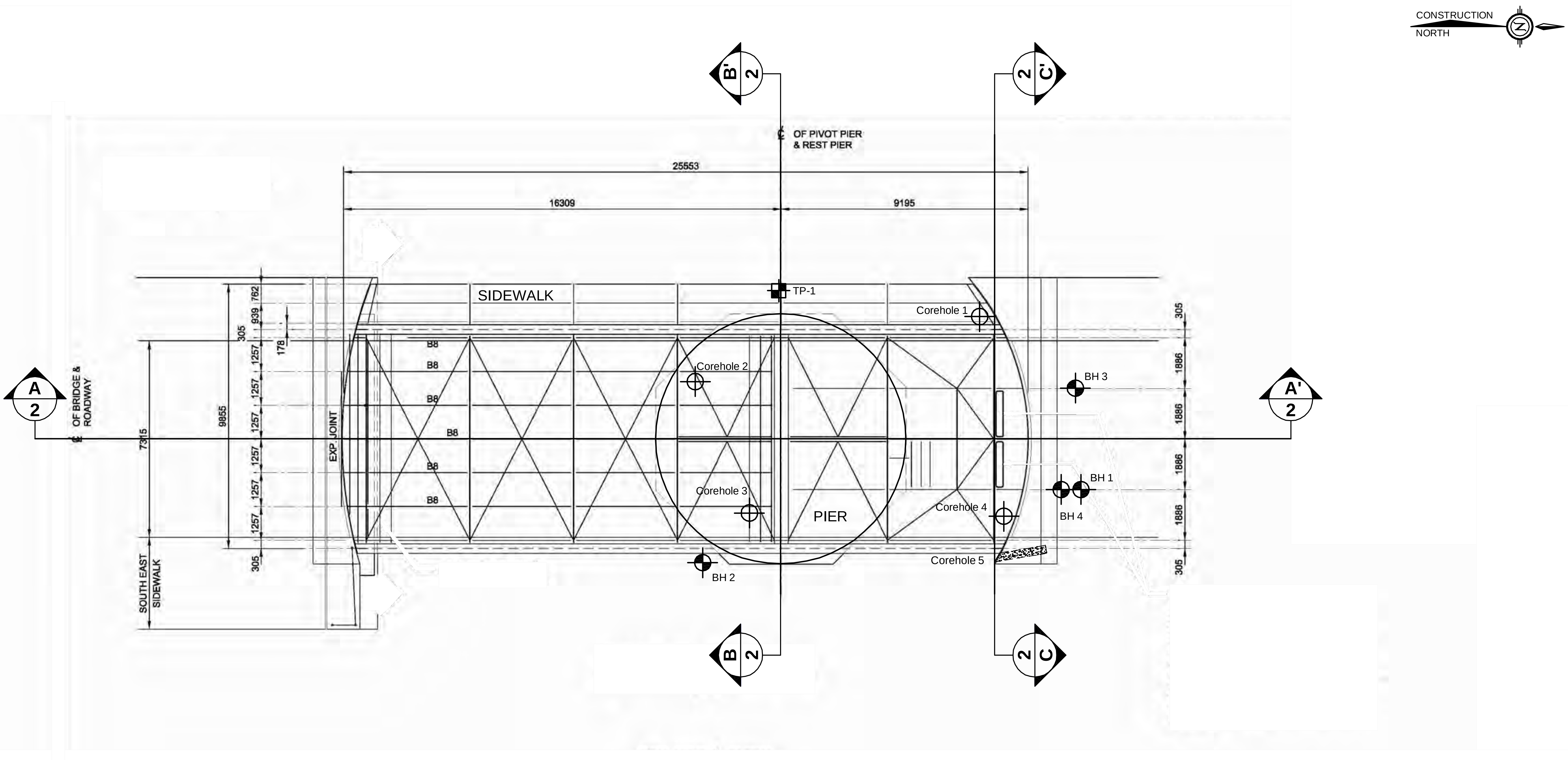
ALL LOCATIONS ARE APPROXIMATE

PROJECT	Delcan Hastings Swing Bridge Trent-Severn Canal, Hastings, Ontario
TITLE	**KEY PLAN**

Golder Associates

PROJECT No. 11-1184-0022			FILE No.	AA01
DESIGN			SCALE AS SHOWN	REV.
CADD	PJV	May 2011	**FIGURE 1**	
CHECK				
REVIEW				

Drawing file: N:\CAD_PROJECTS\2011\11-1184-0022\-AA-\1111840022AA01-kp.dwg May 30, 2011 - 2:16pm



REFERENCE

Base plan provided by Delcan, entitled "Steel Plan and Sections", Drawing No. 101 Sheet 1 of 4, Dated 03/11/2010.

NOT TO SCALE
ALL LOCATIONS ARE APPROXIMATE

LEGEND

- BOREHOLE LOCATION IN PLAN
- COREHOLE LOCATION IN PLAN
- TEST PIT LOCATION IN PLAN

PROJECT
Delcan
Hastings Swing Bridge
Trent-Severn Canal, Hastings, Ontario

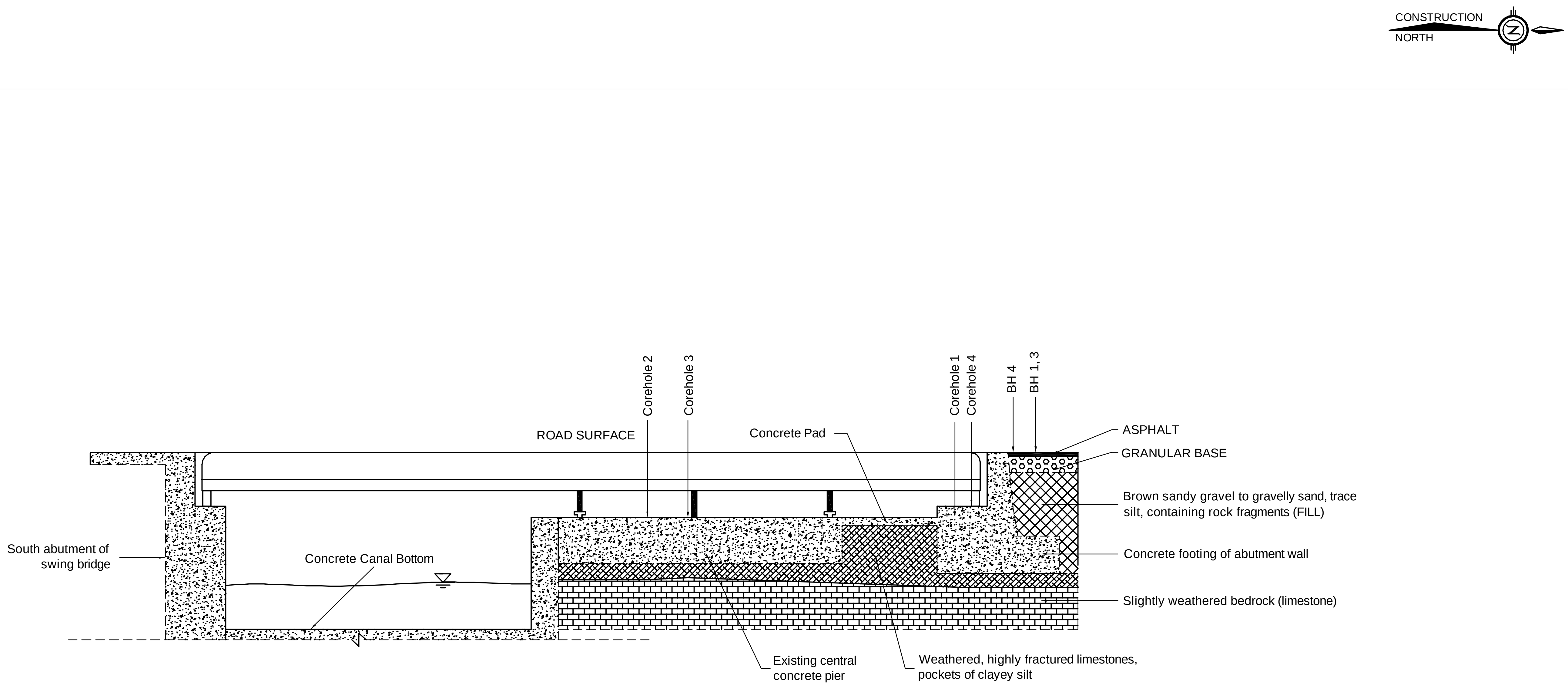
TITLE

BOREHOLE, COREHOLE AND TEST PIT LOCATION PLAN

Golder Associates

PROJECT No. 11-1184-0022			FILE No.	AA01
DESIGN			SCALE AS SHOWN	REV. 0
CADD	MK	MAY 2011		
CHECK				
REVIEW				

FIGURE 2



CROSS SECTION A-A

NOTES

The assumed boundary of the concrete footings and soil strata are based on limited corehole/ borehole data and should be considered as approximate.

NOT TO SCALE
ALL LOCATIONS ARE APPROXIMATE

PROJECT
Delcan
Hastings Swing Bridge
Trent-Severn Canal, Hastings, Ontario

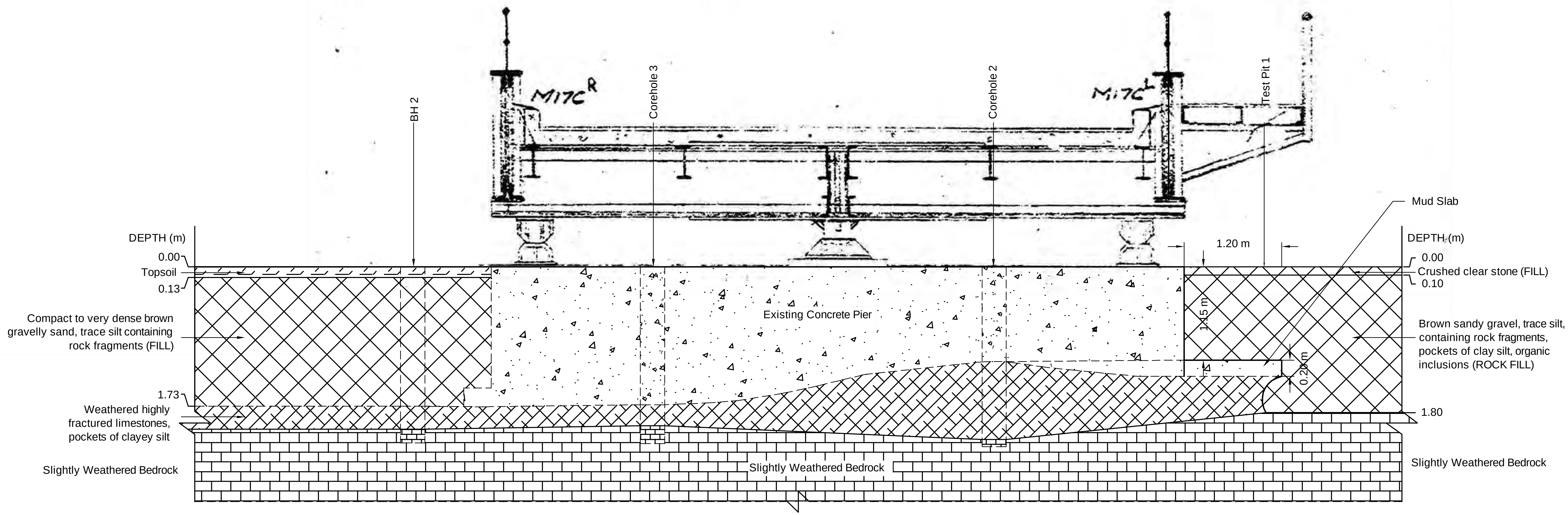
TITLE
CROSS SECTION A - A

Golder Associates

PROJECT No. 11-1184-0022		FILE No. AA01
DESIGN		SCALE AS SHOWN REV. 0
CADD	MK	MAY 2011
CHECK		
REVIEW		

FIGURE 3A

Drawing file: N:\CAD_PROJECTS\2011\11-1184-0022\-AA-\1111840022AA01.dwg May 30, 2011 - 2:15pm



Cross Section B - B

REFERENCE

The cross-section of the bridge structure was created based on the base plan provided by Delcan, entitled "Erection Diagram Department Of Transport Trent Canal Hastings Swing Bridge", Project No. 1710-50, Drawing No. E4, Dated 03/07/1992.

NOTES

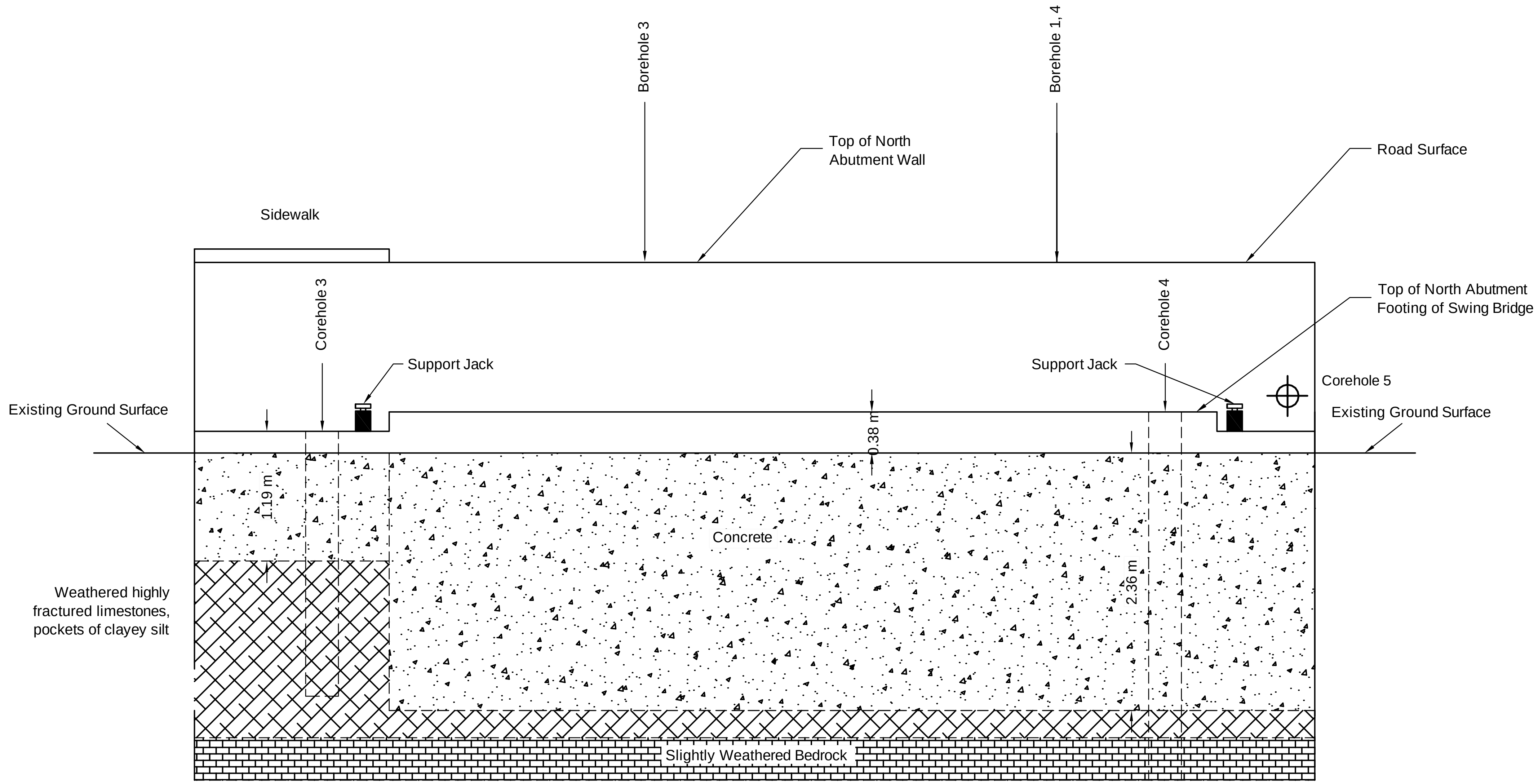
The assumed boundary of the concrete footings, and soil strata are based on limited corehole/ borehole data and should be considered as approximate.

NOT TO SCALE
ALL LOCATIONS ARE APPROXIMATE

PROJECT				
Delcan Hastings Swing Bridge Trent-Severn Canal, Hastings, Ontario				
TITLE				
CROSS SECTION B - B				
PROJECT No. 11-1184-0022			FILE No. AA01	
DESIGN			SCALE AS SHOWN	REV. 0
CADD	MK	MAY 2011	**FIGURE 3B**	
CHECK				
REVIEW				

Golder Associates

Drawing file: N:\CAD_PROJECTS\2011\11-1184-0022\-AA-\1111840022AA01.dwg May 30, 2011 - 2:15pm



Cross Section C - C

NOTES

The assumed boundary of the concrete footings, rockfill and bedrock are based on limited corehole/ borehole data and should be considered as approximate.

NOT TO SCALE
ALL LOCATIONS ARE APPROXIMATE

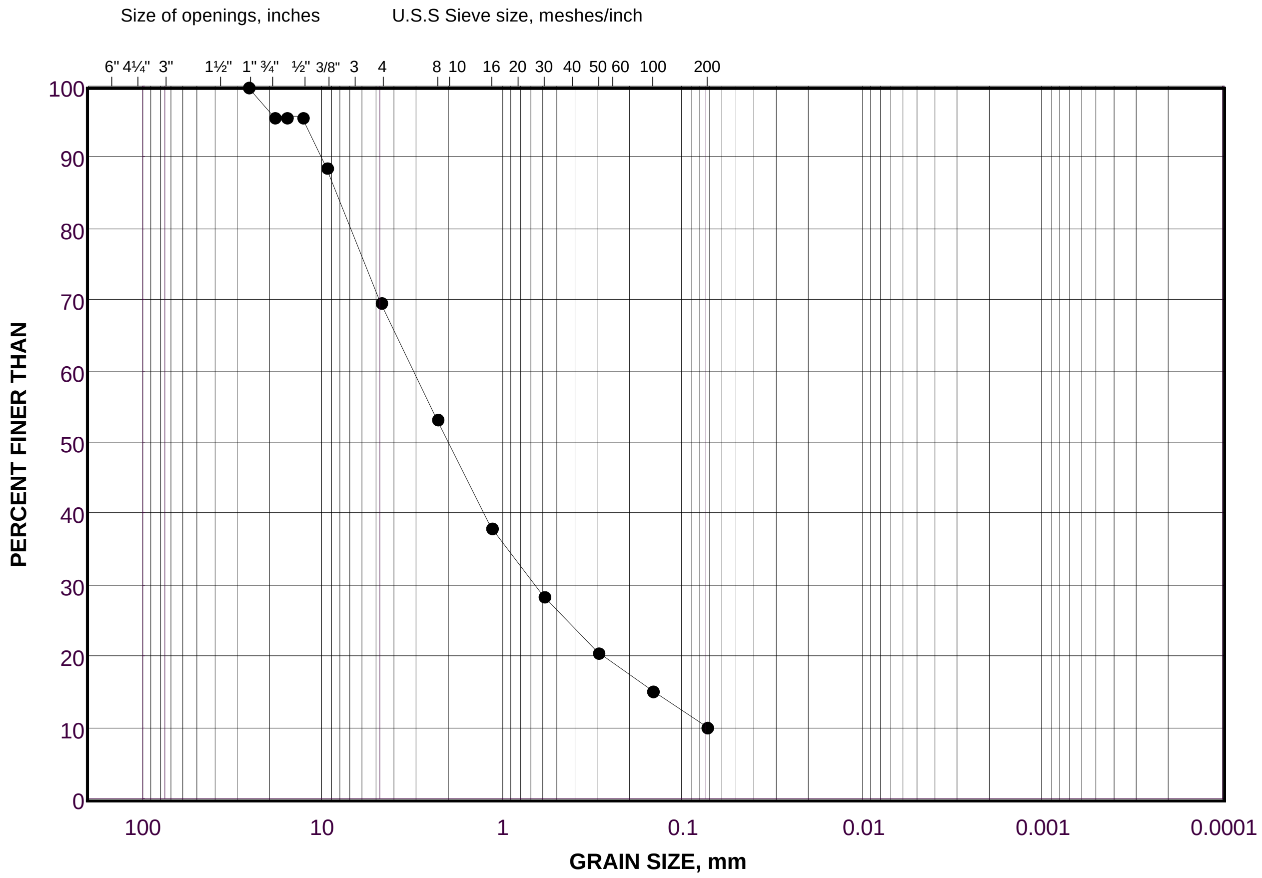
PROJECT	Delcan Hastings Swing Bridge Trent-Severn Canal, Hastings, Ontario
TITLE	**CROSS SECTION C - C**

Golder Associates				
PROJECT No. 11-1184-0022			FILE No.	AA01
DESIGN			SCALE AS SHOWN	REV. 0
CADD	MK	MAY 2011	**FIGURE 3C**	
CHECK				
REVIEW				

GRAIN SIZE DISTRIBUTION

GRAVELLY SAND

FIGURE 4



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	2	2	0.76 - 1.22

Project Number: 11-1184-0022

Checked By: ______________________

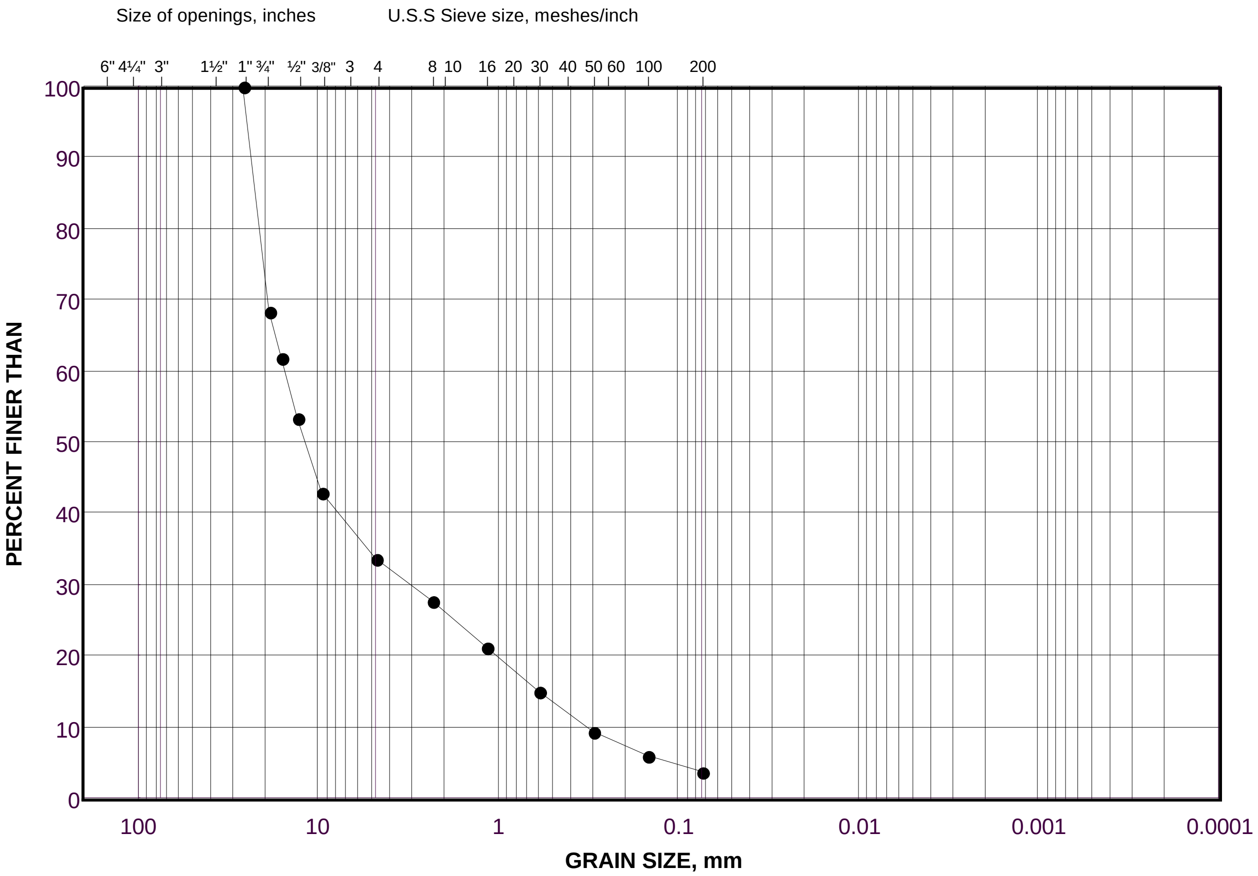
Golder Associates

Date: 30-May-11

GRAIN SIZE DISTRIBUTION

SANDY GRAVEL

FIGURE 5



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	1	4	2.29 - 2.74

Project Number: 11-1184-0022

Checked By: ____________________

Golder Associates

Date: 30-May-11

SITE PHOTOGRAPHS	Figure 6A

No.1: Overview of the Hastings Swing Bridge from north side of bridge, looking southwest.

No.2: Overview of the Hastings Swing Bridge, looking south. The fixed bridge with concrete guard rail is located immediately south of the swing bridge.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6B

No.3: Overview of the Hastings Swing Bridge at the position open to vehicle traffic, looking west, standing on top of the gate of Lock 18.

No.4: Overview of the Hastings Swing Bridge at position open to vehicle traffic, looking east.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6C

No.5: South abutment of swing bridge and south canal concrete wall, looking southeast, taken when the swing bridge was swung away and water in canal was lowered.

No.6: The north canal concrete wall, looking northwest, taken when the swing bridge was swung away and water in canal was lowered.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS

Figure 6D

No.7: Overview of the North Abutment Wall; looking west after the swing bridge was swung away.

No.8: The swing bridge deck, looking south; a combination steel grate and concrete deck.

Project No.	11-1184-0022	**Golder Associates**	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6E

No.9: The north abutment wall and north nosing, looking west. Note the recent repair south of the steel nosing plate on the bridge deck; note the deteriorateation on top of the abutment wall and asphalt patch repair.

No.10: The south abutment wall and south nosing, looking west. Note the recent repair (lighter colour concrete) south of the steel plate on the abutment wall.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6F

No.11: The south abutment underneath the bridge deck, looking south; note the deterioration of the concrete, exposed steel rebars; note the snow, salt and sands or soils falling from the steel grate deck.

No.12: The south abutment underneath the bridge deck, looking south; note the deterioration of the concrete, exposed steel rebars at or immediate below the abutment slab; also note the nearly horizontal deterioration of the concrete at lower portion of the south canal wall.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6G

No.13: The south abutment underneath the bridge deck, looking west; note the snow, salt and sands or soils falling from the steel grate deck; note two steel posts used to support the bridge during the non-operational seasons.

No.14: The south abutment, looking west; note the steel wheel sitting on top of the steel plate to support the bridge during the operational seasons.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

No.15: The north abutment underneath the bridge deck, looking east; note two steel posts used to support the bridge during the non-operational seasons.

No. 16: The north abutment west side of the bridge, looking northeast; note the support hydraulic jack used to support the bridge during the operational seasons.

SITE PHOTOGRAPHS	Figure 6I

No.17: The north abutment east side of the bridge, looking northwest; note the support hydraulic jack used to support the bridge during the operational

No.18: The central concrete pivot pier; note that the concrete at the surface of the pier appeared to be recently resurfaced; note the hydraulic system and the hydraulic jack used to swing the bridge.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6J

No.19: The central concrete pier; note the circular steel track on top of the concrete pier, the balance wheel on the track and steel pillow block rest to support the transverse beam of the swing bridge.

No.20: The north canal concrete wall; note the deterioration of concrete on the wall surface and exposed and eroded steel rebar and the void underneath the steel track on top of the central pier.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS

Figure 6K

No.21: Central pintle with a round steel plate (Approx. Dia. 0.8m) bolted on concrete platform slightly elevated above the pier concrete.

No.22: The west portion of the swing bridge abutment wall (the portion for the sidewalk and stairs). Note the concrete wall was poured in different years and the cracks between different pours.

Project No.	11-1184-0022	**Golder Associates**	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6L

No.23: A close look of the west portion of the swing bridge abutment wall (the portion for the sidewalk and stairs). Note the concrete wall was poured in different years and the cracks between different pours; note the drainage hole on the wall.

No.24: The northeast of the swing bridge; note the nearly-straight-line crack from the edge of the wing wall extending west to a distance of about 1.5 m.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6M

No.25: The retaining wall on the northwest side of the swing bridge; note the upper portion of the concrete was poured in different years and cracks between the different pours; note the deterioration of lower portion concrete.

No.26: Location of the Corehole 1.

.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6N

No.27: Location of Corehole 2.

No. 28: Locations of Coreholes 4 and 5.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6O

No.29: Photograph of the Corehole 1 cores.

No.30: Photograph of the Corehole 2 cores.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6P

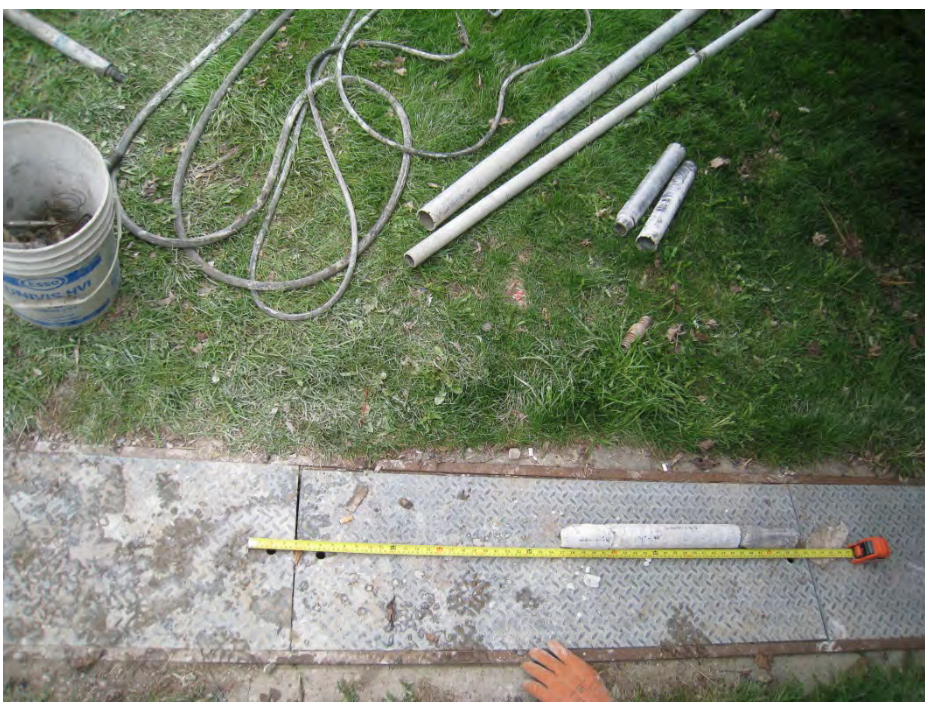
No.31: Photograph of the Corehole 3 cores.

No.32: Photograph of the Corehole 4 cores.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS	Figure 6Q

No.33: Photograph of the Corehole 5 cores.



No.34: Photograph of the Corehole 5 cores.

Project No.	11-1184-0022	Golder Associates	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

SITE PHOTOGRAPHS

Figure 6R

No.35: Photograph of the Test Pit 1.

No.36: Photograph of the Test Pit 1. The limestone bedrock exposed at the bottom of the test pit.

Project No.	11-1184-0022	**Golder Associates**	Inputted by:	AZ
Date:	May, 2011		Checked by:	DL

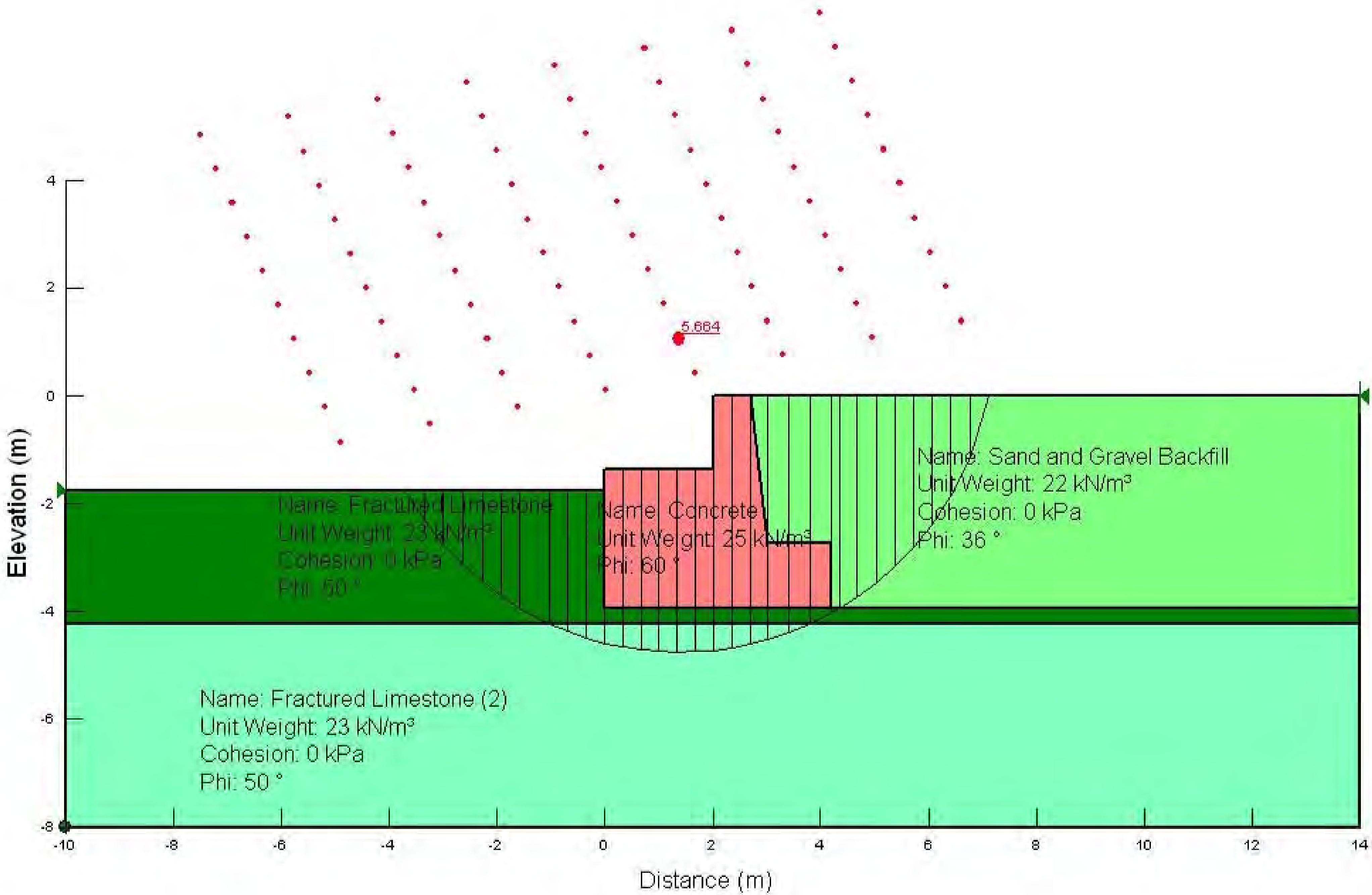


Figure 7- Global Stability Analysis of the North Abutment Wall

APPENDIX A

Important Information and Limitations of This Report

IMPORTANT INFORMATION AND LIMITATIONS TO 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. 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 can not be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

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.

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.

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.

APPENDIX B

Results of Laboratory Compressive Testing for Rock Cores and Concrete Cores

OBTAINING AND TESTING DRILLED CORES FOR COMPRESSIVE STRENGTH TESTING (CSA A23.2-14C)

Job Number: 11-1184-0022

ATTENTION: Mr. Steve Jagdat

Project	**Hastings Swing Bridge, Hastings, ON**

Date Received: May 30, 2011 **Date Tested:** May 30, 2011

Core Number	Sa1	Sa2	Sa1	Sa2
Location	Hole 2	Hole 2	Hole 3	Hole 3
Golder Lab Number	C-11-571	C-11-572	C-11-573	C-11-574
Moisture Condition at time of Test	Dry	Dry	Dry	Dry
Capping Materials	Sulphur	Sulphur	Sulphur	Sulphur
Capped Height (mm)	114.0	113.5	112.0	114.0
Average Diameter (mm)	57.0	57.0	57.0	57.0
Density (Mg/m^3)	2.413	2.399	2.260	2.459
Load (kN)	126.47	140.74	78.93	123.80
Compressive Strength (MPa)	49.6	55.2	30.9	48.5
Corrected Compressive Strength (MPa)	49.6	55.1	30.8	48.5
Remarks:				

Reviewed by: ______________________

Jeremy Rose, Laboratory Manager

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1184-0022	SAMPLE NUMBER	-
CORE HOLE	2	SAMPLE DEPTH, m	2.0-2.1

TEST CONDITIONS

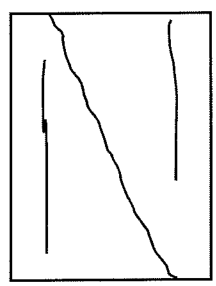
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	1.55

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	8.79	WATER CONTENT, (specimen) %	0.13
SAMPLE DIAMETER, cm	5.68	UNIT WEIGHT, kN/m^3	26.24
SAMPLE AREA, cm^2	25.32	DRY UNIT WT., kN/m^3	26.21
SAMPLE VOLUME, cm^3	222.47	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	595.60	VOID RATIO	0.01
DRY WEIGHT, g	594.83		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	151.1

REMARKS: L/D Ratio not in accordance with ASTM Standard

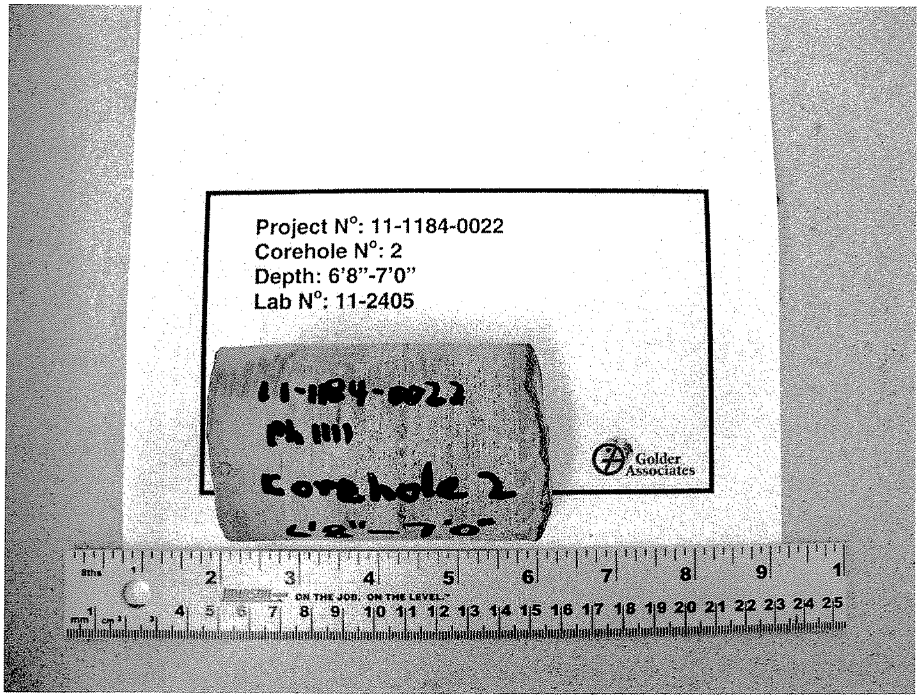
DATE: 5/26/2011

Checked By: MM

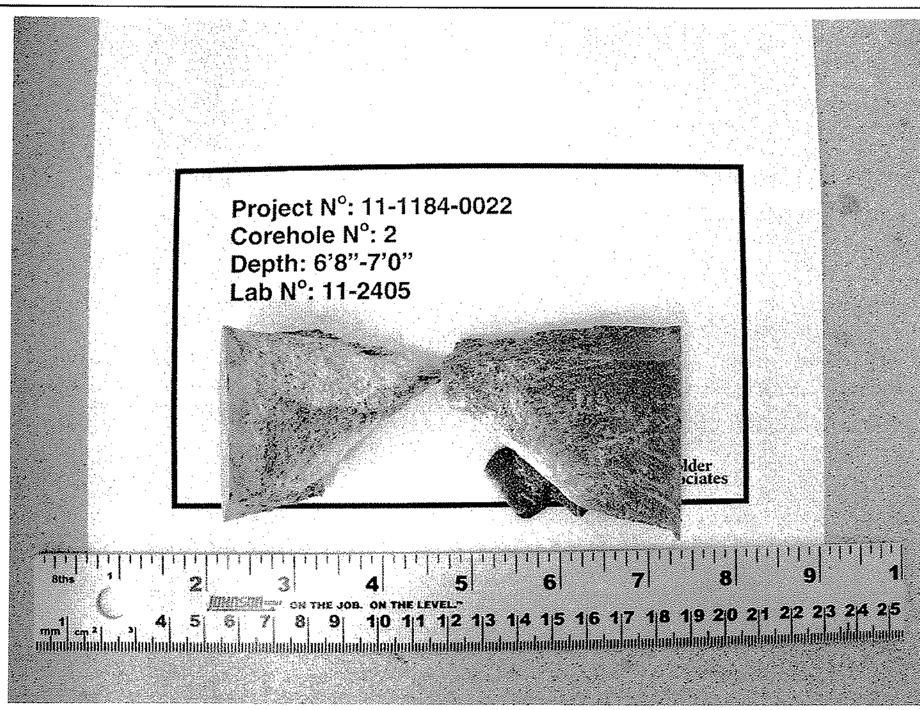
Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 5/26/2011

Project 11-1184-0022

Golder Associates

Drawn AH

Chkd. MM

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1184-0022	SAMPLE NUMBER	-
CORE HOLE	4	SAMPLE DEPTH, m	2.8-3.0

TEST CONDITIONS

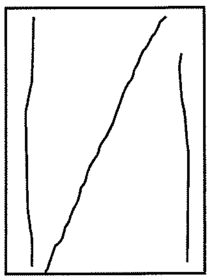
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.36

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.24	WATER CONTENT, (specimen) %	0.19
SAMPLE DIAMETER, cm	5.60	UNIT WEIGHT, kN/m^3	26.19
SAMPLE AREA, cm^2	24.63	DRY UNIT WT., kN/m^3	26.14
SAMPLE VOLUME, cm^3	326.00	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	871.00	VOID RATIO	0.01
DRY WEIGHT, g	869.35		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	90.3

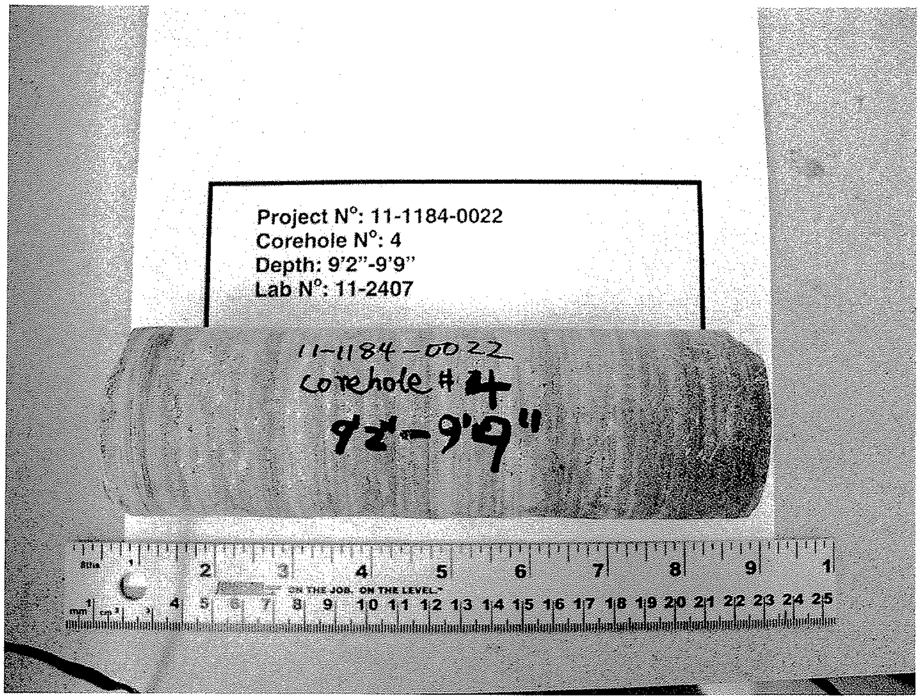
REMARKS:

DATE: 5/26/2011

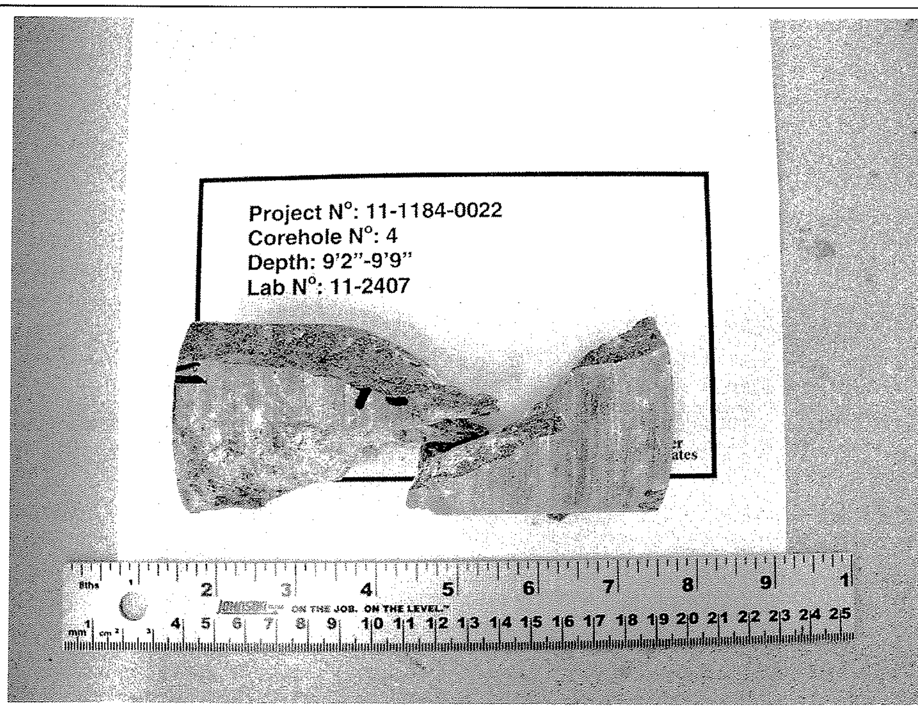
Checked By: [signature]

Golder Associates

UNCONFINED COMPRESSION TEST ASTM D7012-07	FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 5/26/2011
Project 11-1184-0022

Golder Associates

Drawn AH
Chkd. [signature]

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
100, Scotia Court
Whitby, Ontario, L1N 8Y6
Canada
T: +1 (905) 723 2727

Golder Associates