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PWGSC ONTARIO	SPECIFICATION	SECTION 00 00 00
REGION PROJECT	TITLE SHEET	PAGE 1
NUMBER R.055746.001		2014-10-01

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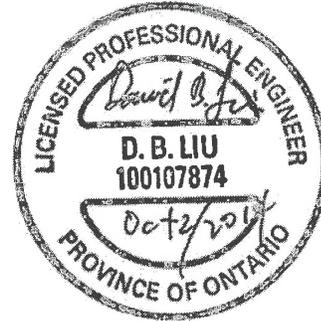
PROJECT TITLE : HASTING SWING BRIDGE PIVOT PIER FOAM INJECTION

PROJECT NUMBER R.0557746.001

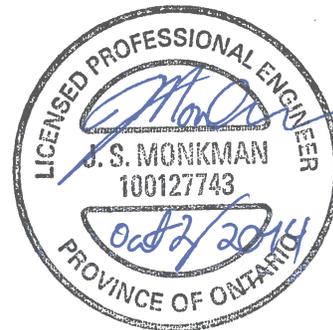
PROJECT DATE 2014-10-01

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Consultant for Building Code Review:



Building Code Designation Number (BCDN):



PART 1 - GENERAL

1.1 WORK COVERED BY CONTRACT DOCUMENTS

- .1 Work of this Contract comprises foam injection under the pivot pier and north abutment of the Hasting Swing Bridge , located in the Town of Hastings Ontario.

1.2 CONTRACT METHOD

- .1 Construct work under lump sum price contract.
- .4 Relations and responsibilities between Contractor and subcontractors assigned by Departmental Representative are as defined in Conditions of Contract.

1.3 COST BREAKDOWN

- .1 Within 48 hours of notification of acceptance of bid furnish a cost breakdown by stages of 1.4 work sequence aggregating contract price.
- .2 Within 48 hours of acceptance of bid submit a list of subcontractors.

1.4 WORK SEQUENCE

- .1 Construct Work in stages to accommodate Owner's intermittent use of premises during construction.
- .2 Coordinate Progress Schedule and coordinate with Owner Occupancy during construction.
- .3 Required stages:
  - .1 Mobilization to site.
  - .2 Drill holes and inject foam under pivot pier.
  - .3 Drill holes and inject foam under north abutment.
  - .4 Core test foam injected areas to verify adequacy of work.
  - .5 Undertake additional foam injection works until determined to be adequate if initial work is found to be inadequate.
  - .6 Demobilization off site
- .4 Maintain fire access/control.
- .5 Contractor to maintain work zone free of standing water by dewatering as required throughout the entire construction period.

1.8 CONTRACTOR USE OF PREMISES

- .1 Contractor has unrestricted use of site until Substantial Performance.

- .2 Coordinate use of premises under direction of Departmental Representative.
- .3 Obtain and pay for use of additional storage or work areas needed for operations under this Contract.

PART 2 - PRODUCTS

2.1 NOT USED

- .1 Not used.

PART 3 - EXECUTION

3.1 NOT USED

- .1 Not used.

## PART 1 - GENERAL

### 1.1 REFERENCES

- .1 Canadian Standards Association (CSA): Canada
  - .1 CSA S350-M1980(R2003), Code of Practice for Safety in Demolition of Structures.
- .2 National Building Code 2010 (NBC):
  - .1 NBC 2010, Division B, Part 8 Safety Measures at Construction and Demolition Sites.
- .3 National Fire Code 2010 (NFC):
  - .1 NFC 2010, Division B, Part 5 Hazardous Processes and Operations, subsection 5.6.1.3 Fire Safety Plan.
- .4 Province of Ontario:
  - .1 Occupational Health and Safety Act Revised Statutes of Ontario 1990, Chapter O.1 as amended, and Regulations for Construction Projects, O. Reg. 213/91 as amended.
  - .2 O. Reg. 490/09, Designated Substances.
  - .3 Workplace Safety and Insurance Act, 1997.
  - .4 Municipal statutes and authorities.
- .5 Treasury Board of Canada Secretariat (TBS):
  - .1 Treasury Board, Fire Protection Standard April 1, 2010  
[www.tbs-sct.gc.ca/pol/doc-eng.aspx?id=17316&section=text](http://www.tbs-sct.gc.ca/pol/doc-eng.aspx?id=17316&section=text).

### 1.2 ACTION AND INFORMATIONAL SUBMITTALS

- .1 Submit in accordance with Section 01 11 01.
- .2 Submit site-specific Health and Safety Plan: Within 7 days after date of Notice to Proceed and prior to commencement of Work. Health and Safety Plan must include:
  - .1 Results of site specific safety hazard assessment.
  - .2 Results of safety and health risk or hazard analysis for site tasks and operation.
  - .3 Measures and controls to be implemented to address identified safety hazards and risks.
- .3 Contractor's and Sub-contractors' Safety Communication Plan.
- .4 Contingency and Emergency Response Plan addressing standard operating procedures specific to the project site to be implemented during emergency situations. Coordinate plan with existing Emergency Response requirements and procedures provided by Departmental Representative.
- .5 Departmental Representative will review Contractor's site-specific Health and Safety Plan and provide comments to Contractor within 5 days after receipt of plan. Revise plan as appropriate and resubmit plan to Departmental Representative within 5 days after receipt of comments from Departmental

Representative.

- .6 Departmental Representative's review of Contractor's final Health and Safety plan should not be construed as approval and does not reduce the Contractor's overall responsibility for construction Health and Safety.
- .7 Submit names of personnel and alternates responsible for site safety and health.
- .8 Submit records of Contractor's Health and Safety meetings when requested.
- .9 Submit 3 copies of Contractor's authorized representative's work site health and safety inspection reports to Departmental Representative weekly.
- .10 Submit copies of orders, directions or reports issued by health and safety inspectors of the authorities having jurisdiction.
- .11 Submit copies of incident and accident reports.
- .12 Submit Material Safety Data Sheets (MSDS).
- .13 Submit Workplace Safety and Insurance Board (WSIB)- Experience Rating Report.

### 1.3 FILING OF NOTICE

- .1 File Notice of Project with Provincial authorities prior to commencement of Work.
- .2 Contractor shall agree to install proper site separation and identification in order to maintain time and space at all times throughout life of project.

### 1.4 SAFETY ASSESSMENT

- .1 Perform site specific safety hazard assessment related to project.

### 1.5 MEETINGS

- .1 Schedule and administer Health and Safety meeting with Departmental Representative prior to commencement of Work.

### 1.6 REGULATORY REQUIREMENTS

- .1 Comply with the Acts and regulations of the Province of Ontario.
- .2 Comply with specified standards and regulations to ensure safe operations at site.

### 1.7 PROJECT/SITE CONDITIONS

- .1 Work at site will involve contact with:
  - .1 Silica in concrete.
  - .2 Lead in paint, flashing, solder in electronic equipment, solder caulking in ball fittings of cast iron pipes, vent and pipe flashings.
  - .3 Benzene in fuel oil, paints and adhesives.

- .4 Guano on steel sections, tops of pier and abutments.
  - .5 Working near and under overhead electrical wires.
  - .6 Working near and around water.
  - .7 Arsenic and acrylonitrile in paints and adhesives.
  - .8 Vinyl chloride in pipes, conduits and interior finishes.
  - .9 Working at height near a lock.
  - .10 Working on or adjacent to moveable equipment (i.e movable bridge).
  - .11 Electrocution.
  - .12 Unpredictable water currents in canal entrance.
  - .13 Vehicular traffic
  - .14 Exposure to boat traffic during navigation season (navigation closed October 15, 2015 to May 24,2016
- .2 Confined spaces in crawl spaces.

## 1.8 GENERAL REQUIREMENTS

- .1 Develop written site-specific Health and Safety Plan based on hazard assessment prior to beginning site Work and continue to implement, maintain, and enforce plan until final demobilization from site. Health and Safety Plan must address project specifications.
- .2 Departmental Representative may respond in writing, where deficiencies or concerns are noted and may request re-submission with correction of deficiencies or concerns either accepting or requesting improvements.
- .3 Relief from or substitution for any portion or provision of minimum Health and Safety standards specified herein or reviewed site-specific Health and Safety Plan shall be submitted to Departmental Representative in writing.

## 1.9 COMPLIANCE REQUIREMENTS

- .1 Comply with Ontario Occupational Health and Safety Act, R.S.O. 1990 Chapter 0.1, as amended.

## 1.10 RESPONSIBILITY

- .1 Be responsible for health and safety of persons on site, safety of property on site and for protection of persons adjacent to site and environment to extent that they may be affected by conduct of Work.
- .2 Comply with and enforce compliance by employees with safety requirements of Contract Documents, applicable federal, provincial, territorial and local statutes, regulations, and ordinances, and with site-specific Health and Safety Plan.
- .3 Where applicable the Contractor shall be designated "Constructor", as defined by Occupational Health and Safety Act and Regulations for Construction Projects for the Province of Ontario.

### 1.11 UNFORSEEN HAZARDS

- .1 Should any unforeseen or peculiar safety-related factor, hazard, or condition become evident during performance of Work, immediately stop work and advise Departmental Representative verbally and in writing.
- .2 Follow procedures in place for Employees Right to Refuse Work as specified in the Occupational Health and Safety Act for the Province of Ontario.

### 1.12 POSTING OF DOCUMENTS

- .1 Ensure applicable items, articles, notices and orders are posted in conspicuous location on site in accordance with Acts and Regulations of Province of Ontario, and in consultation with Departmental Representative.
  - .1 Contractor's Safety Policy.
  - .2 Constructor's Name.
  - .3 Notice of Project.
  - .4 Name, trade, and employer of Health and Safety Representative or Joint Health and Safety Committee members (if applicable).
  - .5 Ministry of Labour Orders and reports.
  - .6 Occupational Health and Safety Act and Regulations for Construction Projects for Province of Ontario.
  - .7 Address and phone number of nearest Ministry of Labour office.
  - .8 Material Safety Data Sheets.
  - .9 Written Emergency Response Plan.
  - .10 Site Specific Safety Plan.
  - .11 Valid certificate of first aider on duty.
  - .12 WSIB "In Case of Injury At Work" poster.
  - .13 Location of toilet and cleanup facilities.

### 1.13 CORRECTION OF NON-COMPLIANCE

- .1 Immediately address health and safety non-compliance issues identified by authority having jurisdiction or by Departmental Representative.
- .2 Provide Departmental Representative with written report of action taken to correct non-compliance of health and safety issues identified.
- .3 Departmental Representative may stop Work if non-compliance of health and safety regulations is not corrected.

### 1.14 BLASTING

- .1 Blasting or other use of explosives is not permitted.

### 1.15 POWDER ACTUATED DEVICES

- .1 Use powder actuated devices only after receipt of written permission from Departmental Representative.

1.16 WORK STOPPAGE

- .1 Give precedence to safety and health of public and site personnel and protection of environment over cost and schedule considerations for Work.
- .2 Assign responsibility and obligation to Competent Supervisor to stop or start Work when, at Competent Supervisor's discretion, it is necessary or advisable for reasons of health or safety. Departmental Representative may also stop Work for health and safety considerations.

PART 2 - PRODUCTS

2.1 NOT USED

- .1 Not used.

PART 3 - EXECUTION

3.1 NOT USED

- .1 Not used.

## PART 1 - GENERAL

### 1.1 DESCRIPTION

- .1 This Section describes requirements for the protection of the environment that apply to the Work. These requirements apply to all Sections of this Specification, without limiting the conditions and approvals imposed by statute.
- .2 Control Work to provide effective environmental, waterbody, and fish habitat protection. Departmental Representative will monitor environmental protection measures and will identify whenever such protection is found to be ineffective. Change protective measures or work procedures as directed by Departmental Representative to ensure environmental, waterbody and fish habitat protection.

### 1.2 SUBMITTALS

- .1 Submittals: in accordance with Section 01 11 01.
- .2 Prior to commencing construction activities or delivery of materials to site, submit Environmental Protection Plan for review and approval by Departmental Representative. Environmental Protection Plan shall present a comprehensive overview of known or potential environmental issues which must be addressed during construction.
- .3 Address topics at level of detail commensurate with environmental issue and required construction tasks.
- .4 Environmental protection plan shall include:
  - .1 Names of persons responsible for ensuring adherence to Environmental Protection Plan.
  - .2 Names and qualifications of persons responsible for manifesting hazardous waste to be removed from site.
  - .3 Names and qualifications of persons responsible for training site personnel.
  - .4 Descriptions of environmental protection personnel training program.
  - .5 Work area plan showing proposed activity in each portion of area and identifying areas of limited use or non-use. Plan shall include measures for marking limits of use areas.
  - .5 Including methods for protection of features to be preserved within authorized work areas.
  - .6 Spill Control Plan: including procedures, instructions, and reports to be used in event of unforeseen spill of regulated substance.
  - .7 Non-Hazardous solid waste disposal plan identifying methods and locations for solid waste disposal including clearing debris.
  - .8 Air pollution control plan detailing provisions to assure that dust, debris, materials, and trash, do not become air borne and travel off project site.
  - .9 Contaminant prevention plan that: identifies potentially hazardous substances to be used on job site; identifies intended actions to prevent introduction of such materials into air, water, or ground; and

- details provisions for compliance with Federal, Provincial, and Municipal laws and regulations for storage and handling of these materials.
- .10 Waste water management plan that identifies methods and procedures for management and/or discharge of waste waters which are directly derived from construction activities.
  - .11 Historical, archaeological, cultural resources and biological resources that defines procedures for identifying and protecting historical, archaeological, cultural resources, and biological resources.

### 1.3 FIRES

- .1 Do not burn rubbish or light other fires on site.

### 1.4 DEFINITIONS

- .1 "Deleterious Material" - any substance that, if added to a waterbody, could degrade water quality or impact fish, fish habitat and aquatic wildlife. This includes, but is not limited to:
  - .1 Concrete dust.
  - .2 Soils (clay, silt, sand).
  - .3 Oil, diesel, or gasoline.
  - .4 Chipped or fresh concrete and admixtures.
  - .5 Alkali water resulting from fresh cementitious grout.
  - .6 Salt.
  - .7 Solvents.

### 1.5 TURBIDITY CONTROL AND DRAINAGE WATER RIVER

- .1 Control turbidity of all water released during the Work.
- .2 Do not pump water directly into the canal or
- .3 Control disposal or runoff of water containing other harmful substances in accordance with local authority requirements.
- .4 Sediment, debris and erosion control measures must be inspected daily to ensure that they are functioning properly and are maintained and upgraded as required.

### 1.6 WORK ADJACENT TO CANAL

- .1 Do not release any deleterious material into canal or river.
- .2 Do not dump excavated material, waste material or debris in canal or river.
- .3 Stockpile excavated or fill materials must be stored and stabilized away from the water. Runoff from the excavated or fill material must be contained from entering the canal or river.

### 1.7 SEDIMENT, DUST AND EROSION PROTECTION

- .1 Before starting work that will create dust or debris, (such as sawing and excavation of concrete, etc.), install effective mitigation techniques for sediment, dust, debris and erosion control to the satisfaction of Departmental Representative. Maintain these protective measures at all times, including

shut down periods.

- .2 Cover or wet down dry materials and rubbish to prevent blowing dust and debris.

#### 1.8 OPERATION AND MAINTENANCE OF EQUIPMENT

- .1 Provide drip trays to prevent the discharge of oil, grease, antifreeze, or any other equipment materials into the ground.
- .2 Equipment and heavy machinery used shall meet or exceed all applicable emission requirements.
- .3 Leave machinery running only while in actual use.
- .4 Conduct all vehicle/equipment maintenance and refueling over impermeable/absorptive material situated at a site that is located at least 10 m away from the water.

#### 1.9 REMOVED MATERIALS

- .1 Unless otherwise specified, materials designated for removal become the Contractor's property. Remove these from site.

#### 1.10 HAZARDOUS MATERIALS

- .1 Place materials defined as hazardous or toxic Materials waste in designated containers.
- .2 Comply with the requirements of the Workplace Hazardous Materials Information System (WHMIS) regarding use, handling, storage, and disposal of hazardous materials; and regarding labelling and the provision of Material Safety Data Sheets (MSDS) acceptable to Human Resources Development Canada, Labour Program.
- .3 Store hazardous materials in secure areas on impermeable pads, provide berms if necessary.
- .4 Hazardous materials on site which will be encountered are lead in paint and silica in the form of dust during concrete removal.

#### 1.11 CLEAN UP

- .1 Clean up work area as work progresses. At the end of each work period, and more often if ordered by the Departmental Representative, remove debris from site, neatly stack material for use, and clean up generally.
- .2 Permit no undue amounts of debris, trash or garbage to accumulate.
- .3 Do not bury rubbish on site.
- .4 Separate and recycle all materials that can be recycled.
- .5 Dispose of waste or volatile materials, such as mineral spirits or oil by taking them to a special designated waste facility. Do not dump these into water, storm or sanitary sewers.

- .6 Ensure all emptied containers are sealed and stored safely for disposal away from children.
- .7 Spills:
  - .1 Report all spills immediately to the Departmental Representative and to the Ontario Spills Action Centre (Telephone No.1-800-268-6060).
  - .2 Using appropriate safety precautions, collect liquid or solidify liquid with an inert, noncombustible material and remove for disposal.
  - .3 Be responsible for all costs of cleaning up any spills to the satisfaction of the Departmental Representative.
  - .4 Have an environmental emergency response plan in place and a spill kit readily available.
- .8 Clean areas under contract to a condition at least equal to that previously existing and to approval of Departmental Representative.

#### 1.12 TRANSPORTING WASTE MATERIALS

- .1 All waste subject to Regulation 558 of the Ontario Environmental Protection Act must be transported with a valid "Certificate of Approval for a Waste Management System" to a site approved by the Ontario Ministry of the Environment to accept that waste.
- .2 Be responsible for obtaining all Waste Generator Numbers, permits, manifests, and all other paperwork necessary to comply.

#### 1.13 NOISE CONTROL

- .1 Minimize the noise levels from construction activities by using proper muffling devices, in addition to appropriate timing and location of these activities to reduce or minimize the effect of noise on nearby residents, recreationists, and wildlife.
- .2 Comply with any local or municipal Noise By-Laws.

#### 1.14 SPILLS OR RELEASE OF DELETERIOUS SUBSTANCE

- .1 Immediately contain, limit spread and clean up in accordance with provincial regulatory requirements.
- .2 All workers shall be fully aware of the spill prevention and response procedures including notification of Departmental Representative.
- .3 The Ontario Ministry of Environment Spills Action Centre must be notified immediately by law at 1-800-268-6060.
- .4 The Departmental Representative shall be immediately informed of all spills that occur onsite.
- .5 Further information on dangerous goods emergency cleanup and precautions including a list of companies performing this work can be obtained from the Transport Canada 24-hour number (613) 996-6666 collect.
- .6 Spill kits will be kept on-site during all project phases.

- .7 Contractor shall take due care to ensure no deleterious materials including sediment-laden runoff leave the worksite, or enter any: surface water, storm water, or sanitary sewers at or near the worksite.
- .8 Equipment fuelling or lubricating shall occur in a designated area with proper controls to prevent the release of deleterious substances, and shall be conducted away from any surface water drains or collection points.
- .9 In accordance with the Fisheries Act, approval must be obtained from DFO for use of any paints, corrosion protective coatings, wood preservatives or any other hazardous material that will be applied to surfaces that will have contact with the marine environment.
- .10 Any equipment remaining on site overnight shall have appropriately placed drip pans.
- .11 The rinse, cleaning water or solvents for glues, wood preservatives and other potentially harmful or toxic substances should be controlled so as to prevent leakage, loss or discharge into the storm drain system or into the marine environment.
- .12 Protect the roadways from tracking of mud, soil, and debris throughout the work.
- .13 Prevent discharges containing asphalt, grout, concrete or other waste materials from reaching storm drains or the marine environment. This includes, but is not limited to:
  - .1 Minimizing the washing of sand or gravel from new asphalt, debris from drilling or cutting or other materials into storm drains and the marine environment by sweeping.
  - .2 Application of fog seals, tack coats or other coatings, if required, during periods when rainfall is unlikely to occur during application.
  - .3 Cleaning equipment off site.
  - .4 Protection of drainage structures with filter fences if required.
- .14 Spill or Leak - according to supplier's MSDS

## PART 2 MATERIALS

- .1 Not used.

## PART 3 - EXECUTION

### 3.1 NOT USED

- .1 Not used.

## PART 1 - GENERAL

### 1.1 SECTION INCLUDES

- .1 Inspection and testing, administrative and enforcement requirements.
- .2 Tests and mix designs.
- .3 Equipment and system adjust and balance.

### 1.2 INSPECTION

- .1 Allow Departmental Representative access to Work. If part of Work is in preparation at locations other than Place of Work, allow access to such Work whenever it is in progress.
- .2 Give timely notice requesting inspection if Work is designated for special tests, inspections or approvals by Departmental Representative instructions, or law of Place of Work.
- .3 If Contractor covers or permits to be covered Work that has been designated for special tests, inspections or approvals before such is made, uncover such Work, have inspections or tests satisfactorily completed and make good such Work.
- .4 Departmental Representatives may order any part of Work to be examined if Work is suspected to be not in accordance with Contract Documents. If, upon examination such work is found not in accordance with Contract Documents, correct such Work and pay cost of examination and correction. If such Work is found in accordance with Contract Documents, Departmental Representative shall pay cost of examination and replacement.

### 1.3 INDEPENDENT INSPECTION AGENCIES

- .1 Independent Inspection/Testing Agencies will be engaged by Departmental Representative for purpose of inspecting and/or testing portions of Work above and beyond those required of the Contractor. Cost of such services will be borne by Departmental Representative.
- .2 Provide equipment required for executing inspection and testing by appointed agencies.
- .3 Employment of inspection/testing agencies does not relax responsibility to perform Work in accordance with Contract Documents.
- .4 If defects are revealed during inspection and/or testing, appointed agency will request additional inspection and/or testing to ascertain full degree of defect. Correct defect and irregularities as advised by Departmental Representative at no cost to Departmental Representative. Pay costs for retesting and re-inspection.

#### 1.4 ACCESS TO WORK

- .1 Allow inspection/testing agencies access to Work, off site manufacturing and fabrication plants.
- .2 Co-operate to provide reasonable facilities for such access.

#### 1.5 PROCEDURES

- .1 Notify appropriate agency and Departmental Representative in advance of requirement for tests, in order that attendance arrangements can be made.
- .2 Submit samples and/or materials required for testing, as specifically requested in specifications. Submit with reasonable promptness and in an orderly sequence so as not to cause delay in Work.
- .3 Provide labour and facilities to obtain and handle samples and materials on site. Provide sufficient space to store and cure test samples.

#### 1.6 REJECTED WORK

- .1 Remove defective Work, whether result of poor workmanship, use of defective products or damage and whether incorporated in Work or not, which has been rejected by Departmental Representative as failing to conform to Contract Documents. Replace or re-execute in accordance with Contract Documents.
- .2 Make good other Contractor's work damaged by such removals or replacements promptly.
- .3 If in opinion of Departmental Representative it is not expedient to correct defective Work or Work not performed in accordance with Contract Documents, Departmental Representative may deduct from Contract Amount difference in value between Work performed and that called for by Contract Documents, amount of which shall be determined by Departmental Representative.

#### 1.7 REPORTS

- .1 Submit 3 copies of inspection and test reports to Departmental Representative.
- .2 Provide copies to Subcontractor of work being inspected or tested, manufacturer or fabricator of material being inspected or tested.

#### 1.8 TESTS AND MIX DESIGNS

- .1 Furnish test results and mix designs as may be requested.
- .2 The cost of tests and mix designs beyond those called for in Contract Documents or beyond those required by law of Place of Work shall be appraised by Departmental Representative and may be authorized as recoverable.

PART 2 - PRODUCTS

2.1 NOT USED

.1 Not Used.

PART 3 - EXECUTION

3.1 NOT USED

.1 Not Used.

## PART 1 - GENERAL

### 1.1 REFERENCES

- .1 ASTM International
  - .1 ASTM D1621-10, Standard Test Method for Compressive Properties of Rigid Cellular Plastics.
  - .2 ASTM D1622/D1622M-14, Standard Test Methods for Apparent Density of Rigid Cellular Plastics.

### 1.2 ACTION AND INFORMATIONAL SUBMITTALS

- .1 Submit in accordance with Section 01 11 01.
- .2 The Contractor to submit the following information subjected to approval by the Departmental Representative 10 working days prior to starting operation.
  - .1 Shop Drawings indicating locations of the holes and product data sheets for the foam being used.
  - .2 The Contractor shall provide a description of the program for monitoring the work, including means of pressure measurement and movement detection.
  - .3 The Contractor shall provide a listing of personnel to perform the work. Personnel list shall include experience and qualification of key personnel.
  - .4 The Contractor shall submit a list of similar work performed in the previous five (5) years, using similar equipment and personnel. Include dates and project locations.
  - .5 The Contractor to provide a list of major components to be used, such as pumps, hoses, pipe fittings and drilling equipment and a list shall include manufacturers data on size, type, pressure rating, capacity and other critical characteristics for each item prior to the commencement of work.
  - .6 The Contractor to provide a work schedule outlining mobilization, drilling sequence and location, grouting and demobilization.
  - .7 Certification of gauges.

### 1.3 QUALITY ASSURANCE

- .1 With the bid documents, the Contractor shall submit the name of the firm that is qualified to do the grouting work. No work shall be done unless and until the firm is approved by the Departmental Representative
  - .1 The Contractor shall retain a geotechnical Engineer for QC purposes. The QC consisting of obtaining core samples through the grouted zones shall be carried out as soon as the grout materials are fully cured.
- .2 Foam injections are to be undertaken when the temperature of the ground and concrete are above 0 Degrees Celsius.

### 1.4 DELIVERY, STORAGE AND HANDLING

- .1 Deliver, store and handle materials in accordance with Section 01 11 01 and

with manufacturer's written instructions.

- .2 Delivery and Acceptance Requirements: deliver materials to site in original factory packaging, labelled with manufacturer's name and address.
- .3 Storage and Handling Requirements:
  - .1 Store materials off ground in a dry location and in accordance with manufacturer's recommendations in clean, dry, well-ventilated area.
  - .2 Replace defective or damaged materials with new.

## PART 2 - PRODUCTS

### 2.1 MATERIALS

- .1 The materials used for the grouting work shall be high density closed cell polyurethane resins supplied by qualified Contractor and approved by Departmental Representative.
- .2 The high density closed cell polyurethane system shall exhibit the following physical characteristics and properties in filling voids for rock and soil stabilization:
  - .1 Density 96 kg/m<sup>3</sup> to ASTM D1622/D1622M-14.
  - .2 Compressive Strength 0.76 Mpa to ASTM D1621-10.
  - .3 The cured material shall be impermeable to water.
  - .4 The cured material shall be resistant to petroleum and other toxic liquids.
  - .5 The cured material shall demonstrate a sound elasticity and shall demonstrate deformations which are approximately proportional to the forces applied (according to Hooke's Law) and with no more than 10% of deviation.
  - .6 The cured material shall demonstrate its ability to substantially retain its initial properties intact after curing.
- .3 A copy of MSDS and Technical Data sheets for a typical foam material is included in an appendix for reference for acceptable product.

## PART 3 - EXECUTION

### 3.1 INSTALLATION / APPLICATION

- .1 It is contractor's full responsibility to drill a sufficient number of holes to ensure the quality of grouting and ensure the voids being sufficiently filled and sealed. The Contractor shall determine the spacing between holes and the total number of holes based on his experience, equipment and grouting pressure and the subsurface conditions, etc. 90% of the voids should be sufficiently filled and sealed based on the evaluation of the core samples obtained from the grouted zones and approved by the Departmental Representative. Should the Contractor not reach the 90% limit then additional holes need to be drilled and filled until the Departmental Representative is satisfied and at no additional cost.
- .2 Grouting pressure and flow rate shall be continuously monitored at the grout

head and at the pump by pressure gauges and flow meters, suitably protected to prevent grout clogging or damage from handling vibration and shock.

- .3 The concrete pier, abutment and canal walls and any other structure adjacent to the grouting area shall be monitored for any movement, especially any upward movements during and after the grouting. A detailed settlement monitoring plan shall be submitted to the Departmental Representative for approval once the grouting measures are determined. Any upward movements of pier and abutment structures are not allowed.

### 3.2 EQUIPMENT

- .1 The drilling equipment shall be capable of drilling through the concrete pier, highly weathered bedrock as specified in the geotechnical report. The drilling equipment shall be capable of drilling within the limited head space under the existing bridge deck. A site visit is mandatory for the contractors bidding for the work to measure the space and determine the suitable equipment for the work.
- .2 The injection pipe shall have adequate strength to maintain the hole and to withstand the required jacking pumping pressures. The drilling equipment shall be capable of installing the pipes to be used.
- .3 The pipes shall be installed such that there is intimate contact with the drilled holes in order to prevent grout leakage and/or premature upward movement of the pipes during injection. Any other means of assuring grout delivery to the bottom of the holes have to be approved by the Departmental Representative.
- .4 No drilling fluids other than air or air injected foam shall be employed to install grout pipes, unless approved by the Engineer. All grout pipes shall be installed to within five degrees of vertical, or as directed by the Engineer.
- .5 The mixer shall be of sufficient capacity to continuously deliver grout for the proposed work
- .6 Gauges shall be provided at the pump and the grout pipe head to measure pressure. Type and location of gauges shall be as approved by the Engineer. All gauges shall be a certified accuracy to within  $\pm 2\%$ .

### 3.3 FIELD QUALITY CONTROL

- .1 The Contractor is to prepare drilling reports which shall contain at least the following information and submit and made available to the Departmental Representative at the end of each working day.
  - .1 Name of Driller and type of drill.
  - .2 Method being used.
  - .3 Date Started and Date Completed.
  - .4 Location of holes, spacing of holes and total number of holes.
  - .5 Depth of holes including material encountered and at what depth each material was met.
- .2 The Contractor to prepare grouting reports which contain at least the following information and submit and made available to the Departmental at the end of

each working day:

- .1 Name of grouting technician.
  - .2 Amount (i.e. weight) of grout material pumped.
  - .3 Date and rate of pumping.
  - .4 Grouting pressure at the hole.
  - .5 Type of pump and depth of hole.
- .3 Departmental Representative to verify results on site prior to the Contractor submitting the reports.

### 3.4 CLEANING

- .1 Progress Cleaning:
  - .1 Leave Work area clean at end of each day.
- .2 Final Cleaning: upon completion remove surplus materials, rubbish, tools and equipment in accordance with Section 01 11 01.
- .3 Waste Management: separate waste materials for reuse and recycling in accordance with Section 01 11 01.

# APPENDIX

## A

# Material Safety Data Sheet



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Dalton, GA

Hickory, NC

Mount Airy, NC

Salt Lake City, UT

## PRODUCT IDENTIFICATION

**Trade Name:** NCFI 24-010 R

**Chemical Family:** Polyol Resin System

**Chemical Name:** Mixture

**Formula:** N/A

**Synonyms:** Polyurethane Resin

**Date Prepared:** 07/05/12

## INGREDIENTS-HAZARD CLASSIFICATION

Name:	CAS NO.	%	PEL
Tertiary Amine Catalysts <sup>1</sup>		< 4	None Established.

<sup>1</sup> Not listed as a carcinogen (NTA, IARC, OSHA)

## SHIPPING INFORMATION

Not regulated when shipped by land, water or air.

## PHYSICAL DATA

**Boiling Point (°F):** >200°F

**Specific Gravity:** 1.07

**Solubility in Water:** High

**% Volatile by Volume:** 0

**Appearance and Odor:** Amber liquid, faint ammonia odor

## FIRE AND EXPLOSION HAZARD DATA

**Flash Point (test method):** >200°F (P-M)

**Flammable Limits (vapor)** N/A

**Extinguishing Media:** Water, dry chemicals, CO<sub>2</sub>

**Special Fire Fighting Procedures:** A self-contained breathing apparatus should be worn to protect against toxic and irritating vapors.

**Unusual Fire and Explosion Hazards**

## REACTIVITY DATA

**Stability:** Stable

**Conditions to Avoid:** N/A

**Polymerization:** Will not occur

**Conditions to Avoid:** N/A

**Incompatibility:** Isocyanates and other chemicals that react with hydroxyl groups.

**Hazardous Decomposition Products:** When burned; CO, CO<sub>2</sub>, NO<sub>x</sub>, and aliphatic fragments, halogens, halogen acids and possibly carbonyl halides.

## HEALTH HAZARD DATA

**Permissible Exposure Limit:** None established.

**Effects of Overexposure:** May cause skin or eye irritation upon contact. Avoid breathing vapors. The dense vapors can displace and reduce breathing air in confined or unventilated spaces causing asphyxiation. Overexposure may cause tremors, confusion, irritation, and may result in cardiac sensitization.

### First Aid Procedures

**Eyes:** Flush with water for at least 15 minutes. See a physician if irritation develops.

**Skin:** Wash with soap and water at first opportunity.

**Inhalation:** Move to fresh air if symptoms develop. If breathing is difficult, give oxygen and call physician.

**Ingestion:** Do not induce vomiting unless instructed to do so by a medical professional.

## SPECIAL PROTECTION INFORMATION

**Ventilation:** Local exhaust ventilation is recommended when working with this product. Uses requiring heating and/or spraying may require more ventilation or personal protective equipment.

**Respiratory Protection:** The specific respirator selected must be based on contamination levels of this material found in the workplace and the working limits of the respirator. A supplied air, full-face mask, positive pressure or continuous flow respirator or a supplied air hood is required when airborne concentrations are unknown or exceed threshold limit values. A positive pressure, self contained breathing apparatus can be used in emergencies or other unusual situations. Full-face air purifying respirators equipped with organic vapor cartridges can be used in certain situations, *see OSHA standard 29CFR 1910.134*. All equipment must be NIOSH approved and maintained.

**Eye Protection:** Goggles or chemical safety glasses.

**Gloves:** Chemically resistant rubber or plastic.

**Other:** Avoid eye and skin contact. Eye wash system and showers should be available.

## SPILL OR LEAK PROCEDURES

Remove or extinguish ignition or combustion sources.

Contain spill. Absorb with sawdust, etc., and shovel into container. Waste material should be disposed of under conditions which meet federal, state, and local environmental regulations.

Wash area with detergent and water.

## SPECIAL PRECAUTIONS

Store between 65°F and 85°F out of sunlight. Keep tightly sealed. Relieve pressure slowly when opening container. R Component drums can be sent to drum reconditioners or disposed of as ordinary industrial waste in compliance with pertinent regulations.

**CAUTION:** Under no circumstances should empty drums be burned or cut open with an electric or gas torch.

# Material Safety Data Sheet



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Salt Lake City, UT

## PRODUCT IDENTIFICATION

**Trade Name:** NCFI ES 24-010 A

**Chemical Family:** Aromatic Isocyanate

**Chemical Name:** Polymethylene polyphenylisocyanate

**Formula:** N/A

**Synonyms:** Polymeric MDI

**Date Prepared:** 07/05/12

## INGREDIENTS-HAZARD CLASSIFICATION

Name:	CAS NO.	%	PEL
Diphenylmethane diisocyanate (MDI) <sup>1</sup>	101-68-8	50	0.02 ppm ceiling
Higher polymers of similar structure	9016-87-9	50	None Established.

<sup>1</sup> Not listed as a carcinogen (NTA, IARC, OSHA)

## SHIPPING INFORMATION

Not regulated when shipped by land, water or air when packaged in single containers of 5000 pounds or less.

## PHYSICAL DATA

**Boiling Point (°F):** 625°F

**Specific Gravity:** 1.24

**Solubility in Water:** Insoluble, reacts

**% Volatile by Volume:** None

**Appearance and Odor:** Brown liquid, slight aromatic odor

## FIRE AND EXPLOSION HAZARD DATA

**Flash Point (test method):** 390°F (P-M)

**Extinguishing Media:** Water, dry chemicals, CO<sub>2</sub>

**Special Fire Fighting Procedures:** A self-contained breathing apparatus should be worn to protect against toxic and irritating vapors.

**Unusual Fire and Explosion Hazards:** At temperatures above 400°F, MDI can polymerize/decompose causing pressure build-up in closed containers and possibly rupture. Avoid water contamination in closed containers which may cause rupture (CO<sub>2</sub> is evolved).

## REACTIVITY DATA

**Stability:** Stable

**Conditions to Avoid:** Contamination with water

**Polymerization:** May occur from contact with water, alcohols, glycols or other materials containing active hydrogens.

**Incompatibility:** Water, alcohols, amines, strong bases.

**Hazardous Decomposition Products:** By high heat or fire; CO, CO<sub>2</sub>, NO<sub>x</sub>, benzene, toluene, aliphatic fragments, and traces of HCN.

## HEALTH HAZARD DATA

**Permissible Exposure Limit:** 0.02 ppm ceiling for MDI.

**Effects of Overexposure:** May cause skin or eye irritation upon contact. Inhalation of MDI vapors may cause breathlessness, chest discomfort, coughing and reduced pulmonary functions. Exposure may produce asthma-like symptoms, also may lead to allergic sensitivity.

### First Aid Procedures

**Eyes:** Flush with flowing water for at least 15 minutes, then obtain medical attention.

**Skin:** Remove contaminated clothing and wash off with soap & water.

**Inhalation:** Remove to fresh air, administer oxygen if necessary.

**Ingestion:** Drink large amounts of water. See a physician.

## SPECIAL PROTECTION INFORMATION

**Ventilation:** MDI has a very low vapor pressure at room temperature. General/local ventilation typically control exposure levels very adequately. Uses requiring heating and/or spraying may require more aggressive engineering controls or personal protective equipment. Monitoring is required to determine engineering controls.

**Respiratory Protection:** The specific respirator selected must be based on contamination levels of this material found in the workplace and the working limits of the respirator. A supplied air, full-face mask, positive pressure or continuous flow respirator or a supplied air hood is required when airborne concentrations are unknown or exceed threshold limit values. A positive pressure, self contained breathing apparatus can be used in emergencies or other unusual situations. Full-face air purifying respirators equipped with organic vapor cartridges can be used in certain situations, *see OSHA standard 29CFR 1910.134*. All equipment must be NIOSH approved and maintained.

**Eye Protection:** Wear goggles or chemical safety glasses.

**Gloves:** Chemically resistant rubber or plastic.

**Other:** Avoid eye and skin contact. Eye wash system and safety showers should be available.

## SPILL OR LEAK PROCEDURES

Contain spill. Absorb with sawdust, etc., and shovel into open top drum. Decontaminate absorbent and spill area with 2% detergent/water solution. Let waste stand for 1 to 2 days, then dispose of waste in a licensed facility. Respiratory protection/ventilation is recommended during clean-up.

## SPECIAL PRECAUTIONS

Store between 65°F and 85°F out of sunlight. Keep tightly sealed to prevent moisture contamination. Relieve pressure slowly when opening container. Once opened, protect contents from water with dry atmosphere (-40°F dew point). If isocyanate becomes contaminated, do not reseal. Empty isocyanate drums or other container should be decontaminated by filling with water or decontamination solution, preferably outdoors. Allow to stand for 24-48 hours, open to the atmosphere. **DO NOT SEAL DRUMS OR CONTAINERS.** Drain the drums and puncture to prevent reuse. Dispose of as ordinary industrial waste.

# Material Safety Data Sheet



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Hickory, NC

Mount Airy, NC

Salt Lake City, UT

Date Prepared: 2/22/89

Last Revision Date: 6/16/00

## SARA 313 INFORMATION

The isocyanate (A) component product of this NCFI system contains the following chemical(s) subject to the reporting requirements of Section 313 of Title III of the Superfund Amendments and Reauthorization Act (SARA) of 1986, EPCRA Section 313 (40 CFR 372) and the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA).

<u>CHEMICAL NAME</u>	<u>CAS NUMBER</u>	<u>CERCLA RQ</u>	<u>CONCENTRATION</u>
Methylene Bis Phenylisocyanate (Same as Diphenylmethane diisocyanate (MDI))	101-68-8	5000 lbs.	See MSDS – A Component
Polymeric Diphenylmethane diisocyanate	9016-87-9		See MSDS – A Component

### IMPORTANT NOTICE

This notification is a part of the Material Safety Data Sheet document and must not be detached. Any copying and redistribution of the Material Safety Data Sheet shall include copying of this notice and attaching the copy to the redistributed Material Safety Data Sheet copies.

**NCFI POUR SYSTEM 24-010**

**DESCRIPTION:**

NCFI 24-010 is a two component, water blown, all PMDI based low density spray polyurethane foam system designed for soil stabilization, road bed construction, deep hole injection and void filling. NCFI 24-010 is dispensed using 1/1 by volume ratio equipment. This system is available in regular and extremely fast (for Sub-Arctic conditions) speeds.

**DISTINGUISHING CHARACTERISTICS:**

- Tolerant to Mixing Variations and Conditions
- Inert After Curing
- Constant Volume with no Shrinkage

**TYPICAL RESIN PROPERTIES:**

	<u>24-010 R</u>	<u>24-010 A</u>
Viscosity	500 cps	200 cps
Lbs./Gallon	8.7 lbs.	10.2 lbs.
Appearance	transparent, amber liquid	transparent, brown liquid
Shelf Life	6 months	6 months

**MIX RATIO:**

	<u>24-010 R</u>	<u>24-010 A</u>
By Weight	100 parts	119 parts
By Volume	100 parts	100 parts

**TYPICAL REACTION PROPERTIES:**

Machine Mix @ 140°F

	<u>Regular</u>	<u>Arctic</u>
Rise Time (sec)	5	3
Tack Free (sec)	11	7
Firm Time (sec)	90	65
Density (FRC)	2.5 pcf	2.6 pcf

**TYPICAL PHYSICAL PROPERTIES:**

Core Density	2.5 pcf
Closed Cell Content	>85%
Moisture Vapor Transmission	2-4 perm in.
Water Absorption, ASTM D2842	≤0.06 lbs/ft <sup>2</sup>
Resistance to Solvents	Excellent
Resistance to Mold and Mildew	Excellent
Maximum Service Temperature	180°F

\*The above values are average values obtained from laboratory experiments and should serve only as guide lines.

## NCFI 24-010 APPLICATION INFORMATION

### EQUIPMENT AND COMPONENT RATIOS:

NCFI 24-010 should be sprayed using standard spray equipment with 1/1 by volume proportioning pumps capable of maintaining 800-1200 psi dynamic pressure. The Gusmer 20/35 with GX7 gun is preferred. Preheater temperatures should be set at a minimum of 130°F. 140°F is the optimum process temperature. NCFI 24-010 **R** is connected to the **resin/polyol** pumps with NCFI 24-010 **A** being connected to the **isocyanate** pumps.

### STORAGE AND USE OF CHEMICALS:

Keep temperature of chemicals at 70°F for several days before use. Cold chemicals can cause poor mixing, pump cavitation or other process problems due to higher viscosity at lower temperatures. Storage temperature should not exceed 100°F. Prolonged exposure to temperatures below 60°F can cause the 'A' component to freeze. Do not store in direct sunlight. Keep drums tightly closed when not in use and under nitrogen pressure of 2 - 3 psi after they have been opened.

### PREPARATION OF SURFACE TO BE SPRAYED:

All surfaces to be sprayed should be clean, dry, and free of dew or frost. All metal to which foam is to be applied must be free of oil, grease, etc.

### OPTIMUM ADHESION TEMPERATURE OF SURFACE TO BE SPRAYED:

On general work where the surface to be sprayed will remain at ambient temperature or cooler, the surface should be between 70°F and 120°F. In this range the warmer the surface the better the adhesion.

### SAFE HANDLING OF LIQUID COMPONENTS:

Use caution in removing bungs from the container. Loosen the small bung first and let any built up gas escape before completely removing. Avoid prolonged breathing of vapors. In case of chemical contact with eyes, flush with water for at least 15 minutes and get medical attention. For further information refer to "MDI-Based Polyurethane Foam Systems: Guidelines for Safe Handling and Disposal" publication AX-119 published by Alliance For The Polyurethanes Industry 1300 Wilson Blvd, Suite 800, Arlington, VA 22209.

### Caution:

Polyurethane products manufactured or produced from this liquid system may present a serious fire hazard if improperly used or allowed to remain exposed or unprotected. The character and magnitude of any such hazard will depend on a broad range of factors which are controlled and influenced by the manufacturing and production process, by the mode of application or installation and by the function and usage of the particular product. *Any flammability rating contained in this literature is not intended to reflect hazards presented by this or any other material under actual fire conditions. These ratings are used solely to measure and describe the product's response to heat and flame under controlled laboratory conditions.* Each person, firm or corporation engaged in the manufacture, production, application, installation or use of any polyurethane product should carefully determine whether there is a potential fire hazard associated with such product in a specific usage, and utilize all appropriate precautionary and safety measures

The information on our data sheets is to assist customers in determining whether our products are suitable for their applications. The customers must satisfy themselves as to the suitability for specific cases. North Carolina Foam Industries warrants only that the material shall meet its specifications; this warranty is in lieu of all other written or unwritten, expressed or implied warranties and North Carolina Foam Industries expressly disclaims any warranty of merchantability, fitness for a particular purpose, or freedom from patent infringement. Accordingly, buyer assumes all risks whatsoever as to the use of the material. Buyer's exclusive remedy as to any breach of warranty, negligence or other claim shall be limited to the purchase price of the material. Failure to adhere strictly to any recommended procedures shall relieve North Carolina Foam Industries of all liability with respect to the material or the use thereof.

# 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



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Appendix C: Concrete Core Strength Testing Results

Appendix D: Rock Core Strength Testing Results

Appendix E: Report of the Previous Geotechnical Investigation carried out by Golder

## 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 through 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 m 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:

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 existing 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 fracture 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 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 impact 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**

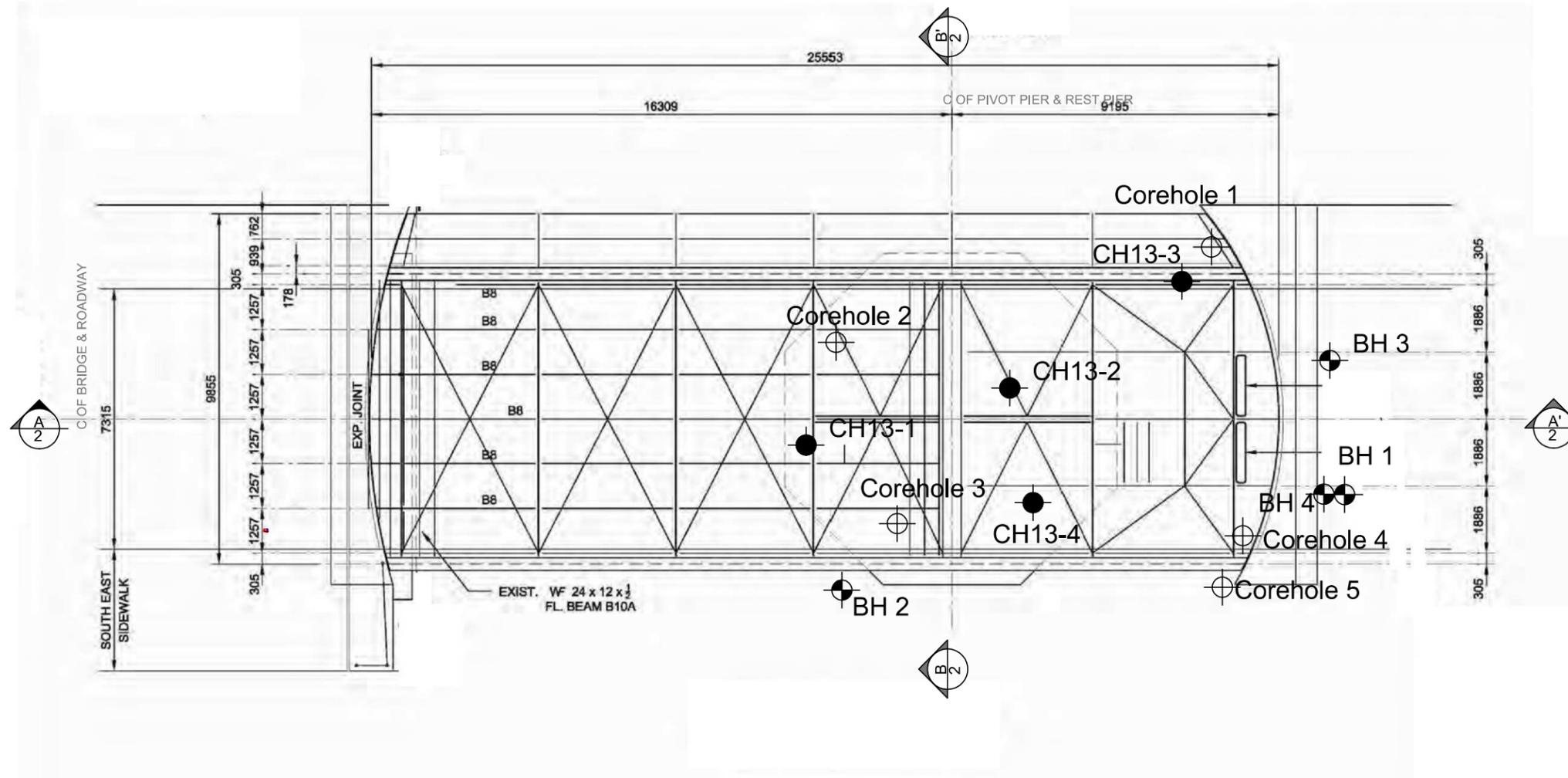
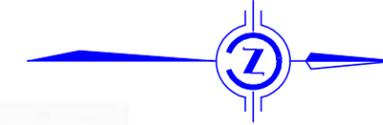


David B. Liu, P.Eng.  
Senior Geotechnical Engineer



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

for



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- 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**

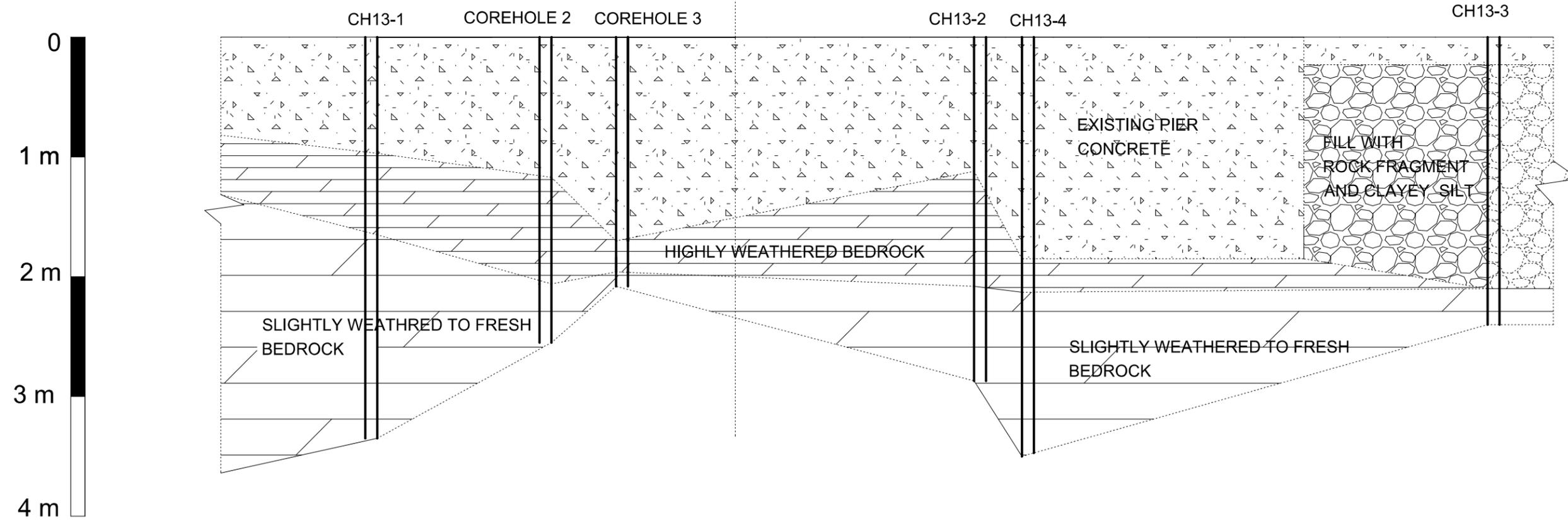
**NOT TO SCALE**

**All locations are Approximate**

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

# CROSS SECTION A-A

C/L



**Notes:**

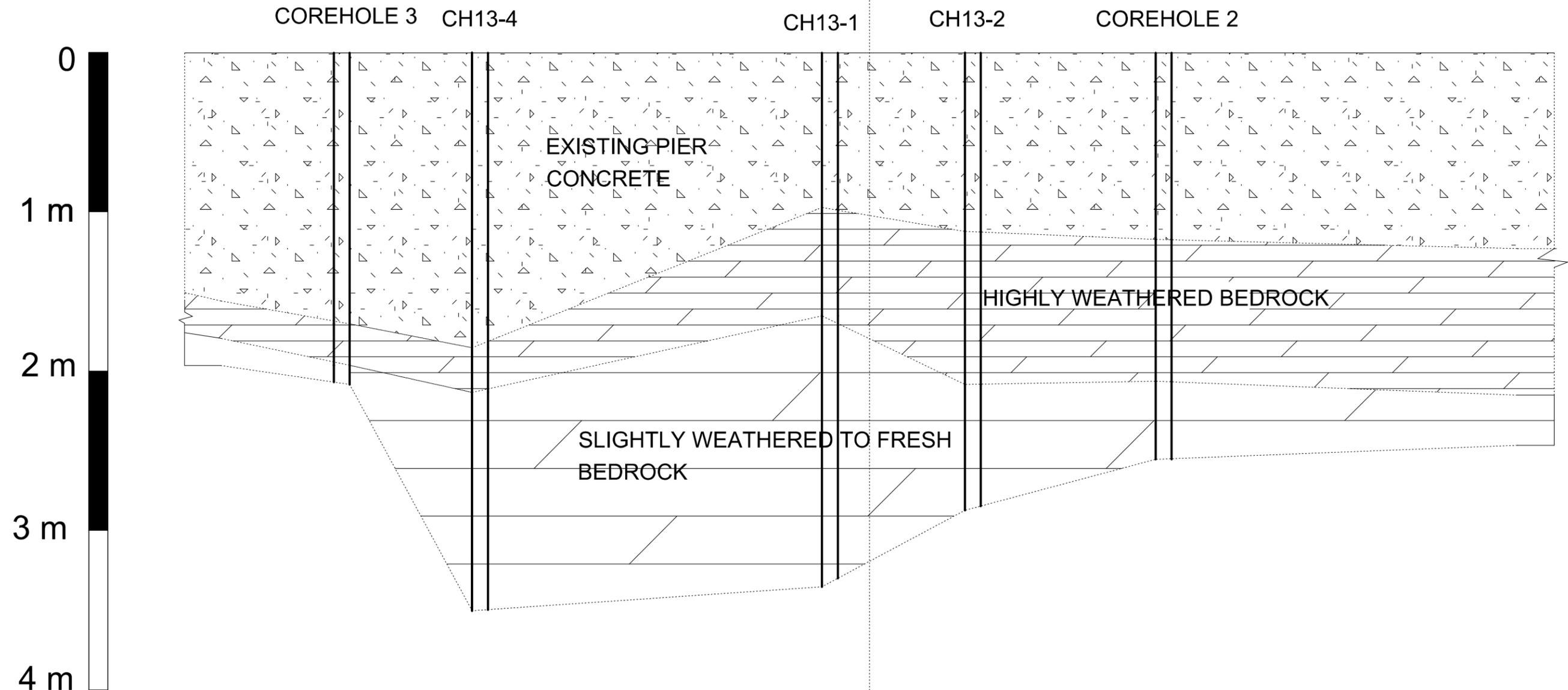
1. Not to scale, all locations are approximate
2. The assumed boundary of the concrete footings & soil strada are based on limited corehole data and should be considered approximate

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

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# CROSS SECTION B-B

C/L



**Notes:**

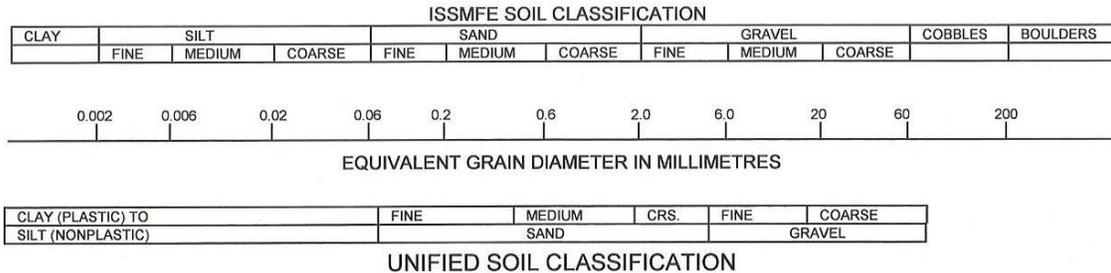
1. Not to scale, all locations are approximate
2. The assumed boundary of the concrete footings & soil strada are based on limited corehole data and should be considered pproximate

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

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## Drawing 2: Notes on Soil Sample Descriptions

- 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.



- 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.
- 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.

PROJECT: Geotechnical Investigation - Bridge Replacement CLIENT: Associated Engineering PROJECT LOCATION: Hastings Swing Bridge, Hastings, Ontario DATUM: N/A BH LOCATION: See Corehole Location Plan	<b>DRILLING DATA</b> Method: Coring Diameter: 107 mm Date: Oct/01/2013 REF. NO.: 1842-910 ENCL NO.: 3
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SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS AND GRAIN SIZE DISTRIBUTION (%)		
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			SHEAR STRENGTH (kPa)											WATER CONTENT (%)	
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.																			

SPL SOIL LOG 1882-910 '2013'10'28.GPJ SPL\_GDT 10/2/14

GROUNDWATER ELEVATIONS  
 Measurement

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

PROJECT: Geotechnical Investigation - Bridge Replacement CLIENT: Associated Engineering LOCATION: Hastings Swing Bridge, Hastings, Ontario DATUM: N/A BH LOCATION: See Corehole Location Plan	<b>DRILLING DATA</b> Method: Coring Diameter: 107 mm Date: Oct/01/2013 REF. NO.: 1842-910 ENCL NO.: 3
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(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE		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 <sup>3</sup> ) E (GPa)	
			NUMBER	SIZE													
1.0	<b>LIMESTONE:</b> Highly weathered, grey, highly fractured fossiliferous, layers of clayey silt	W.L. 1.0 mBGL Oct 01, 2013	1	93mm	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	<b>LIMESTONE:</b> Slightly weathered to fresh, grey to dark grey, fossiliferous.		3	93mm	80	41	23	3	Fresh								
1.8			4	93mm	100	32	32	1	Fresh Fracture: 3.07m-3.20m								
2.3			5	93mm	100	45	40	1	Fresh								
2.8																	
3.4	End of Corehole																

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

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<sub>s(50)</sub>

PROJECT: Geotechnical Investigation - Bridge Replacement CLIENT: Associated Engineering PROJECT LOCATION: Hastings Swing Bridge, Hastings, Ontario DATUM: N/A BH LOCATION: See Corehole Location Plan	<b>DRILLING DATA</b> Method: Coring Diameter: 107 mm Date: Oct/01/2013 REF. NO.: 1842-910 ENCL NO.: 4
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SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS AND GRAIN SIZE DISTRIBUTION (%)	
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			SHEAR STRENGTH (kPa)											WATER CONTENT (%)
0.0	<b>CONCRETE:</b> 1120mm																		
1.1	<b>LIMESTONE:</b> Highly weathered, grey to dark grey, highly fractured fossiliferous.		1	RC		∇ W. L. 1.2 mBGL Oct 01, 2013													
2			2	RC															
2.1	<b>LIMESTONE:</b> Slightly weathered to fresh, grey to dark grey, fossiliferous.		3	RC															
			4	RC															
2.9	<b>END OF BOREHOLE</b> Note: 1) Water level at 1.2m upon completion. 2) Refer to rock core log.																		

SPL SOIL LOG 1882-910 '2013'10'28.GPJ SPL\_GDT 10/2/14

**GROUNDWATER ELEVATIONS**  
 Measurement

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

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

(m) ELEV DEPTH	ROCK DESCRIPTION	GROUND WATER CONDITIONS	CORE SAMPLE		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 <sup>3</sup> )	E (GPa)
			NUMBER	SIZE													
1.1	<b>LIMESTONE:</b> Highly weathered, grey, highly fractured fossiliferous, layers of clayey silt	∇	1	93mm	58	15		0	34	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	<b>LIMESTONE:</b> 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																

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

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<sub>S(50)</sub>

PROJECT: Geotechnical Investigation - Bridge Replacement CLIENT: Associated Engineering PROJECT LOCATION: Hastings Swing Bridge, Hastings, Ontario DATUM: N/A BH LOCATION: See Corehole Location Plan	<b>DRILLING DATA</b> Method: Coring Diameter: 107 mm Date: Oct/01/2013 REF. NO.: 1842-910 ENCL NO.: 5
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SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS AND GRAIN SIZE DISTRIBUTION (%)	
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			"N" BLOWS 0.3 m	SHEAR STRENGTH (kPa)										WATER CONTENT (%)
0.0	CONCRETE: 230mm																	
0.2	FILL: rock fragments with clayey silt																	
1																		
2																		
2.1	Highly weathered, grey to dark grey, highly fractured fossiliferous Limestone, layers of clayey silt, T.C.R.=70%, SCR = 30%																	
2.4	R.O.D. = 0% <b>END OF BOREHOLE</b> Notes: 1) Water level at 1.0m upon completion.																	

W. L. 1.0 mBGL  
Oct 01, 2013

SPL SOIL LOG 1882-910 '2013'10'28.GPJ SPL\_GDT 10/2/14

GROUNDWATER ELEVATIONS  
 Measurement

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

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			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m <sup>3</sup> )	REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m			20	40	60	80	100						
0.0	CONCRETE: 1850mm																	
1.9	LIMESTONE: Highly weathered, grey, highly fractured fossiliferous, pockets 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														
			3	RC														
			4	RC														
3.5	END OF BOREHOLE Notes: 1) Water level at 2.2m upon completion. 2) Refer to rock core log.																	

W. L. 2.2 mBGL  
Oct 02, 2013

SPL SOIL LOG 1882-910 '2013'10'28.GPJ SPL\_GDT 10/2/14

GROUNDWATER ELEVATIONS  
 Measurement

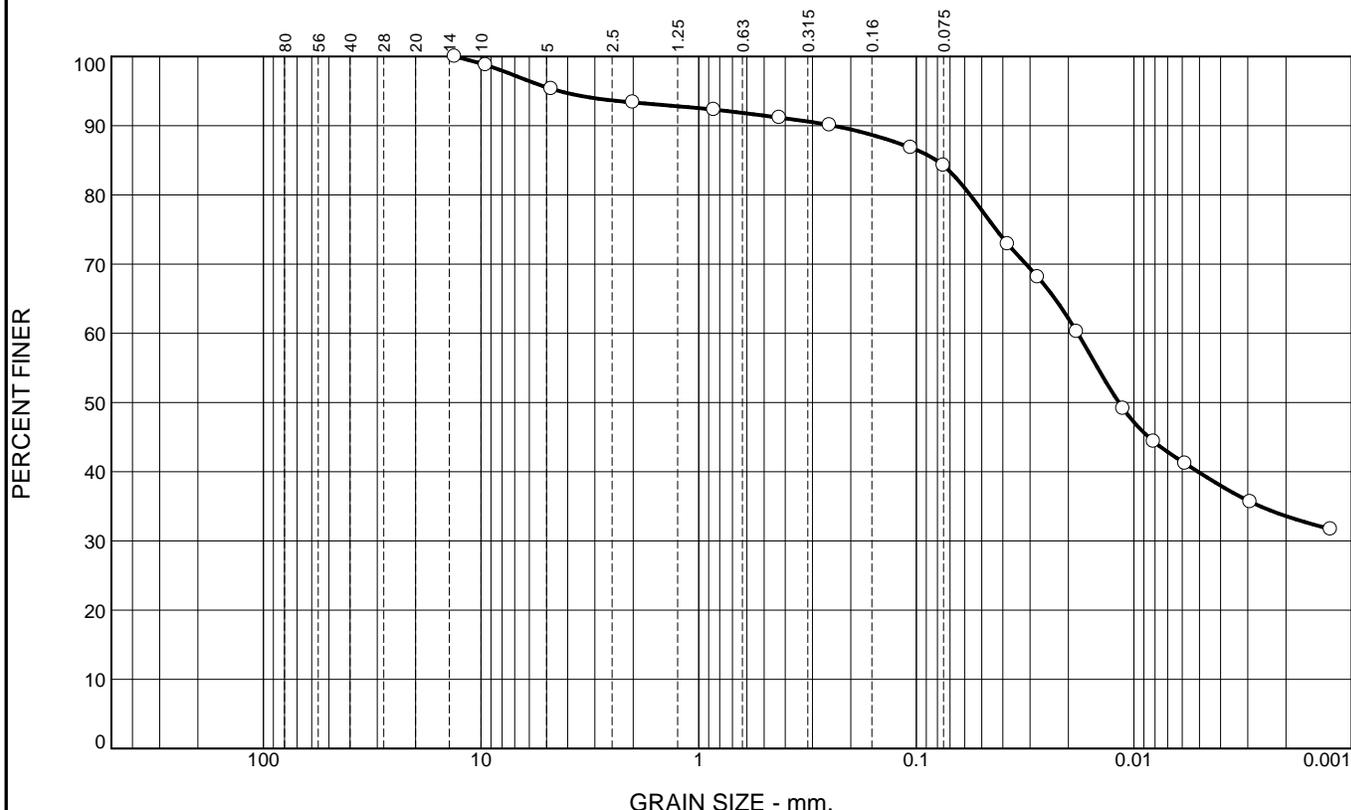
GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

PROJECT: Geotechnical Investigation - Bridge Replacement CLIENT: Associated Engineering LOCATION: Hastings Swing Bridge, Hastings, Ontario DATUM: N/A BH LOCATION: See Corehole Location Plan	<b>DRILLING DATA</b> 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		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 <sup>3</sup> ) E (GPa)
			NUMBER	SIZE												
1.9	<b>LIMESTONE:</b> Highly weathered, grey, highly fractured fossiliferous, pockets of clayey silt ( <i>continued</i> )		1	93mm	90	20		0	12	Slightly weathered  100mm Clay layer at 2.03m						
2.1	<b>LIMESTONE:</b> Slightly weathered to fresh, grey to dark grey, fossiliferous.	▽	W. L. 2.2 mBGL Oct 02, 2013													
2.8			2	93mm	92	73		35	3							
3.3			3	93mm	84	53		26	4							
3.5	End of Corehole		4	93mm	100	100		100								

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		

**Soil Description**  
Clayey silt, trace gravel, some sand

**Atterberg Limits**  
 PL=                      LL=                      PI=

**Coefficients**  
 D<sub>90</sub>= 0.2414      D<sub>85</sub>= 0.0807      D<sub>60</sub>= 0.0181  
 D<sub>50</sub>= 0.0117      D<sub>30</sub>=                      D<sub>15</sub>=  
 D<sub>10</sub>=                      C<sub>u</sub>=                      C<sub>c</sub>=

**Classification**  
 USCS=                      AASHTO=

**Remarks**  
 Sampled by Andy on Oct 2, 2013

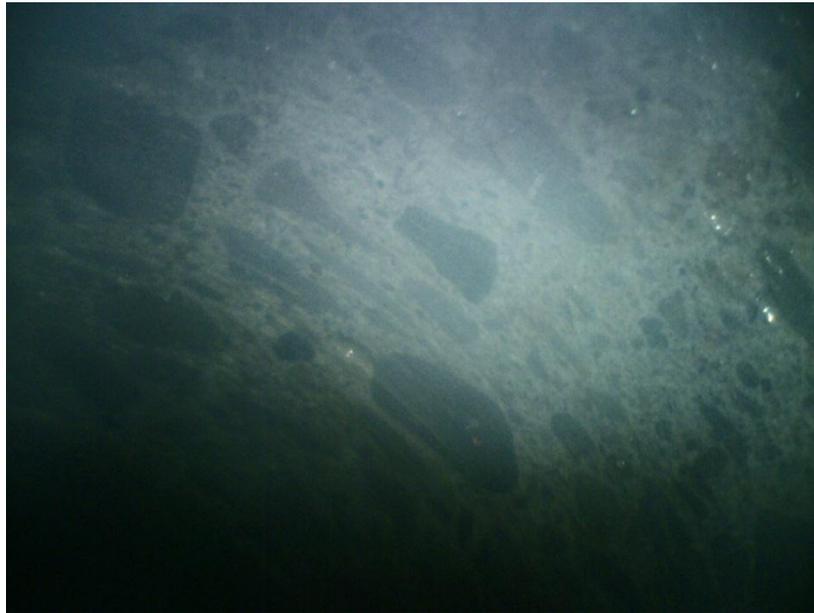
\* (no specification provided)

**Location:** CH4 (SA1)      **Depth:** 78" - 84"      **Date:**

**Sample Number:** MM-0216

# **APPENDIX A**

## **PHOTOGRAPHS INSIDE COREHOLES**



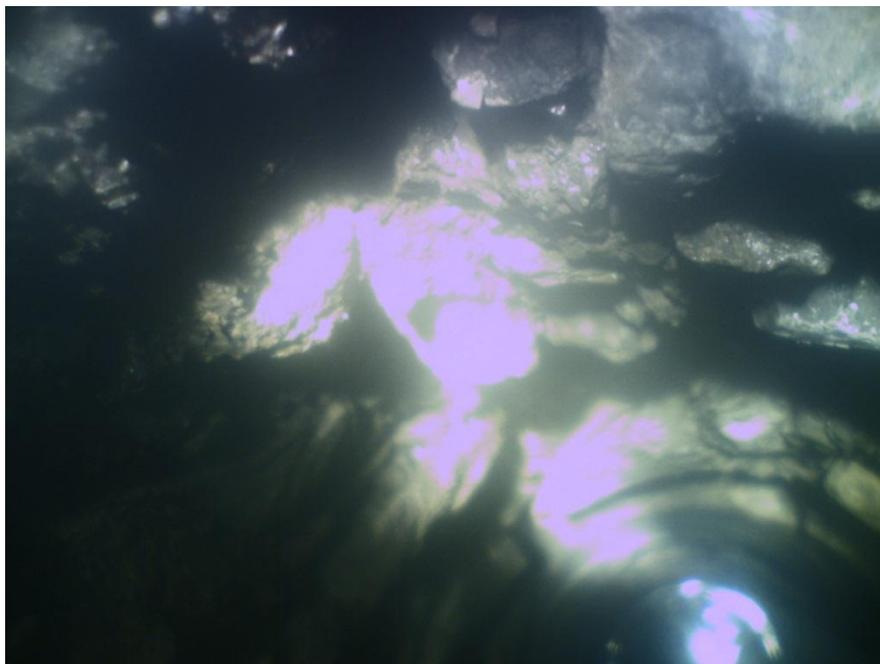
**Photograph 1: no cracking or fracture of the pier concrete noted inside the corehole**



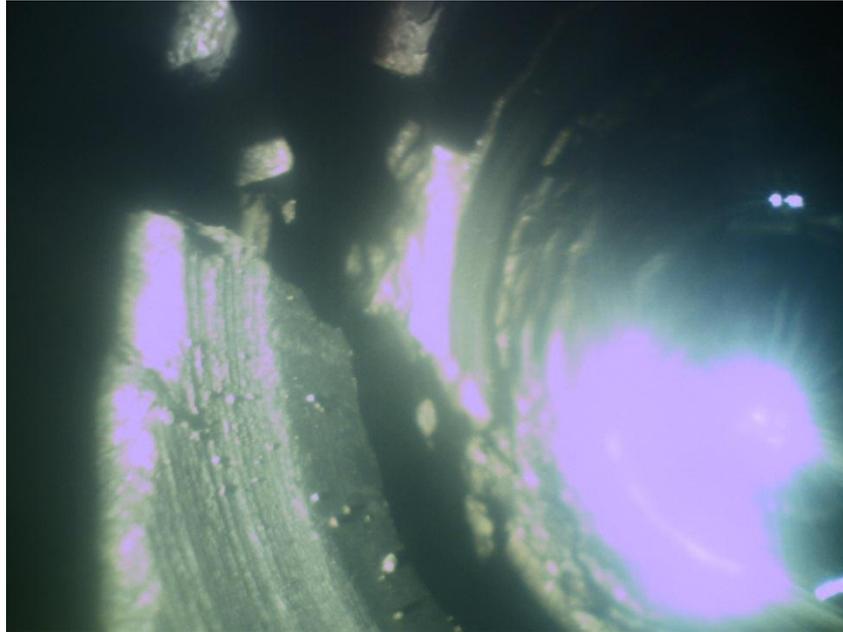
**Photograph 2: no cracking or fracture of the pier concrete noted inside the corehole**



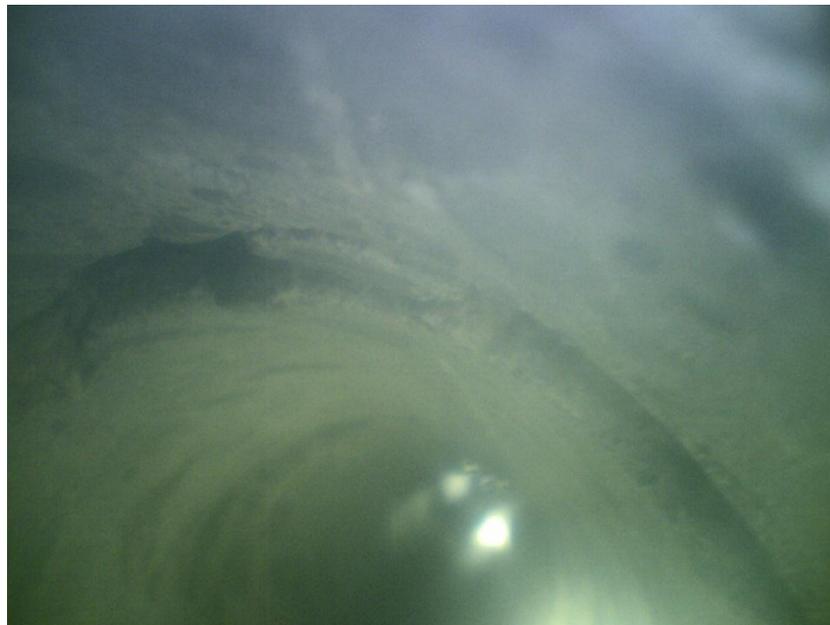
**Photograph 3: highly weathered rock between pier concrete and bedrock**



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



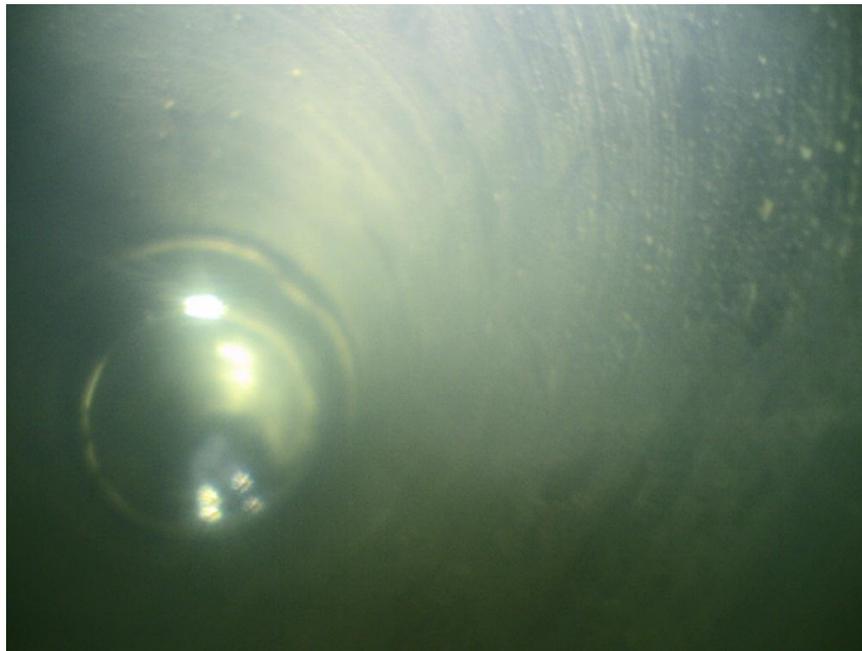
**Photograph 5: void between pier concrete and bedrock and highly weathered rock pieces**



**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**

**APPENDIX 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

**Concrete Strength  
Test Report  
100mm X 200mm Cylinder**

**Project Name:** Hasting Swing Bridge, Hastings.

**Project Number:** 1842-910

**Client:** Associated Engineering

**Lab Number:** MC-0134

**Specified Day Strength MPa**

**Mix Reference:** Concrete Cores

Sample No.	Date Cast	Date Tested	Curing	Age (Days)	T.O.F.*	Unit Mass (Kg/m <sup>3</sup> )	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 (m<sup>3</sup>):**

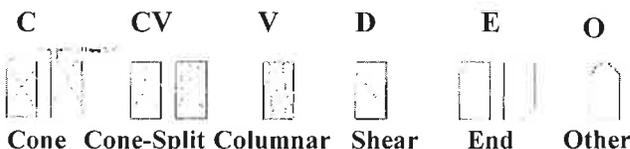
**By What Authority:** Not Applicable

**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



**\*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  $10^{-5} \text{ s}^{-1}$ , 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)**

<b>Sample (depths indicated)</b>	<b>Bulk Density (g/cm<sup>3</sup>)</b>	<b>UCS (MPa)</b>	<b>Young's Modulus (GPa)</b>	<b>Poisson's ratio</b>
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

A world of  
capabilities  
delivered locally

**Report Number:** 11-1184-0022

**Distribution:**

4 Copies - Delcan Corporation  
2 Copies - Golder Associates Ltd.





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### 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 28<sup>th</sup>, 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.



## GEOTECHNICAL EVALUATION HASTINGS SWING BRIDGE

- 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 rig 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 existing 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 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 are no cracks 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



## GEOTECHNICAL EVALUATION HASTINGS SWING BRIDGE

existing 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 =  $\gamma$  = 23 kN/m<sup>3</sup>
- Unit weight of water =  $\gamma_w$  = 9.8 kN/m<sup>3</sup>
- "Unfactored coefficient of friction between Concrete and fractured limestone bedrock =  $\mu$  = 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/m <sup>3</sup> )	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 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 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.



## GEOTECHNICAL EVALUATION HASTINGS SWING BRIDGE

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	=	$\gamma$	=	21 kN/m <sup>3</sup>
- Unit weight of fractured limestone	=	$\gamma$	=	23 kN/m <sup>3</sup>
- Unit weight of abutment wall	=	$\gamma$	=	24 kN/m <sup>3</sup>
- Unit weight of water	=	$\gamma_w$	=	9.8 kN/m <sup>3</sup>
- "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	=	$\mu$	=	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.



## GEOTECHNICAL EVALUATION HASTINGS SWING BRIDGE

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.**

David B. Liu, P. Eng.  
Geotechnical Engineer



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

DBL/TJG/ms/sv/leb

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## LIST OF ABBREVIATIONS

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

<b>I</b>	<b>SAMPLE TYPE</b>	<b>III</b>	<b>SOIL DESCRIPTION</b>	
	AS Auger sample		(a) Cohesionless Soils	
	BS Block sample		<b>Density Index</b>	N
	CS Chunk sample		(Relative Density)	<u>Blows/300 mm</u>
	DO Drive open			<u>or Blows/ft.</u>
	DS Denison type sample		Very loose	0 to 4
	FS Foil sample		Loose	4 to 10
	RC Rock core		Compact	10 to 30
	SC Soil core		Dense	30 to 50
	ST Slotted tube		Very dense	over 50
	TO Thin-walled, open		(b) Cohesive Soils	
	TP Thin-walled, piston		<b>Consistency</b>	<b><math>c_u, s_u</math></b>
	WS Wash sample			<b>kPa</b> <b>psf</b>
<b>II</b>	<b>PENETRATION RESISTANCE</b>		Very soft	0 to 12
	<b>Standard Penetration Resistance (SPT), N:</b>		Soft	12 to 25
	The number of blows by a 63.5 kg. (140 lb.)		Firm	25 to 50
	hammer dropped 760 mm (30 in.) required		Stiff	50 to 100
	to drive a 50 mm (2 in.) drive open		Very stiff	100 to 200
	sampler for a distance of 300 mm (12 in.)		Hard	over 200
	<b>Dynamic Penetration Resistance; <math>N_d</math>:</b>			over 4,000
	The number of blows by a 63.5 kg (140 lb.)		<b>IV. SOIL TESTS</b>	
	hammer dropped 760 mm (30 in.) to drive		w	water content
	uncased a 50 mm (2 in.) diameter, 60° cone		$w_p$	plastic limit
	attached to "A" size drill rods for a distance		$w_l$	liquid limit
	of 300 mm (12 in.)		C	consolidation (oedometer) test
			CHEM	chemical analysis (refer to text)
<b>PH:</b>	Sampler advanced by hydraulic pressure		CID	consolidated isotropically drained triaxial test <sup>1</sup>
<b>PM:</b>	Sampler advanced by manual pressure		CIU	consolidated isotropically undrained triaxial
<b>WH:</b>	Sampler advanced by static weight of hammer			test with porewater pressure measurement <sup>1</sup>
<b>WR:</b>	Sampler advanced by weight of sampler and		$D_R$	relative density (specific gravity, $G_s$ )
	rod		DS	direct shear test
			M	sieve analysis for particle size
<b>Piezo-Cone Penetration Test (CPT):</b>			MH	combined sieve and hydrometer (H) analysis
An electronic cone penetrometer with			MPC	Modified Proctor compaction test
a 60° conical tip and a projected end area			SPC	Standard Proctor compaction test
of 10 cm <sup>2</sup> pushed through ground			OC	organic content test
at a penetration rate of 2 cm/s. Measure-			SO <sub>4</sub>	concentration of water-soluble sulphates
ments of tip resistance ( $Q_t$ ), porewater			UC	unconfined compression test
pressure (PWP) and friction along a			UU	unconsolidated undrained triaxial test
sleeve are recorded electronically			V	field vane test (LV-laboratory vane test)
at 25 mm penetration intervals.			$\gamma$	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

$\pi$	= 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

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stresses (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	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 $\rho$ . Unit weight symbol is $\gamma$ 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
$C_a$	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
$\sigma'_p$	pre-consolidation pressure
OCR	Overconsolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (e) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction = $\tan \delta$
$c'$	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

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE		SHEAR STRENGTH Cu, kPa		WATER CONTENT PERCENT							
								20	40	60	80	10 <sup>-6</sup>	10 <sup>-5</sup>	10 <sup>-4</sup>	10 <sup>-3</sup>		
0	50 mm Diameter Coring Machine	GROUND SURFACE CONCRETE						25	50	75	100	5	10	15	20		
1		Weathered, grey to dark grey, highly fractured fossiliferous Limestone, pockets of clayey silt T.C.R. = 70% S.C.R. = 35% R.O.D. = 0%															
2					1.19												
3	END OF COREHOLE																
10	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)																

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

SHEET 1 OF 1

BORING DATE: May 6, 2011

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

LDN\_BHS 11-1184-0022.GPJ GLDR\_LDN.GDT 5/30/11 DATA INPUT: MK MAY 2011

DEPTH SCALE  
1 : 50



LOGGED: AZ  
CHECKED:

PROJECT: 11-1184-0022  
 LOCATION: SEE FIGURE 2

# RECORD OF COREHOLE 3

SHEET 1 OF 1

BORING DATE: May 6, 2011

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE		SHEAR STRENGTH Cu, kPa		WATER CONTENT PERCENT							
								20	40	60	80	10 <sup>-6</sup>	10 <sup>-5</sup>	10 <sup>-4</sup>	10 <sup>-3</sup>		
0	50 mm Diameter Coring Machine	GROUND SURFACE CONCRETE						25	50	75	100	5	10	15	20		
1																	
2		Weathered, grey to dark grey, highly fractured fossiliferous 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 fossiliferous Limestone BEDROCK T.C.R. = 100% S.C.R. = 90% R.O.D. = 80%			1.96												
2					2.08												
3		END OF COREHOLE															
4																	
5																	
6																	
7																	
8																	
9																	
10																	

LDN\_BHS 11-1184-0022.GPJ GLDR\_LDN.GDT 5/30/11 DATA INPUT: MK MAY 2011

DEPTH SCALE  
1 : 50



LOGGED: AZ  
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DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER		TYPE	SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT					
								20	40	60	80	nat V. + rem V. ⊕	Q - U -	10 <sup>-6</sup>			10 <sup>-5</sup>
0	50 mm Diameter Coring Machine	GROUND SURFACE CONCRETE															
2.36		Weathered, grey to dark grey, highly fractured fossiliferous Limestone, pockets of clayey silt T.C.R. = 80% S.C.R. = 80% R.O.D. = 0%															
2.79 3.00		Slightly weathered grey to dark grey fine grained fossiliferous Limestone BEDROCK T.C.R. = 100% S.C.R. = 93% R.O.D. = 93% END OF COREHOLE															
10		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)															

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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		SAMPLES		ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER		TYPE	SHEAR STRENGTH Cu, kPa		nat V. + Q - ● rem V. ⊕ U - ○		WATER CONTENT PERCENT Wp   ○ W   WI				
25	50					75		100	5	10	15	20				
0	50 mm Diameter Coring Machine	GROUND SURFACE														
		CONCRETE														
1.73		END OF COREHOLE														

LDN\_BHS 11-1184-0022.GPJ GLDR\_LDN.GDT 5/30/11 DATA INPUT: MK MAY 2011

DEPTH SCALE  
1 : 50



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			SAMPLES			ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m		SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT					
									20	40	60	80	nat V. + rem V. ⊕	Q - U - ⊙	10 <sup>-6</sup>	10 <sup>-5</sup>		
		GROUND SURFACE							25	50	75	100						
0	150 mm Diameter SOLID STEM AUGERS Track Mounted Power Auger	TOPSOIL		0.13	1	AS	-											
1		Compact to very dense brown gravelly sand, trace silt, containing rock fragments (FILL)			2	50 DO	30											M
2		Probably weathered Limestone BEDROCK		1.73	3A	50 DO	53											
2		END OF BOREHOLE DUE TO AUGER REFUSAL TO FURTHER AUGERING ON PROBABLE BEDROCK		2.00	3B													
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

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

LDN\_BHS 11-1184-0022.GPJ GLDR\_LDN.GDT 5/30/11 DATA INPUT: MK MAY 2011

DEPTH SCALE  
1 : 50



LOGGED: AZ  
CHECKED:

PROJECT: 11-1184-0022

# RECORD OF BOREHOLE BH-3

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			SAMPLES		ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE		SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT					
								20	40	60	80	nat rem	V. V.	+ ⊕	Q - U		
0		GROUND SURFACE						25	50	75	100						
		ASPHALT (120 mm)															
		GRANULAR BASE		0.12		1A											
		Dense to compact sandy gravel, trace silt, containing rock fragments (FILL)		0.50		1B											
1						2											
		Very loose to loose brown silty sand, trace to some gravel, containing rock fragments (FILL)		1.37		3											
2						4											
		Probable CONCRETE (Existing Footing)		2.74													
3																	
4		Probably weathered Limestone BEDROCK		3.96													
		END OF BOREHOLE DUE TO AUGER REFUSAL TO FURTHER AUGERING ON PROBABLE BEDROCK		4.11													
5																	
6																	
7																	
8																	
9																	
10																	

Borehole open and dry upon completion of drilling, May 7, 2011

LDN\_BHS 11-1184-0022.GPJ GLDR\_LDN.GDT 5/30/11 DATA INPUT: MK MAY 2011

DEPTH SCALE  
1 : 50



LOGGED: AZ  
CHECKED:

PROJECT: 11-1184-0022

# RECORD OF BOREHOLE BH-4

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		SAMPLES			ELEVATION	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	INSTALLATION AND GROUNDWATER OBSERVATIONS	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE		BLOWS/0.3m	SHEAR STRENGTH Cu, kPa				WATER CONTENT PERCENT					
									20	40	60	80	nat V. + rem V. ⊕	Q - U - ⊙	10 <sup>-6</sup>			10 <sup>-5</sup>
0	150 mm Diameter SOLID STEM AUGERS Track Mounted Power Auger	GROUND SURFACE						25	50	75	100							
		ASPHALT (120 mm)																
		GRANULAR BASE		0.12	1	AS	-											
		Brown gravelly sand, trace silt (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	
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

LDN\_BHS 11-1184-0022.GPJ GLDR\_LDN.GDT 5/30/11 DATA INPUT: MK MAY 2011

DEPTH SCALE  
1 : 50



LOGGED: AZ  
CHECKED:

PROJECT: 11-1184-0022

# RECORD OF TEST PIT TP-1

SHEET 1 OF 1

LOCATION: SEE FIGURE 2

EXCAVATION DATE: May 3, 2011

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

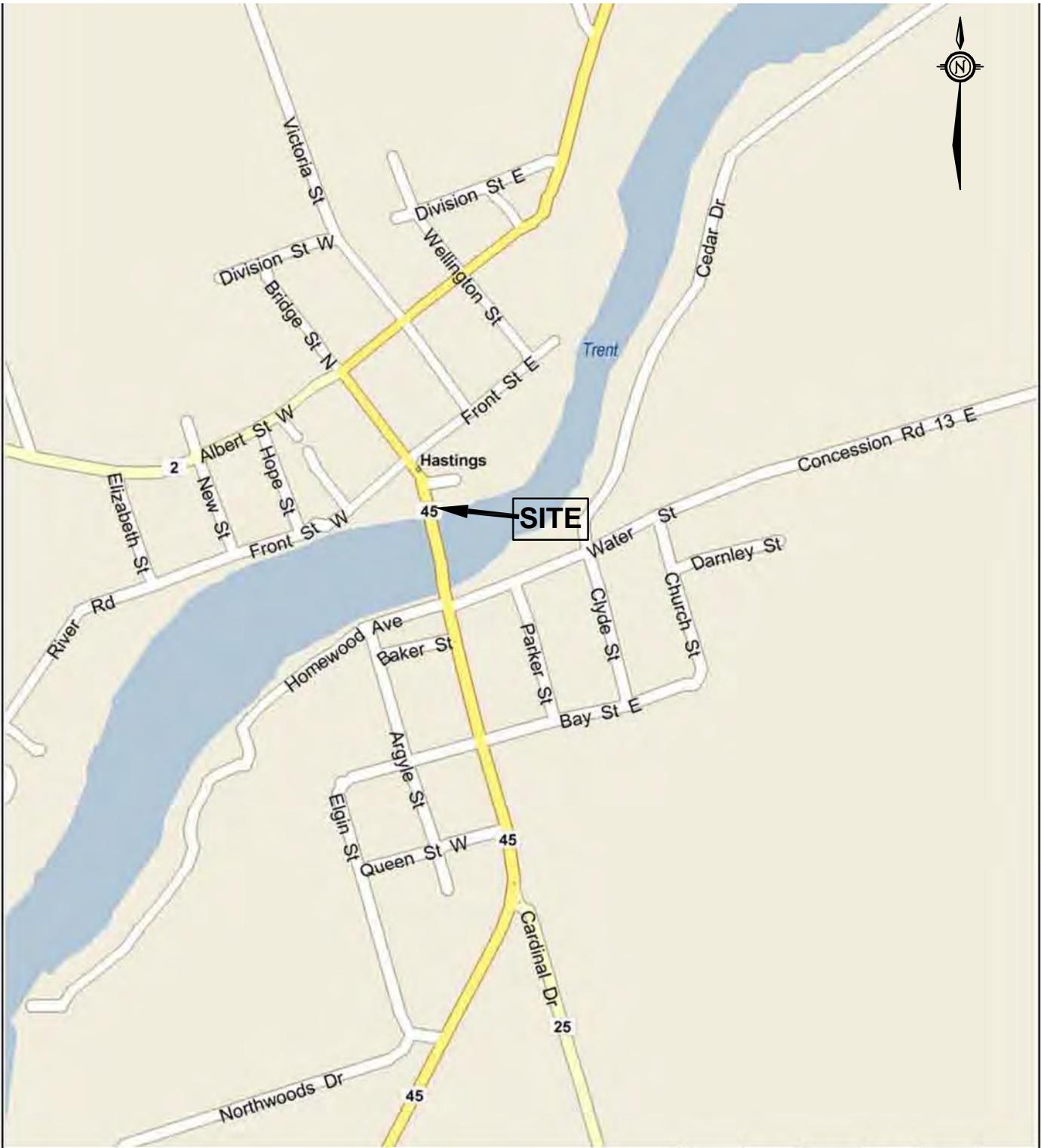
LDN\_BHS 11-1184-0022.GPJ GLDR\_LDN.GDT 5/30/11 DATA INPUT: MK MAY 2011

DEPTH SCALE  
1 : 50



LOGGED: AZ  
CHECKED:

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



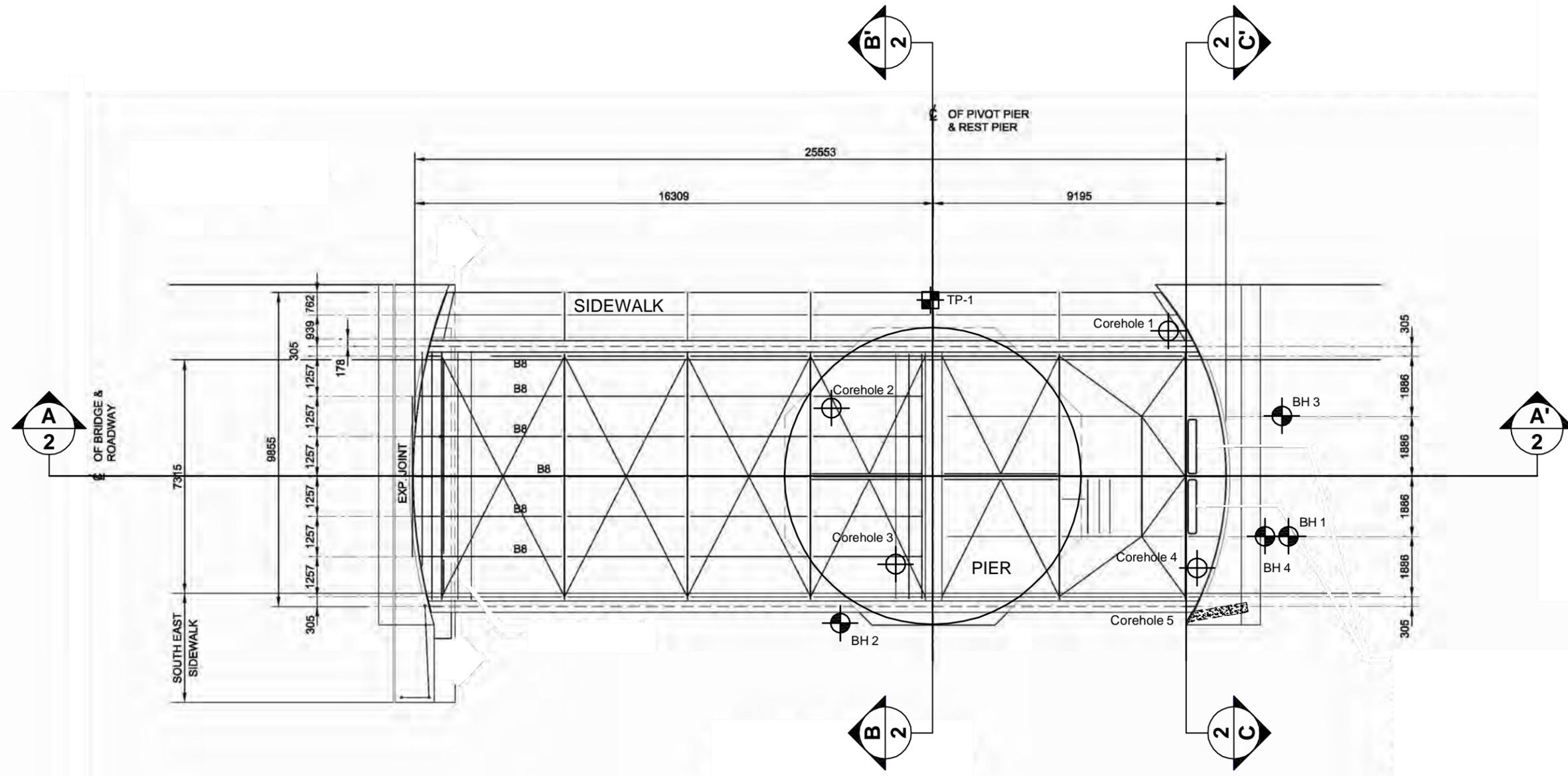
Base map by Microsoft Streets and Trips, 2008



ALL LOCATIONS ARE APPROXIMATE

PROJECT		Delcan Hastings Swing Bridge Trent-Severn Canal, Hastings, Ontario	
TITLE		<b>KEY PLAN</b>	
PROJECT No. 11-1184-0022		FILE No. AA01	
DESIGN		SCALE	AS SHOWN
CADD	PJV	REV.	
CHECK	May 2011	<b>FIGURE 1</b>	
REVIEW			





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

**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

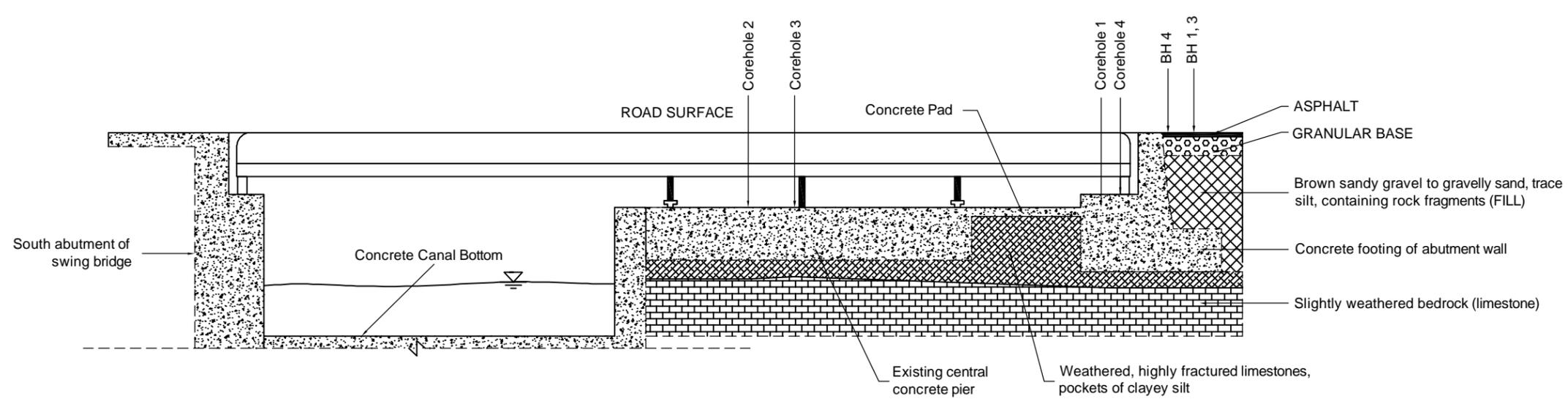
**LEGEND**

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

PROJECT		Delcan Hastings Swing Bridge Trent-Severn Canal, Hastings, Ontario	
TITLE		<b>BOREHOLE, COREHOLE AND TEST PIT LOCATION PLAN</b>	
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

NOT TO SCALE  
ALL LOCATIONS ARE APPROXIMATE

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

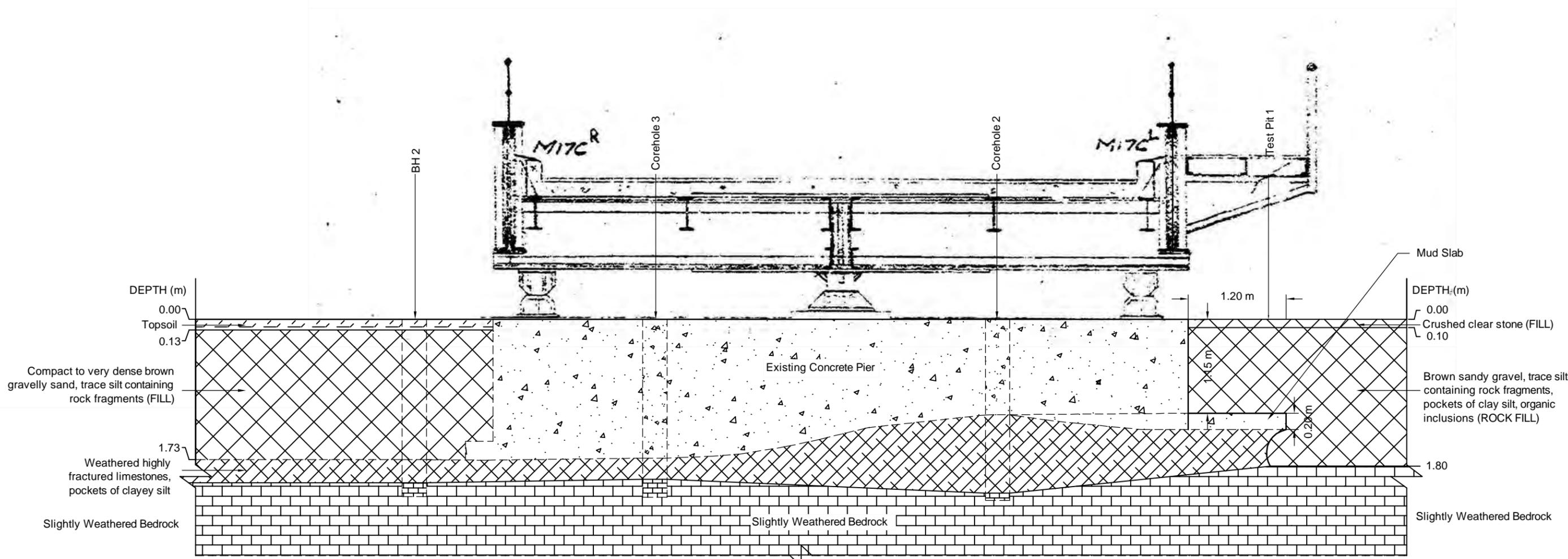
PROJECT		Delcan Hastings Swing Bridge Trent-Severn Canal, Hastings, Ontario	
TITLE		CROSS SECTION A - A	
PROJECT No.	11-1184-0022	FILE No.	AA01
DESIGN		SCALE	AS SHOWN
CADD	MK	MAY 2011	REV. 0
CHECK			
REVIEW			



FIGURE 3A

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

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.

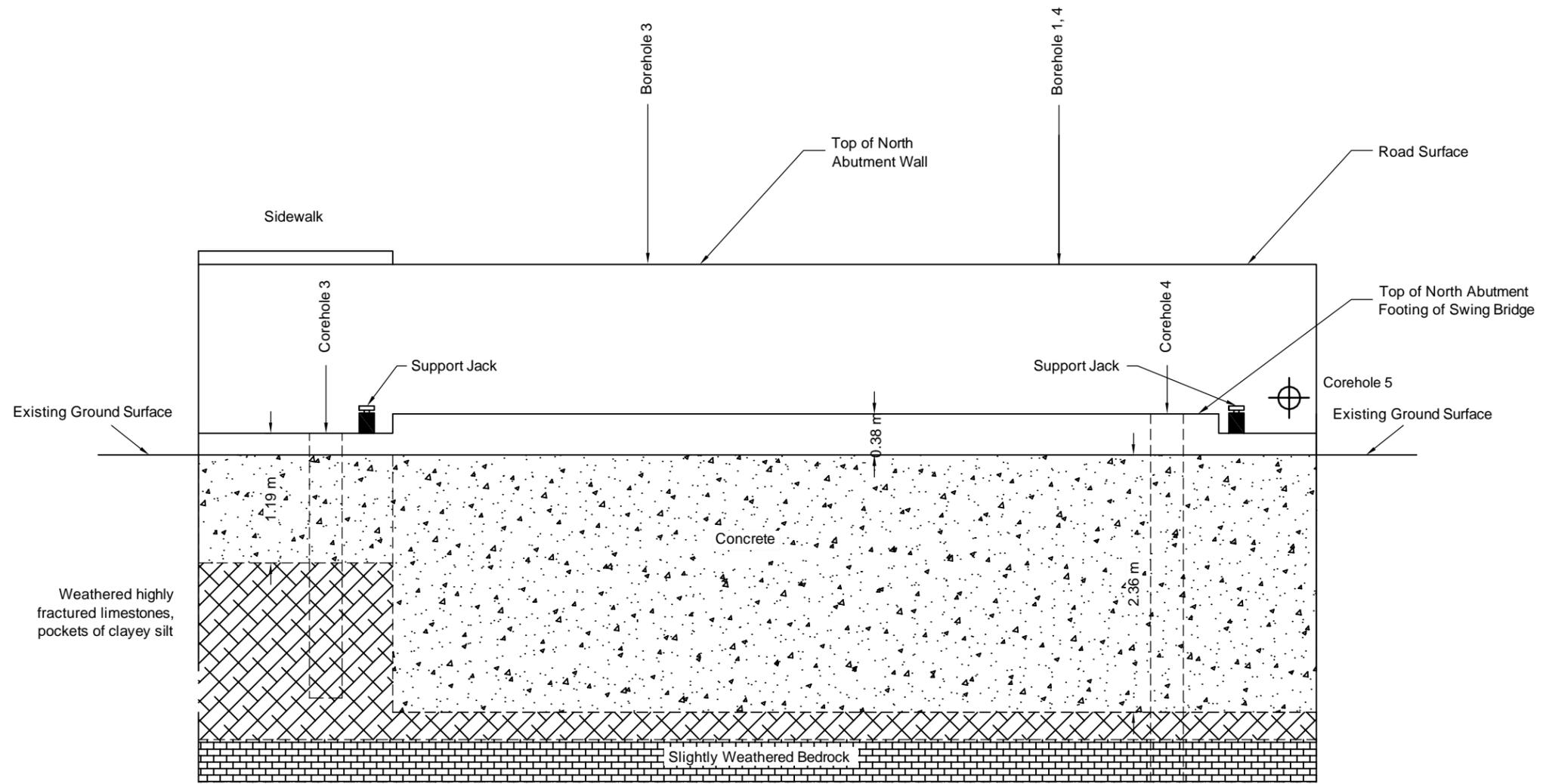
**NOTES**

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				<b>CROSS SECTION B - B</b>			
PROJECT No. 11-1184-0022		FILE No.		AA01			
DESIGN		SCALE	AS SHOWN	REV.	0		
CADD	MK	MAY 2011					
CHECK							
REVIEW							
				<b>FIGURE 3B</b>			

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



**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		<b>CROSS SECTION C - C</b>	
PROJECT No.	11-1184-0022	FILE No.	AA01
DESIGN		SCALE	AS SHOWN
CADD	MK	MAY 2011	REV. 0
CHECK			
REVIEW			

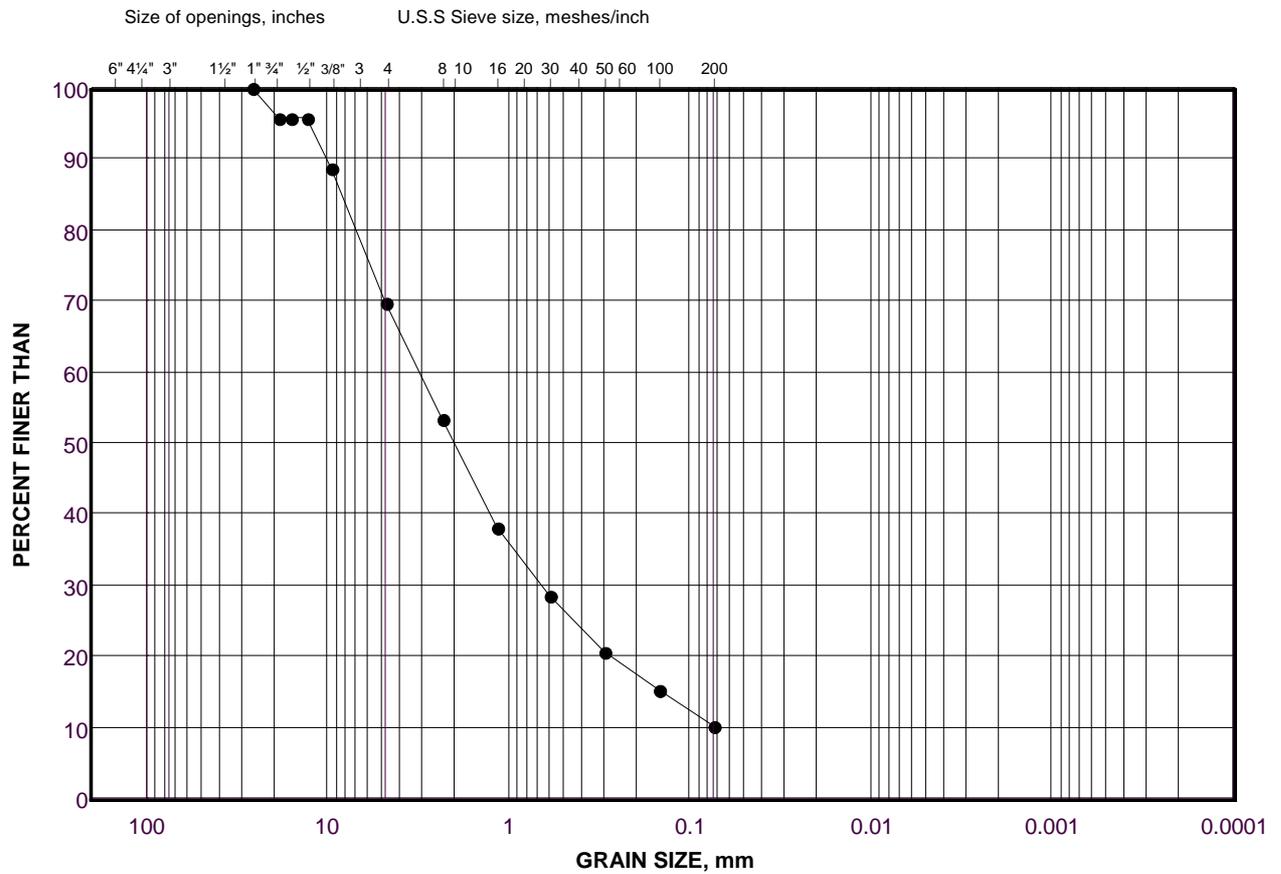


**FIGURE 3C**

# GRAIN SIZE DISTRIBUTION

## GRAVELLY SAND

FIGURE 4



<b>COBBLE</b>	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
<b>SIZE</b>	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: \_\_\_\_\_

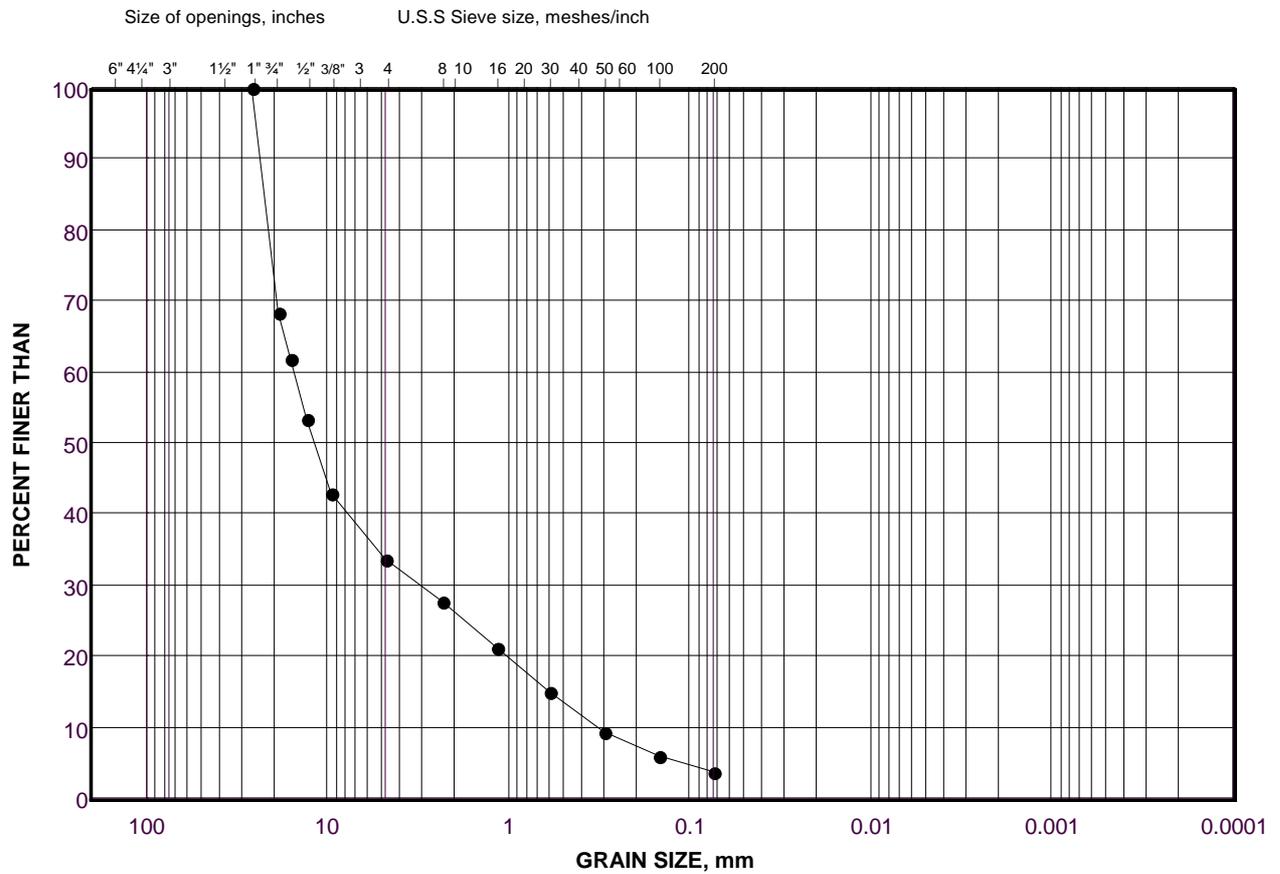
**Golder Associates**

Date: 30-May-11

# GRAIN SIZE DISTRIBUTION

## SANDY GRAVEL

FIGURE 5



<b>COBBLE</b>	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
<b>SIZE</b>	<b>GRAVEL SIZE</b>		<b>SAND SIZE</b>			<b>FINE GRAINED</b>

**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
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
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
Date:	May, 2011

**Golder Associates**

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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
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
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
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
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 deterioration 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
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
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 rebar; 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 rebar 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
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
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
Date:	May, 2011

**Golder Associates**

Inpitted by:	AZ
Checked by:	DL

# SITE PHOTOGRAPHS

Figure 6H



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.

Project No.	11-1184-0022
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
Checked by:	DL

# 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
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
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
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**Golder Associates**

Inputted by:	AZ
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
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
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
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
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
Date:	May, 2011

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# SITE PHOTOGRAPHS

Figure 6N



No.27: Location of Corehole 2.



No. 28: Locations of Coreholes 4 and 5.

Project No.	11-1184-0022
Date:	May, 2011

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Checked by:	DL

# SITE PHOTOGRAPHS

Figure 60



No.29: Photograph of the Corehole 1 cores.



No.30: Photograph of the Corehole 2 cores.

Project No.	11-1184-0022
Date:	May, 2011

**Golder Associates**

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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
Date:	May, 2011

**Golder Associates**

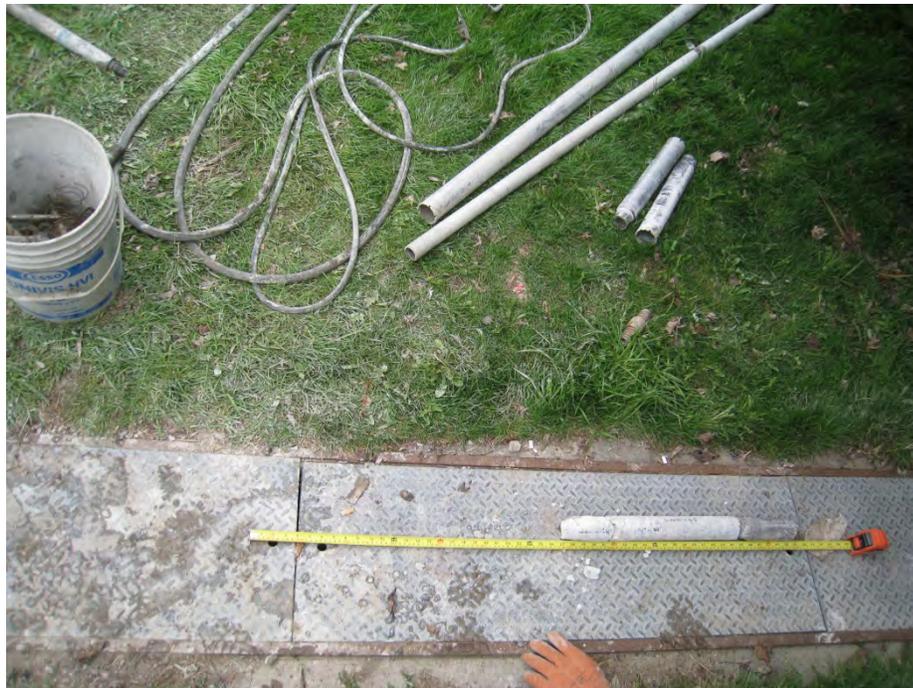
Inputted by:	AZ
Checked by:	DL

# 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
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
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
Date:	May, 2011

**Golder Associates**

Inputted by:	AZ
Checked by:	DL

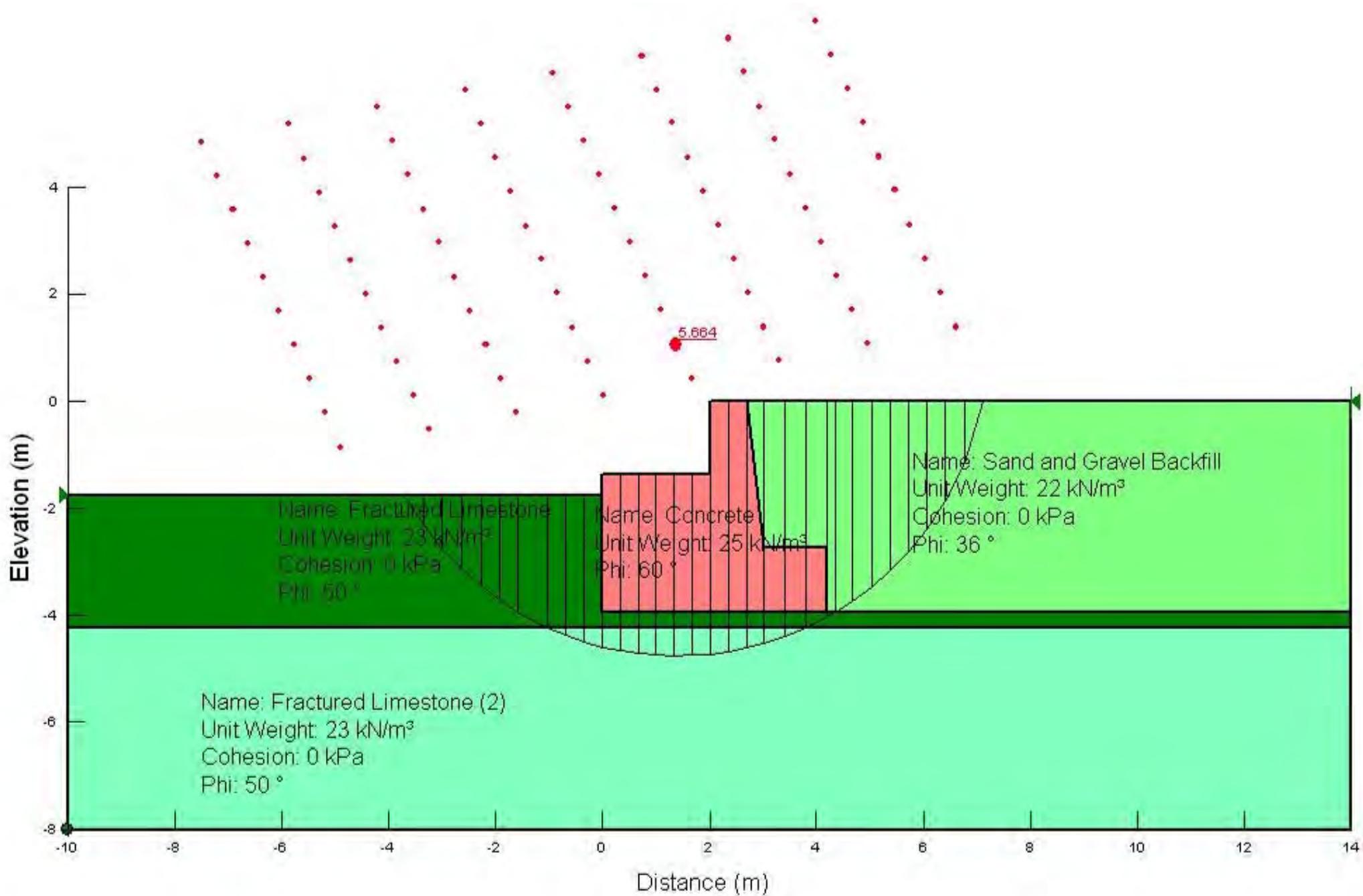


Figure 7- Global Stability Analysis of the North Abutment Wall



# **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



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## IMPORTANT INFORMATION AND LIMITATIONS TO THIS REPORT

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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

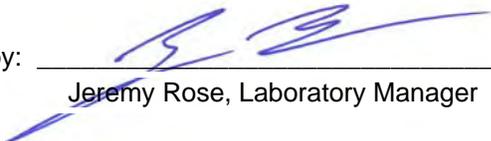
<b>Project</b>	<b>Hastings Swing Bridge, Hastings, ON</b>
----------------	--

**Date Received:** May 30, 2011

**Date Tested:** May 30, 2011

Core Number	Sa1	Sa2	Sa1	Sa2
<b>Location</b>	Hole 2	Hole 2	Hole 3	Hole 3
<b>Golder Lab Number</b>	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 <sup>3</sup> )	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

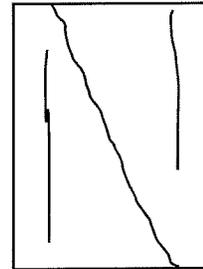
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	1.55

### SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	8.79	WATER CONTENT, (specimen) %	0.13
SAMPLE DIAMETER, cm	5.68	UNIT WEIGHT, kN/m <sup>3</sup>	26.24
SAMPLE AREA, cm <sup>2</sup>	25.32	DRY UNIT WT., kN/m <sup>3</sup>	26.21
SAMPLE VOLUME, cm <sup>3</sup>	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
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REMARKS: L/D Ratio not in accordance with ASTM Standard DATE:

5/26/2011

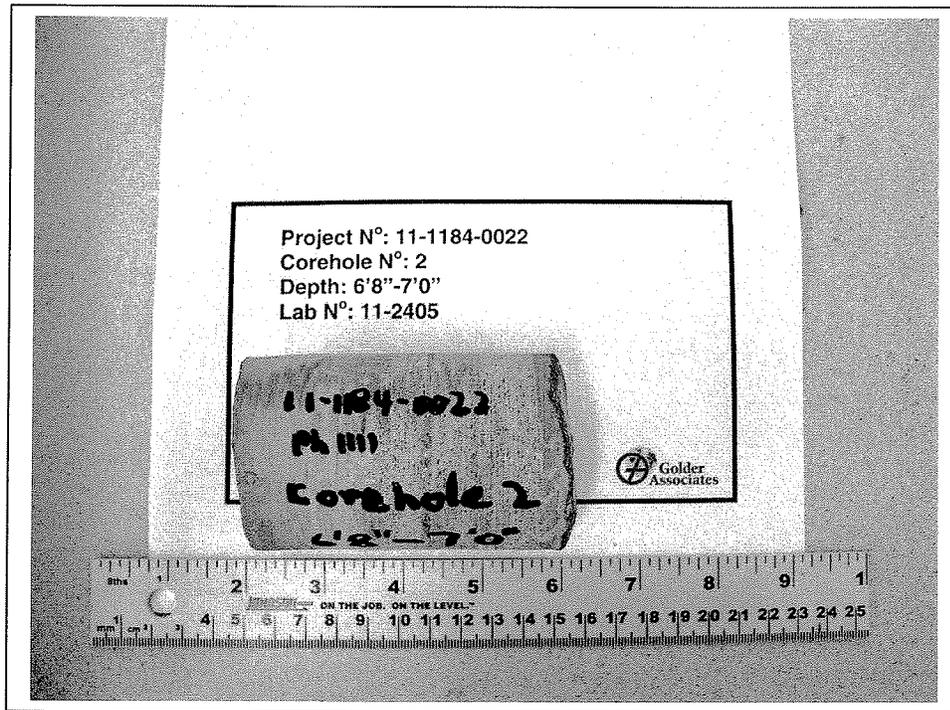
Checked By: *ML*

**Golder Associates**

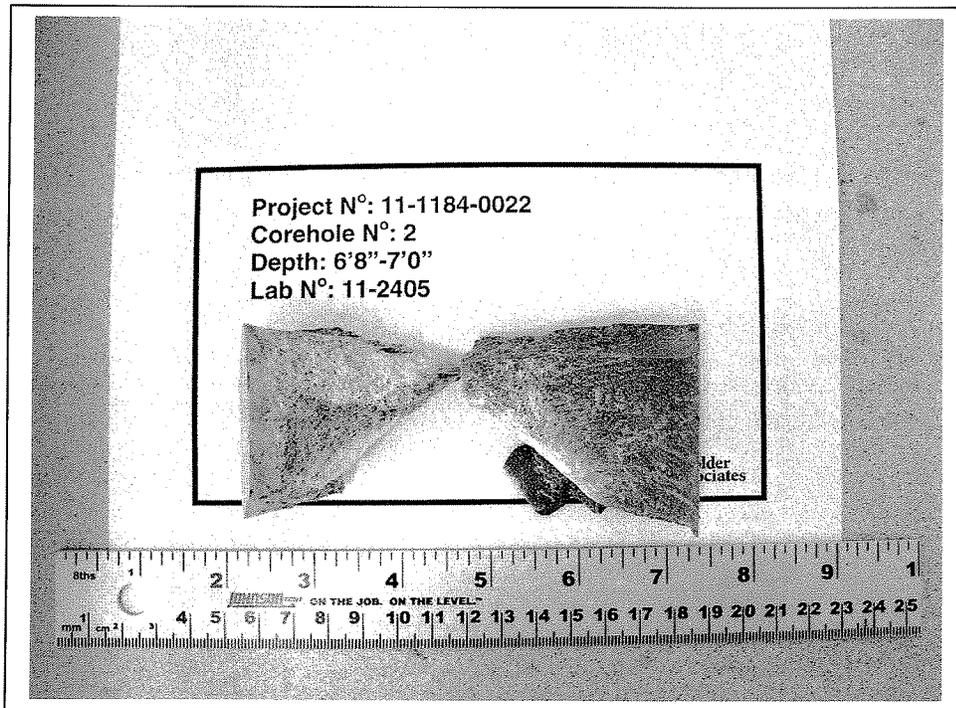
# UNCONFINED COMPRESSION TEST

ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

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

**Golder Associates**

Drawn AH  
Chkd. MH

# 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

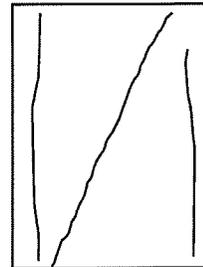
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.36

### SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	13.24	WATER CONTENT, (specimen) %	0.19
SAMPLE DIAMETER, cm	5.60	UNIT WEIGHT, kN/m <sup>3</sup>	26.19
SAMPLE AREA, cm <sup>2</sup>	24.63	DRY UNIT WT., kN/m <sup>3</sup>	26.14
SAMPLE VOLUME, cm <sup>3</sup>	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
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REMARKS:

DATE:

5/26/2011

Checked By: *MM*

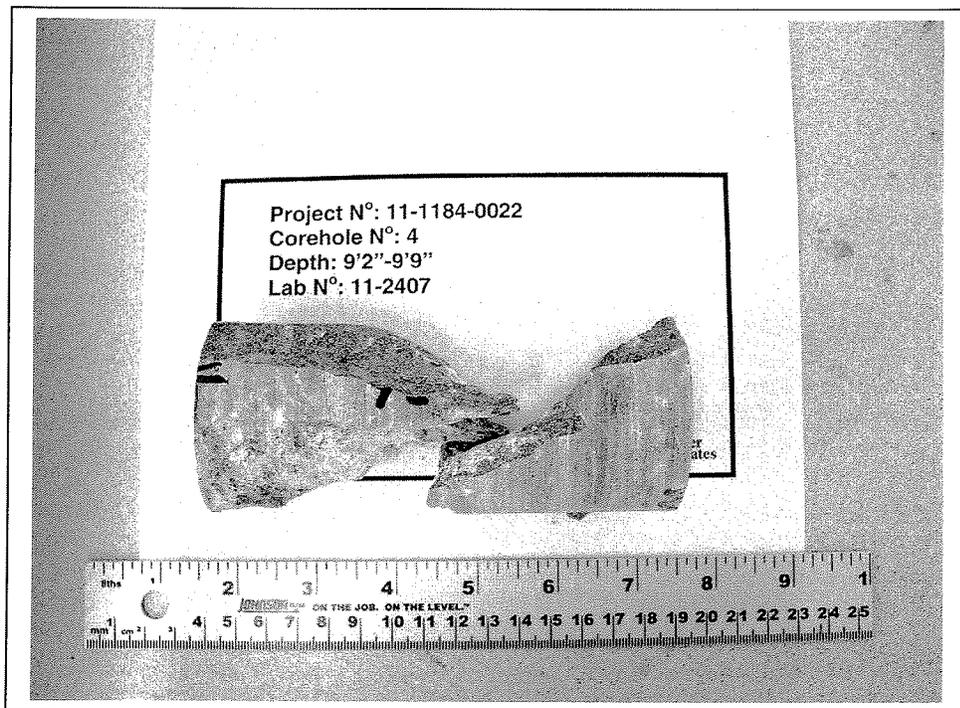
**Golder Associates**

UNCONFINED COMPRESSION TEST  
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

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

**Golder Associates**

Drawn AH  
Chkd. [Signature]

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# APPENDIX

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