

**FOUNDATION INVESTIGATION AND DESIGN REPORT
CONSTRUCTION ACCESS ROAD FOR BRIDGE REHABILITATION
QEW BRIDGE OVER CREDIT RIVER
MISSISSAUGA, ONTARIO
W.P. 2186-07-00
GEOCRES Number: 30M12-324**

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation carried out at the location of the proposed construction access road to the Credit River floodplain for rehabilitation of the existing Queen Elizabeth Way (QEW) bridge over the river.

The purpose of the investigation was to explore the subsurface conditions along the proposed access road alignment and, based on the data obtained, to provide a borehole locations and soil strata drawing, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. The subsurface conditions at the floodplain near the toe of the valley slope were also explored. A model of the subsurface conditions was developed from the data obtained during the course of the present investigation.

Thurber was retained by SNC Lavalin Inc. (SLI) to carry out the foundation investigation at this site on behalf of the Ministry of Transportation Ontario (MTO).

2 SITE DESCRIPTION

The access road site is located at the northwest quadrant of the QEW bridge on the tableland just west of the Credit River valley. The existing QEW Credit River bridge comprises of six spans and holds six lanes of traffic.

The river valley is incised up to approximately 18 m below the surrounding tableland. The valley slopes are steep and are predominantly formed through shale bedrock. On the tableland, shale is also present at shallow depth underneath overburden soils or fill along the access road alignment. The drainage at the site flows towards Credit River, which flows southward to Lake Ontario.

On the plateau to the north of QEW, the terrain is largely flat with the ground surface varying between Elevations 94 m and 98 m. Vegetation is moderate consisting mainly of tall grass, shrubs and occasional small to large trees.

The project area is located adjacent to the Credit River valley. From published geological information, this area is situated within the physiographic region known as the Iroquois Plain. In this area, the relatively thin native soil deposits typically consist of cohesive soils (some tills) overlying shale bedrock of the Georgian Bay Formation. The till is known to consist of shale and limestone fragments. Alluvial deposits in the form of clayey silts, silts and fine sands are present within the river floodplain.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out in two phases. Phase one consisted of drilling on the plateau and phase two involved drilling on the river floodplain. The field work was carried out between September 30 and October 5, 2010 for phase one and between November 4, 2010 and November 5, 2010 for phase two. Three boreholes were drilled and sampled to depths between 12.2 m and 24.4 m during phase one. An additional four boreholes were drilled and sampled to depths ranging from 0.8 m to 4.3 m for phase two. The boreholes were numbered 10-01 to 10-05, 10-03A and 10-03B. The approximate locations of all boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix C.

The planned borehole locations were staked and/or marked in the field by Thurber, but some of them needed to be relocated to avoid overhead and underground utilities. Utility clearance was obtained at all borehole locations by Thurber prior to drilling. The northing and easting co-ordinates and ground surface elevations of the completed boreholes were provided by J.D. Barnes.

Walker Drilling Limited of Utopia, Ontario supplied a track mounted Diedrich D-50 Turbo drill rig and conducted the drilling, sampling and in-situ testing operations for all the boreholes in phase 1 (10-01, 10-02 and 10-05). Auger drilling techniques were used to advance the boreholes through soils and weathered rock. Soil and weathered rock samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Once the top of weathered shale was established and the shale was deemed to be suitable for coring (reasonable recovery anticipated), Boreholes 10-01, 10-02 and 10-05 were further advanced 9.5 m to 19.8 m into bedrock by HQ size rotary coring techniques to recover core samples. Phase 2 boreholes (10-03, 10-03A, 10-03B and 10-04) were drilled using a portable tri-pod drilling rig. Soil samples were obtained using a split spoon sampler in conjunction with SPT. The split spoon sampler was used to advance the phase 2 boreholes to refusal.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in Boreholes 10-01 and 10-02 to permit monitoring of the groundwater level within the bedrock. Another shallow piezometer was installed in Borehole 10-02 within the soil. At this site, 19 mm diameter Schedule 40 PVC pipes with 1.5 m long slotted screens were installed in the boreholes. The sand screen surrounding the pipe was about 2 m to 3 m in length. Bentonite holeplug seals were placed above the sand screen in each installation.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

All rock cores were logged, and properties including the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

On completion of drilling and sampling, the boreholes deeper than 3 m below existing ground surface were grouted with bentonite to the ground surface. Borehole completion details are presented in Table 1 immediately following the text.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A. Selected samples including those of weathered shale were subjected to gradation analysis. Atterberg Limits Tests were performed on some of the samples. The results of this testing program are presented on the Record of Borehole sheets in Appendix A and on the figures in Appendix B.

Point load and Unconfined Compression (UC) testing was carried out at selected locations on the rock cores and the results are shown in Table 2 attached immediately following the text in Appendix B, and on the Records of Boreholes in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these records and on the "Borehole Locations and Soil Strata" drawing in Appendix C. General description of the stratigraphy is given in the following paragraphs. The factual information established at the borehole locations governs any interpretation of site conditions.

In general, the site is underlain by relatively thin deposits of overburden soils overlying shale bedrock. The overburden soils range in thickness from about 0.6 to 3.7 m. The overburden generally consists of topsoil/fill, clayey silt and silty clay.

5.1 Topsoil

Topsoil was encountered in all boreholes except 10-03A and 10-03B to depths ranging between 50 and 150 mm as shown below.

Borehole	Topsoil Thickness (mm)
10-01	150
10-02	150
10-03	50
10-04	50
10-05	150

Topsoil thickness may vary between and beyond the boreholes.

5.2 Clayey Silt

Alluvial deposits were encountered in the Credit River floodplain. These soils consist of a clayey silt matrix, with some sand, trace gravel, mixed with occasional layers of shale fragments, wood fibres and organics. The thickness of the clayey silt interlayers ranges from 0.8 m to 3.7 m. The underside of the clayey silt layer ranges between Elevations 72.6 and 75.7 m. The clayey silt has a typically stiff to very stiff consistency as indicated by SPT 'N' values of 8 to 26 blows per 0.3 m penetration. The moisture content of the clayey silt layer ranges from 10% to 55%.

Some shale fragments were encountered within the clayey silt layer at and just below 0.6 m depth, or Elevation 75.6 m, in Boreholes 10-03A and 10-03B.

Grain size analyses conducted on several samples retrieved from this unit are presented in Figures B1 and B2. These results indicate that the clayey silt contains approximately 0 to 3% gravel, 23% to 44% sand, 36 to 59% silt, and 15% to 22% clay. Atterberg Limits tests were also conducted on a representative sample of this soil and the results are presented in Figures B5. The clayey silt sample had a measured plasticity index of 15% and a corresponding liquid limit of 33%, respectively. These values are indicative of a cohesive soil of low plasticity (group symbol of CL).

5.3 Peat

A layer of buried peat with occasional rootlets was encountered in the Credit River floodplain within the clayey silt layer in Boreholes 10-3A and 10-3B. The thickness of the peat varies from 0.6 m to 1.0 m. The underside of the peat layer varies from Elevations 72.6 to 73.2 m. The amorphous peat has a largely firm consistency as indicated by SPT 'N' values varying from 4 to 7 blows per 0.3 m penetration. The moisture content of selected peat samples ranged from 55% to 87%.

5.4 Silty Clay

In Boreholes 10-01, 10-02 and 10-05 on the plateau, the topsoil is underlain by a deposit of silty clay with sand which extends to depths ranging from 0.6 to 3.2 m below existing ground surface, or approximate Elevations 91.2 to 95.6 m. The silty clay is typically brown in colour. Roots, rootlets, shale fragments and sand seams were also present within the silty clay.

Standard Penetration Tests (SPT) conducted within this deposit gave 'N' values ranging from 12 to 39 blows per 0.3 m penetration indicating a stiff to hard consistency. The measured moisture contents of samples recovered from this unit ranged from about 11% to 23%.

Grain size analyses conducted on several samples of this soil are presented in Figures B3. These results indicate that the silty clay contains approximately 0 to 10% gravel, 5 to 59% sand, 22 to 47% silt and 16 to 58% clay. Atterberg Limits tests were also conducted on representative samples from this stratum and the results are presented in Figure B6. The silty clay samples had measured plasticity indices ranging between 15% and 24%, and corresponding liquid limits ranging from 37% to 55%, respectively. These values are indicative of a cohesive soil of intermediate to high plasticity (group symbol of CI to CH).

5.5 Shale Bedrock

The overburden soils described above are underlain by shale bedrock. In the plateau boreholes, weathered shale was encountered at depths of between 0.6 and 3.2 m below existing ground surface, or Elevations 91.2 to 95.6 m. The depth to shale bedrock increases towards the Credit River valley. In the floodplain boreholes, the depth to shale bedrock ranges from 0.8 to 3.7 m, or Elevations 72.6 to 75.7 m. Augering and SPT sampling within the weathered shale was carried out in all boreholes except for 10-03, where split spoon sampler resistance was encountered above the shale. Relatively higher resistance to augering was encountered at several locations within the weathered shale, inferring the presence of harder limestone interbeds. Bedrock was proven by coring beyond the augered depth in Boreholes 10-01, 10-02 and 10-05. The following table summarizes the depths and elevations of weathered shale encountered at the borehole locations.

Borehole Number	Depth to Weathered Shale (m)	Top of Weathered Shale Elevation (m)
10-01*	0.6	95.6
10-02*	3.2	91.2
10-03A	3.7	72.6
10-03B	3.7	72.6
10-04	3.5	72.8
10-05*	1.4	93.9

* Proved by coring below augered depth

The shale encountered at this site is fine grained, thinly bedded, reddish brown near the surface but predominantly grey in colour that is typical of the Georgian Bay Formation. The shale is interbedded with hard, grey limestone with occasional clay seams. The shale is typically in a highly to moderately weathered state within the upper 2 m. Below this zone, the degree of weathering decreases with depth, and the rock becomes moderately to slightly weathered and harder with depth. The hard limestone interbeds typically range from 25 mm to 100 mm in thickness, with occasional layers up to 150 mm.

Total Core Recovery (TCR) of the bedrock was generally between 95% and 100%. The Rock Quality Designation (RQD) values ranged between 30% and 100%, but most values varied from 80% to 100% indicating a typically fair to good rock quality. The lower RQD values of the order of 30% were associated with the first core run within the upper weathered zone in several boreholes.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, generally ranged from 0 to 5 with occasional values between 10 and 20. Occasional zones of multiple fractures were present within the upper weathered portion of the rock. The discontinuities and bedding planes in the rock cores were largely horizontal with occasional sub-vertical joints.

Point load tests were carried out at selected intervals on the rock cores, and Unconfined Compression (UC) tests were conducted on selected rock cores. The Unconfined Compressive Strengths (UCS) of the rock cores, as inferred from the point load test results and directly measured from UC tests, are summarized in Table 2 attached immediately following the text. The strength profiles with depth are also presented on the Records of Boreholes in Appendix A. Laboratory details of the UC tests are presented in Appendix B.

The point load test results infer that the UCS of the cores of shale range from less than 1 to about 13 MPa. The strength of the shaley limestone ranges from about 2 to 48 MPa. The strength of the hard limestone interbeds range between 28 MPa and 200 MPa.

5.6 Groundwater Conditions

Groundwater conditions were observed in the open boreholes during and upon completion of drilling. Standpipe piezometers were installed and sealed in Boreholes 10-01 and 10-02 to permit longer term groundwater monitoring. To date, four sets of piezometric readings has been obtained since completion of installation and is presented in the following table.

Borehole	Date	Ground Surface Elevation (m)	Groundwater	
			Depth (m)	Elevation (m)
10-01 (sealed in shale)	October 4, 2010	96.2	4.5	91.7
	October 5, 2010		9.2	87.0
	October 12, 2010		9.3	86.9
	December 17, 2010		10.0	86.2
10-02 (sealed at soil- shale interface)	October 4, 2010	94.4	Dry	-
	October 5, 2010		Dry	-
	October 12, 2010		Dry	-
	December 17, 2010		Dry	-
10-02 (sealed in shale)	October 4, 2010	94.4	8.5	85.9
	October 5, 2010		8.5	85.9
	October 12, 2010		8.7	85.7
	December 17, 2010		9.3	85.1

It is anticipated that the groundwater level at the floodplain is largely governed by the water level in the Credit River.

It is noted that all groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events.

6 MISCELLANEOUS

Borehole locations and ground surface elevations were provided to Thurber by J.D. Barnes Ltd. as arranged by SNC-Lavalin.

The drilling and sampling equipment was supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The field work was supervised on a full time basis by Messrs. Stephane Loranger and George Azzopardi of Thurber Engineering Ltd.

Laboratory testing was carried out at Thurber's Laboratory in Oakville, Ontario.

Overall supervision of the field program was conducted by Dr. Sydney Pang, P. Eng. Compilation of data and preparation of the report were carried out by Dr. Sydney Pang, P. Eng, P.Eng and Mr. Luke Gilarski.

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



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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report provides interpretation of the geotechnical data in the factual report and presents foundation design recommendations to assist the design team to select and design a suitable construction access road from the upland down the steep valley slope to the floodplain of the Credit River for bridge rehabilitation. The construction access is required for the delivery of materials and construction equipment for the building of a temporary bridge platform and subsequent rehabilitation from the underside of the deck of the Credit River Bridge. It is understood that this access road will be in active use for two construction seasons after which it will be left in place for 10 years or longer for future rehabilitation and/or construction of a new crossing. Foundation design recommendations are also provided for temporary footings required to support the temporary bridge platform to be erected under the bridge. The general arrangements of the proposed works are outlined on preliminary drawings provided by SLI.

8 OPTIONS FOR CONSTRUCTION ACCESS ROAD

It is understood that SLI has considered several alternatives other than an access road down the steep valley slope. These alternatives include the following:

- Delivery of equipment to the floodplain by barge,
- Lower equipment to the floodplain or carry out the rehabilitation work from the top of the existing bridge deck.

Both alternatives are not feasible for reasons such as no suitable landing area is available for launching a barge, low headroom of other bridges along the river, limited strength of the deck and unacceptable lane closure requirements associated with working from the bridge deck. As a result, SLI has concluded that creating an access route through the plateau and down to the floodplain is the only practical way to facilitate the bridge rehabilitation work. A site access review indicates that the only available location for a construction access road is at the northwest quadrant of the Credit River Bridge under the existing Hydro One corridor via Mississauga Road north of the QEW.



Construction of the access road from other quadrants is not feasible as they would be too close to the existing residential areas or the floodplain is too narrow.

In order to select an access route alignment involving minimum earthworks and cutting into the valley slope, a site visit was carried out to the northwest quadrant on December 10, 2010 which was attended by representatives from the MTO Foundations office, MTO project management, SLI and Thurber. Consideration was given to making use of what appeared to be the remnants of an old sidehill access route down the valley slope used during the previous rehabilitation and reconstruction of the bridge in the 1980's, instead of a new access road involving deep cuts into the valley slope. Subsequent studies carried out by SLI indicated that it would not be feasible to make use of the old access route due to the following reasons:

- A very steep grade of up to 18%; additional cut and fill will be required to reduce the grade to 10%.
- Environmental regulations prohibiting placement of fill within a 30 m zone from the river flow channel,
- Presence of an oil pipeline directly below and overhead cables (mounted on poles) directly above the route alignment.

In conclusion, a new access road involving deep cuts into the valley slope remains to be the only feasible alternative. The foundation discussion and recommendations presented herein relate to the design and construction of a new access road.

The proposed primary access road will be up to 550 m in length extending from Stations 0+000 to 0+550. Between Stations 0+000 and 0+300, the road will be formed in a cut of up to about 2 m depth. A sloped open cut is currently proposed for this section. The primary access road will generally follow the existing ground surface beyond which the road will be formed in cut with a road grade of 8% to 10% to reach the floodplain. The cut will be up to the order of 13m depth below existing grade at the west plateau of the river valley.

It is understood that consideration is also being given to constructing a secondary access road to connect to an elevated temporary construction access deck. This secondary access is to branch out from the primary access at approximate Station 0+450 beyond which it would make a 90° turn south and continue perpendicular to the bridge centreline.

The discussion and recommendations presented in this report are to be read in conjunction with the drawings showing the proposed design provided by SLI (Appendix C) and on the factual data obtained during the course of this investigation.

9 ACCESS ROAD CUT

9.1 Cut Design Criteria

The following criteria are to be considered in the design of the access road cut.

- A minimum design life of 12 years.
- Maximum access road grade of 10%.
- A 30 m setback from the river flow channel with respect to fill placement.
- Stay a minimum 15 m away from Hydro One poles, and a minimum 3 m away from Enersource poles.
- Avoid utility relocation as this will affect the construction schedule.
- Number of equipment passages directly above the oil pipelines should be minimized.

9.2 Cut Configuration Alternatives

This section presents discussions on road cut alternatives, and foundation recommendations for feasible and/or preferred options for this site.

For access road cut slopes formed through soil and the underlying shale bedrock, the options considered are as follows:

- Option 1 – Full depth inclined slopes through soils and shale bedrock
- Option 2 – Inclined slopes through soils and highly weathered shale overlying vertical or near vertical cuts in intact shale protected by shotcrete
- Option 3 – Inclined slopes through soils and highly weathered shale overlying vertical or near vertical cuts in intact shale supported by shoring
- Option 4 – Inclined slopes through soils and highly weathered shale overlying multi-tiered sub-vertical slopes with intermediate horizontal benches in intact shale.

A comparison of these cut alternatives based on advantages and disadvantages of each is included in Appendix D.

Discussions on the options listed above are as follows:

Option 1

The full depth inclined slope option involving slope inclinations of 2H : 1V in soils and highly weathered shale (upper 2 m) overlying 1H : 1V or flatter inclinations within the underlying intact shale is not recommended. The reasons are that this involves extensive excavation and eventual backfilling, erosion protection requirements, site constraints involving setback distances from hydro poles and oil pipelines etc., and requirements for periodic maintenance after completion of the current rehabilitation work.

Option 4

Slope inclinations of 2H : 1V in soils and highly weathered shale (upper 2 m), overlying 3 m high, 1H : 4V inclined tiers in intact shale with a horizontal 3 m wide bench between each pair of adjacent tiers have also been considered. The sub-vertical shale faces are less exposed to rainfall and surface runoff, and do not collect as much snow, whereas the benches collect ravelling rock fragments. However, this option also involves substantial excavation, requirements for erosion protection and periodic maintenance after completion of the current rehabilitation work, and is therefore not recommended.

Options 2 and 3

From a foundation/geotechnical perspective and to minimize excavation quantity and maintenance, the feasible alternatives are to have a 2H : 1V slope through soils and highly weathered shale (upper 2m) overlying a shotcreted or shored vertically sided cut in intact shale.

9.3 Access Road Cuts

The current design configuration of the cut is as follows:

- Stations 0+000 and 0+310 – depth of cut is up to 2 m below the existing ground surface. The proposed road grade varies between approximate Elevations 96 and 100 m. Based on extrapolating the results of Borehole 10-01 of the present investigation and previous boreholes along the QEW advanced by others, it is anticipated that this cut would be formed predominantly through the stiff to very stiff silty clay into the upper, highly weathered portion of the shale bedrock.
- Stations 0+310 and 0+483 – depth of cut increases uniformly at an 8% to 10% grade up to about 13 m below existing ground surface at the crest of slope. The proposed base of cut slopes from approximate Elevations 98 to 82 m. Results of Boreholes 10-01, 10-02 and 10-05, inclusive, indicate that the cut will be formed through up to 3 m of stiff to hard silty clay well into the intact shale bedrock.
- Stations 0+483 and 0+550 – depth of cut decreases along the valley slope until the floodplain is reached. The proposed base of cut decreases from approximate Elevations 82 to 76 m. Results of Boreholes 10-03, 10-03A, 10-3B and 10-04 indicate that the cut will be formed through intact shale bedrock. Minor filling not exceeding 1 m is required at the toe of the slope.

9.3.1 Earth and Highly Weathered Shale Cut Design

The extent of the earth cuts will be limited at this site. The access road cut will extend through a thin veneer of topsoil into 0.4 to 3.0 m of typically stiff to very stiff and occasionally hard silty clay, with sand and trace gravel. Shale fragments may be

encountered within this deposit. The upper 2 m of shale is typically highly weathered and is considered to behave in a manner similar to earth. A typical sideslope inclination of 1H : 1V used for a temporary cut is not applicable for this project due to the relatively long service life of more than 10 years. It is recommended that sideslopes through soils and highly weathered shale be formed with inclinations not steeper than 2H : 1V to maintain global stability.

In areas where cut slopes in highly weathered shale are formed at inclinations not steeper than 2H : 1V, the exposed shale must be temporarily protected from erosion until adequate vegetation growth is established.

After starting to branch away from the primary access at approximate Station 0+450, the secondary access typically runs parallel to and at an elevation higher than the primary access, except at the 90° turn near the crest of the slope where the secondary access turns away and slopes to a lower elevation which remains constant until termination under the QEW bridge deck. Within this section on the west abutment forward slope of the existing bridge, a small shoring wall will be required to form the secondary access.

9.3.2 Cut Design in Intact Shale

Rock cutting up to 10 m below bedrock surface will be required for the access between approximate Stations 0+300 and 0+510. Below the upper 2 m of highly weathered shale, a shotcreted or shored, vertically sided excavation are considered feasible alternatives for the intact shale for this project. Similar shoring or protection system will be required for retaining the existing high mast pole foundation near Station 0+050. It is important to note that exposed shale will be subject to weathering and rapid deterioration upon exposure to air and water. The exposed rock face must therefore be protected shortly after excavation.

9.3.2.1 Shotcreting of Rock Face

A technically feasible option is to form vertical or near vertical cuts in the intact shale and the resulting rock face protected by applying shotcrete on wire mesh that is to be secured by rock dowels. The shotcrete on wire mesh protects the underlying rock from further weathering and deterioration. Rock shotcrete should be designed by specialists experienced in such designs and applied as per SP 299S08, a copy of which is attached in Appendix F. The shotcrete must be of good quality and applied by specialist contractors experienced in such construction methods (SP 299S08). Below a design groundwater elevation of 87 m and where water seepage from the rock face is evident, it is important to provide drain (weep) holes to prevent build-up of hydrostatic pressure and ice behind the shotcrete protective layer. Formation of ice at the interface between shotcrete and the rock would result in separation and deterioration of the shotcrete layer. Rock drains should be provided as per SP 299S09, a copy of which is attached in Appendix F. All rock surfaces should be scaled of loose rock and rubble prior to placing the the wire mesh and shotcreting.

The design of the required shotcreting should be carried out by specialists experienced in such designs.

9.3.2.2 Shoring of Rock Cut

Based on the site conditions and our understanding of the project requirements, consideration may also be given to using soldier piles and wood lagging for the shoring system. Foundation recommendations for shoring system design are provided in a later section of this report.

The excavation, soldier pile and lagging board installation should be co-ordinated such that the duration of rock face exposure is minimized. Prior to installing the lagging boards, the exposed rock faces should be scaled and any loose rock fragments should be removed. The space between the lagging and the rock face should be packed with free-draining granular materials. In areas of over-excavation, deteriorated shale surface, or where it is otherwise impractical to backfill with granular materials, lean mix concrete should be used behind the lagging to fill the voids. At elevations below the groundwater level and where seepage from the rock face is evident, it is recommended that a layer of non-woven geotextile be placed against the back face of the lagging boards prior to putting in the backfill materials. The geotextile serves as a filter/separator to mitigate migration of fines with the seepage water. For positive drainage, the granular backfill should be connected to drainage ditches along the edge of the access road. The contract documents should contain an NSSP specifying these detailed requirements. Suggested wordings for this NSSP are provided in Appendix F.

9.3.3 Erosion Protection

Vegetation cover should be established on all exposed soil and shale slopes to protect against surficial erosion. General reference may be made to OPSS 572 for more detailed requirements, where applicable.

9.3.4 Access Road Pavement Design

The pavement of the access road may consist of compacted granular materials placed on the rock subgrade. The actual thickness of the granulars depends on the anticipated traffic volume but should not be less than 600 mm. The minimum thickness of each type of granular materials is as follows:

OPSS Granular A	200 mm
OPSS Granular B Type II	400 mm

The designer may choose to alter the combination of granular thicknesses as required.

The road surface should have a minimum 2% grade towards side ditches running parallel to the road. For shotcreted or shored cuts, a longitudinal ditch on one side of the road should be adequate provided that the ditch capacity is sufficient to handle the anticipated volume of

water. Ditches on both sides of the road would be required to accommodate ravelling rock pieces if there are exposed rock faces. The sideslope of the ditches should not be steeper than 2H : 1V, and should be protected from erosion.

9.3.5 Maintenance

Maintenance of the access road pavement and the cut will be required during and after the currently proposed phase of rehabilitation. Such maintenance may be carried out on an as-needed basis. The items of maintenance may include repairing of spalling shotcrete and wire mesh, cleaning of ditches, grading and resurfacing of the access road with additional gravel. It is recommended that provisions be included in the contract to address these maintenance requirements.

10 EXCAVATION

10.1 General

All temporary excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the overburden fill, silty clay may be classified as Type 3 soils. The upper 2 m (highly weathered portion) of the shale may be classified as a Type 2 material.

Vertical sides of temporary excavations formed in the Georgian Bay Shale bedrock below the upper weathered zone should be stable in the short term, i.e. no more than three weeks. If the lagging boards are not installed and the space behind is not backfilled within that time, the shale may deteriorate upon exposure to air and water, and start to slough into the excavation. If prolonged exposure is inevitable or occurs unexpectedly, all disturbed and deteriorated materials on the rock face should be removed and the void filled with granular backfill or lean mix concrete. Suggested wordings for an NSSP are provided in Appendix F.

10.2 Earth Excavation

The extent of earth excavation will be relatively small at this site. The excavation will extend through 0.5 m to 3 m of stiff to very stiff silty clay. Some shale fragments may be encountered in these soils. Given a service life of more than 10 years, all open cut slopes should be designed for an inclination of 2H : 1V or flatter. The upper 2 m of weathered shale should be considered as soil for the purpose of slope inclination requirements.

10.3 Rock Excavation

The bulk of the excavation to form the access road cut will be extended into the relatively sound shale with hard limestone interbeds. It is anticipated that the shale becomes progressively harder with depth. Heavy excavating equipment, ripping machinery and rock breakers/splitters will be required to break up hard limestone and other intact shale slabs.

The contract documents should contain an NSSP alerting the contract bidders that rock excavation may require the use of such equipment. Suggested wordings for this NSSP are provided in Appendix F.

Any rock excavation should be carried out in accordance with OPSS 902.

It is also recommended that chemical testing be conducted on the shale bedrock for assessing disposal options prior to issuing the contract.

Blasting is likely not an acceptable alternative at this site.

10.4 Shoring

Vertical cuts through shale may also be supported by shoring. An item titled "Protection System" as per OPSS 539 should be included in the contract documents. It is recommended that the shoring at this site be designed to Performance Level II requirements.

Detailed design of shoring systems should be the responsibility of the Contractor. A feasible alternative for shoring at this site is a soldier pile and lagging wall. The soldier piles will need to be socketted in shale below the base of the cut. The socket holes may be advanced by coring through the shale bedrock to sufficient depth below the base of cut to develop fixity. It is anticipated that the system can be stiffened by walers and/or cross bracings, where applicable.

For a soldier pile and lagging wall, the lateral pressure diagram as shown in Figure E1 in Appendix E may be used for design using the parameter values shown below.

γ_{soil}	=	bulk soil unit weight	=	20 kN/m ³
γ_{shale}	=	bulk shale unit weight	=	23 kN/m ³
γ_w	=	water unit weight	=	10 kN/m ³
K_a	=	active earth pressure coefficient		
	=	0.35 (fill)		
	=	0.33 (silty clay)		
	=	0.22 (shale)		
h_w	=	water level		
	=	0 (assuming no hydrostatic pressure build-up behind a presumably permeable wall)		
H	=	depth to base of excavation (exposed shale surface) (m)		

For rock sockets formed in the intact shale with limestone interbeds, the ultimate passive force that can be mobilized by the embedded portion of a socket is given by:

$$P_p = 6 \cdot c \cdot D \cdot L$$

where c = 300 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)

D	=	diameter of socket, m
L	=	depth of socket in rock, m

It should be pointed out that the lateral pressures acting on the retaining system from intact bedrock should be minimal. Such a wall is primarily used to prevent minor sloughing of the shale face and to protect the shale from further weathering and/or deterioration. All shoring systems should be designed by a Professional Engineer experienced in such designs.

10.4.1 Rock Dowels

Along the deeper sections of the access road, consideration should be given to using rock dowels to maintain contact between the shoring and the rock face. The dowels should be grouted in pre-drilled holes but do not need to be prestressed. It is anticipated that the rock dowels will not be required to sustain appreciable loading. Should these dowels be used, it is recommended that they be installed sub-horizontally, say at 20°, to take into consideration the horizontal bedding planes of the shale. For planning and design purposes, a dowel may be designed for a nominal grouted length of 3 m within the rock mass.

Grouted rock anchors should not be required to maintain stability of the wall. Should grouted rock anchors be considered for this or other reasons, further foundation recommendations can be provided where required.

10.5 Protection of Utilities

Protection must be provided to all existing utilities including the buried oil pipelines, hydro poles, overhead cables and any other utilities that are present within the construction zone.

11 GROUNDWATER CONTROL

11.1 General

Primary Access Cut

Based on piezometric measurements in the boreholes, the groundwater level ranges between approximate Elevations 85 m and 87 m within the plateau. This indicates that the proposed base of the road cut will be up to 5 m below the groundwater level near the crest of the plateau.

Groundwater seepage from perched water in the silty clay and at the soil-shale interface is anticipated to be small. As excavation for the cut proceeds downslope, however, seepage from the bedding planes and fractures in the shale will increase. This seepage water, in addition to surface water accumulating within the cut, must be controlled and drained. Gravity drainage measures must be in place and operational to allow the excavation work to be carried out in the dry. It is expected that the rate of seepage will decrease over time as the local groundwater table is drawn down.

11.2 Temporary Drainage

The design of temporary groundwater control systems is the responsibility of the Contractor. Some unwatering methods that are considered feasible for this site include the following:

- Within the cut, drainage ditches supplemented by pumping from filtered sumps can be used to control groundwater seepage, surface runoff and precipitation. Surface runoff from the surrounding plateau should be diverted away from the cut at all times.
- During construction, pumping from filtered sumps may be required to remove water accumulated within the cut, especially during the wet seasons and immediately after severe rainstorms and snowmelts. Filtered sumps must be designed properly so that construction drainage water containing eroded soil, weathered rock and fines do not flow into the Credit River.

11.3 Longer Term Drainage

A drainage system designed to be operational throughout the service life of the access road will be required to remove seepage water from the vertical cut faces, inclined cut slopes and the subgrade, and to prevent accumulation of precipitation and surface runoff. Positive drainage of seepage water behind the shoring walls has been addressed in the previous Section 7.2.2. Long term drainage measures that are considered feasible for this site include the following:

- Longitudinal ditches may be designed to run on one or both sides of the roadway parallel to the toe of the shoring walls or shotcreted rock face. The capacities of these facilities should be sufficient to handle the combined volume of surface water and seepage water from the cut faces.
- Gravel sheeting may be considered for use in areas where seepage occurs from exposed inclined slopes.

12 FOUNDATION DESIGN FOR TEMPORARY CONSTRUCTION ACCESS DECK

It is understood from SLI that spread footings have been proposed as temporary foundation support for launching the temporary access deck. Boreholes 10-03, 10-3A and 10-3B advanced during the present investigation indicate that the subsurface under the bridge consists of 3.5 to 3.7 m of stiff to very stiff clayey silt alluvium with a 0.6 to 0.7 m interlayer of compact shale, and a 0.6 to 1.0 m layer of firm peat (compressed) in Boreholes 10-03A and 10-03B. The soils overlie inferred shale. Water levels observed in open boreholes during drilling ranged between 1.5 and 2.1 m below existing floodplain grade. These depths appear to be consistent with the water level in the Credit River at the time of the investigation.

Information provided by SLI indicates that conservation and environmental authorities responsible for the river floodplain have expressed concerns of the potential disturbance that could occur as a result of the proposed construction activities. At the time of preparation of this report, it is not known as to whether minor sub-excavation and/or fill placement would be allowed for the footing construction.

Based on the borehole results, it is not advisable to place footings on peat. The presence of peat, though apparently compressed, could result in excessive and unpredictable total and differential settlements.

It is recommended that augered caissons (drilled shafts) be used to carry the load down to the shale bedrock estimated to be at approximate Elevation 72.5 m within the floodplain. A socket depth within the shale of at least 1 m should be allowed. For design purposes, a factored geotechnical resistance at ULS of 1,500 kPa may be assumed for 1.2 m nominal diameter end-bearing caissons on shale bedrock.

Caisson installation should be in accordance with Special Provision No. 903S01. The caisson installation equipment should be able to dislodge and remove any obstructions such as cobbles, boulders and rock slabs. Hard interbedded layers of limestone within the shale bedrock may require the use of coring or rock breaking equipment in addition to the auger equipment. Temporary steel liners should be available on site in case support to the caisson hole sidewalls are required. The contract documents must contain a statement to alert bidders of these facts. The suggested wording for an NSSP addressing this issue is included in Appendix F.

The resistance value provided above is based on the assumption that the base of each caisson is cleaned of loose material prior to placement of concrete. The caisson excavation should be dewatered (if necessary) to allow cleaning of the base and walls, and prior to placing concrete. Concrete should be placed with minimum delay after the socket is drilled and cleaned.

The bases of the caisson excavations should be inspected by a foundation/geotechnical engineer to confirm that the exposed footing subgrade conforms to the design requirements and has been adequately prepared.

13 CONSTRUCTION CONCERNS

During construction, the Contract Administrator should employ experienced foundation/geotechnical staff to observe foundation construction activities.

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

- rock mechanics specialists to be retained to periodically inspect all exposed rock faces and rock slopes to confirm face stability during construction
- full time supervision of excavation, road protection installation, drainage measures, temporary foundation construction and associated works

-
- foundation/geotechnical specialists to be retained to inspect the exposed cut face in case clarification is required as to the location of the soil-shale interface.

14 CLOSURE

Engineering analysis and preparation of this foundation design report was carried out by Dr. Sydney Pang, P.Eng.

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



Engineering Analysis and Report Preparation by:
Sydney Pang, P.Eng.,
Associate, Senior Geotechnical Engineer



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

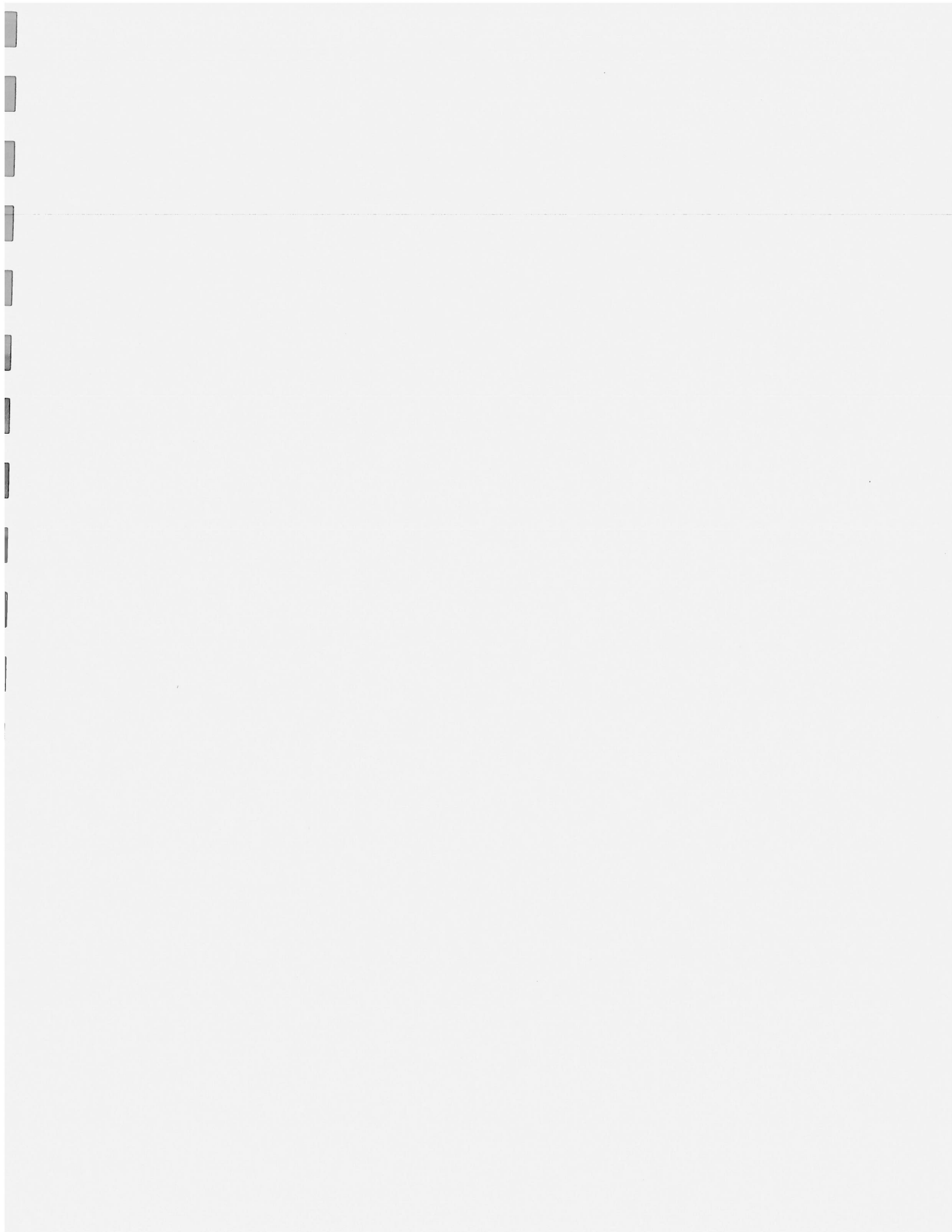


Table 1
Borehole Completion Details

Borehole	Piezometer Tip Depth / Elevation (m)	Completion Details
10-01	12.2 / 83.9	Piezometer with 4.5 m long slotted screen installed with sand filter to 6.5 m (Elevation 89.7 m), bentonite seal to 0.6 m depth, then soil cuttings to surface.
10-02	3.7 / 90.7 (upper)	Piezometer with 1.5 m long slotted screen installed with sand filter to 1.8 m (Elevation 92.6 m), bentonite seal to 0.6 m depth, then soil cuttings to surface.
	18.4 / 76.0 (lower)	Piezometer with 7.5 m long slotted screen installed with sand filter to 12.2 m (Elevation 82.2 m), bentonite seal to 0.6 m depth, then soil cuttings to surface.
10-03	None installed	Bentonite seal to surface.
10-03A	None installed	Bentonite seal to surface.
10-03B	None installed	Bentonite seal to surface.
10-04	None installed	Bentonite seal to surface.
10-05	None installed	Bentonite seal to 2.7 m depth, then soil cuttings to surface.

TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road

Run	Depth (m)	UCS (MPa)	Rock Type	Test			
Borehole		10-01		Total Rock Core			
2	3.4	0.7	Shale	Rock Type	Shale	Limestone	
2	3.6	0.6	Shale	Average (MPa)	3.3	49.6	
2	3.9	2.7	Shale	Minimum (MPa)	0.6		
2	4.4	3.4	Shale	Maximum (MPa)	10.7		
3	4.7	10.7	Shale				
3	5.1	3.0	Shale	Run #	Average (MPa)		
3	5.4	0.7	Shale	2	1.9		
3	5.7	6.8	Shale	3	4.7		
3	5.8	3.4	Shale	4	2.0		
3	6.2	3.4	Shale	5	2.8		
4	6.5	2.0	Shale	6	4.7		
4	6.9	0.7	Shale	7	14.6		
4	7.3	0.7	Shale				
4	7.6	4.8	Shale				
5	7.9	0.7	Shale				
5	8.3	2.7	Shale				
5	8.7	5.4	Shale				
5	8.9	2.9	Shale				
5	9.2	2.0	Shale				
6	9.6	9.4	Shale				
6	10.0	1.3	Shale				
6	10.2	6.8	Shale				
6	10.7	1.4	Shale				
7	11.0	4.8	Shale				
7	11.6	0.7	Shale				
7	11.7	3.4	Shale				
7	12.2	49.6	Limestone				
Borehole		10-02		Total Rock Core			
1	4.4	0.5	Shale	Rock Type	Shale	Limestone	Shale/Limestone
1	4.4	0.5	Shale	Average (MPa)	4.2	109.5	26.6
2	5.0	0.7	Shale	Minimum (MPa)	0.5	95.2	5.1
2	5.0	0.5	Shale	Maximum (MPa)	13.8	120.4	48.1
2	5.7	0.5	Shale				
2	6.0	0.5	Shale	Run #	Average (Mpa)		
2	6.0	8.9	Shale	2	2.2		
3	6.4	1.7	Shale	3	3.6		
3	6.4	3.1	Shale	4	2.8		
3	6.9	4.1	Shale	5	1.9		
3	7.4	0.7	Shale	6	5.0		
3	7.4	8.6	Shale	7	19.7		

TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road

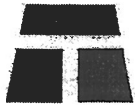
Run	Depth (m)	UCS (MPa)	Rock Type	Test
4	7.8	0.7	Shale	8 6.1
4	7.8	4.5	Shale	
4	8.7	1.7	Shale	
4	8.7	4.4	Shale	
5	9.5	0.7	Shale	9 25.2
5	9.5	5.4	Shale	
5	10.1	0.5	Shale	
5	10.1	3.2	Shale	
5	10.5	0.5	Shale	10 2.0
5	10.5	1.4	Shale	
6	11.0	0.5	Shale	
6	11.0	13.8	Shale	
6	11.8	0.7	Shale	11 6.6
6	11.8	7.6	Shale	
6	12.4	2.6	Shale	
7	12.5	1.7	Shale	
7	12.5	6.5	Shale	12 20.0
7	12.9	112.9	Limestone	
7	13.2	1.7	Shale	
7	13.2	4.1	Shale	
7	13.6	10.7	Shale	13 3.3
7	13.7	0.7	Shale	
8	14.0	0.7	Shale	
8	14.0	8.5	Shale	
8	14.8	1.7	Shale	14 7.2
8	14.8	9.5	Shale	
8	15.1	3.4	Shale	
8	15.1	13.0	Shale	
9	15.5	0.7	Shale	15 9.2
9	15.5	2.6	Shale	
9	16.0	0.7	Shale	
9	16.0	1.5	Shale	
9	16.4	120.4	Limestone	
10	17.0	0.7	Shale	
10	17.0	3.9	Shale	
10	17.6	0.7	Shale	
10	17.6	2.5	Shale	
10	18.0	0.7	Shale	
10	18.0	3.8	Shale	
11	18.6	10.1	Shale	
11	18.6	12.9	Shale	

Credit River Access Road

Run	Depth (m)	UCS (MPa)	Rock Type	Test			
11	19.1	0.7	Shale	UC			
11	19.1	7.3	Shale				
11	19.3	11.9	Shale				
11	19.6	0.7	Shale				
11	19.6	2.7	Shale				
12	19.9	3.4	Shale				
12	19.9	11.5	Shale				
12	20.3	1.7	Shale				
12	20.3	7.2	Shale				
12	21.0	95.2	Limestone				
12	21.1	1.7	Shale				
12	21.1	6.3	Shale				
12	21.3	5.1	Shale/Lime				
12	21.3	48.1	Shale/Lime				
13	21.9	0.7	Shale				
13	21.9	5.9	Shale				
14	22.4	5.0	Shale				
14	22.4	9.4	Shale				
15	23.0	5.1	Shale				
15	23.0	8.7	Shale				
15	23.9	6.7	Shale				
15	23.9	12.4	Shale				
15	24.3	13.4	Shale				
Borehole		10-05		Total Rock Core			
1	3.2	1.4	Shale	Rock Type	Shale	Limestone	Shale/Limestone
2	3.5	0.9	Shale	Average (MPa)	3.0	73.1	3.7
2	4.0	0.8	Shale	Minimum (MPa)	0.7	27.9	2.0
2	4.2	1.1	Shale	Maximum (MPa)	10.8	200.5	5.4
2	4.8	4.1	Shale				
3	5.0	0.7	Shale	Run #	Average (Mpa)		
3	5.3	8.4	Shale	1	1.4		
3	5.6	1.5	Shale	2	1.7		
3	6.0	1.3	Shale	3	2.6		
3	6.3	1.3	Shale	4	4.6		
4	6.4	1.3	Shale	5	9.6		
4	6.7	8.5	Shale	6	3.8		
4	7.1	2.7	Shale	7	7.7		
4	7.4	10.8	Shale	8	4.7		
4	7.5	2.0	Shale	9	61.2		
4	7.8	2.0	Shale/Lime				
5	8.1	1.4	Shale				

**TABLE 2 - Point Load and Unconfined Compressive Strength Test Results
Credit River Access Road**

Run	Depth (m)	UCS (MPa)	Rock Type	Test
5	8.5	1.4	Shale	
5	8.9	0.7	Shale	
5	9.0	35.0	Limestone	
6	9.4	3.4	Shale	
6	9.8	0.7	Shale	
6	10.0	2.9	Shale	
6	10.3	5.4	Shale	
6	10.7	6.8	Shale	
7	11.0	2.0	Shale	
7	11.3	27.9	Limestone	
7	11.6	1.3	Shale	
7	11.9	0.7	Shale	
7	12.2	6.8	Shale	
8	12.5	4.7	Shale	
8	13.0	2.7	Shale	
8	13.5	5.9	Shale	
8	13.7	5.4	Shale/Lime	
9	14.1	37.7	Limestone	
9	14.4	200.5	Limestone	
9	14.6	64.3	Limestone	
9	15.0	1.3	Shale	
9	15.2	2.0	Shale	



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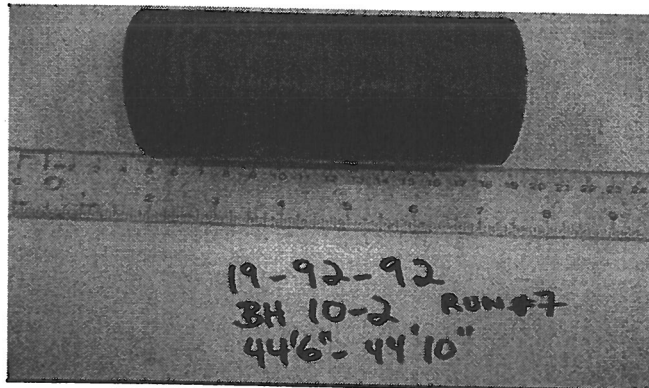
UNCONFINED COMPRESSION TEST REPORT

ASTM D 2938 - 95 (with Stress vs. Strain Measurements)

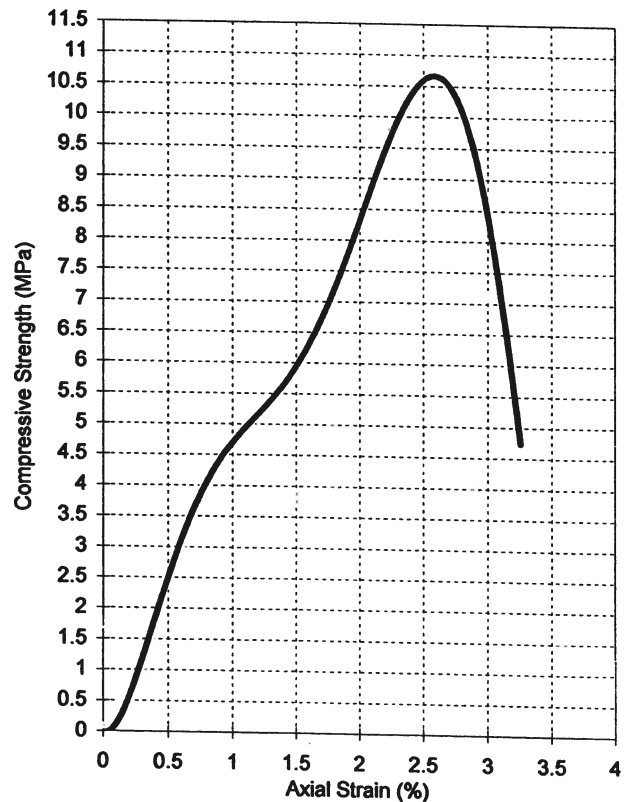
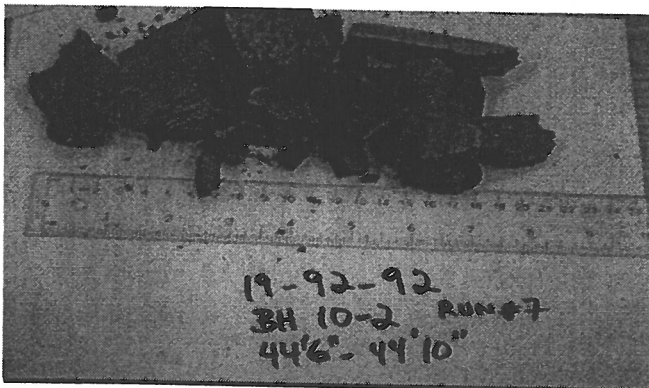
CLIENT:	SNC Lavalin	FILE NUMBER:	19-92-92
PROJECT NAME:	QEW	REPORT DATE:	12-Oct-10
BOREHOLE No.:	BH10-2	TEST DATE:	12-Oct-10
SAMPLE No.:	HQ RUN#7		
SAMPLE DEPTH:	44'6"-44'10"		
DESCRIPTION:	Shale, Fresh, Thinly Bedded, Dark Grey, Weak		

Avg. Height (cm):	11.84	Weight (g):	957.0
Avg. Diameter (cm):	6.25	Wet Density (kg/m ³):	2,635
H. to Dia. Ratio**:	1.89 : 1	Dry Density (kg/m ³):	2,544
Cross Sectional Area (cm ²):	30.68	Moisture Content* (%):	3.6
Sample Volume (cm ³):	363.25	Temperature during Test (° C)	20

ORIGINAL SPECIMEN



FRACTURED SPECIMEN



AVG. RATE OF STRAIN TO FAILURE:	1.3 % / min
MAXIMUM COMPRESSIVE LOAD:	32.8 kN
UNCONFINED COMPRESSIVE STRENGTH:	10.7 MPa @ 2.6% strain

Note: * The moisture content was obtained before the test.
** Dimension of Specimen does not conform to ASTM Standard.

TEST DONE BY: EA
REVIEWED BY: WM

UCS Test Report BH10-2 RUN#7.xls



UNCONFINED COMPRESSION TEST REPORT

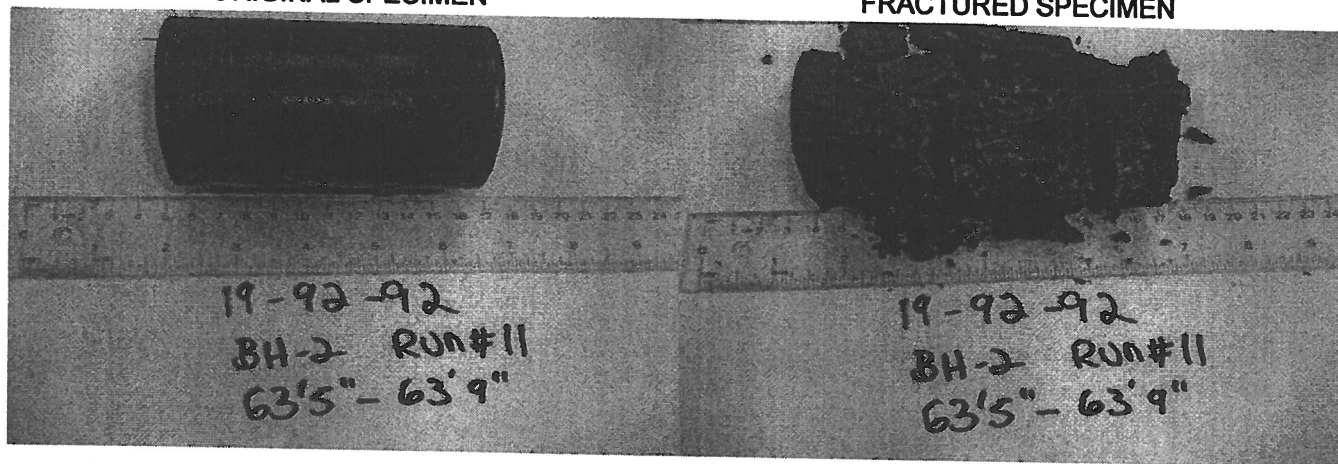
ASTM D 2938 - 95

CLIENT:	SNC Lavalin	FILE NUMBER:	19-92-92
PROJECT NAME:	QEW	REPORT DATE:	25-Oct-10
BOREHOLE No.:	BH-2	TEST DATE:	21-Oct-10
SAMPLE No.:	HQ RUN 11		
SAMPLE DEPTH:	63'5"-63'9"		
DESCRIPTION:	Shale, Fresh, Dark Grey		

Avg. Height (cm):	10.55	Weight (g):	834.0
Avg. Diameter (cm):	6.21	Wet Density (kg/m ³):	2,610
H. to Dia. Ratio**:	1.7:1	Dry Density (kg/m ³):	2,506
Cross Sectional Area (cm ²):	30.29	Moisture Content* (%):	4.1
Sample Volume (cm ³):	319.54		

ORIGINAL SPECIMEN

FRACTURED SPECIMEN



AVG. RATE OF STRAIN TO FAILURE:	1.4% / min
MAXIMUM COMPRESSIVE LOAD:	35.9 kN
UNCONFINED COMPRESSIVE STRENGTH:	11.9 MPa

Note: * The moisture content was obtained before the test.
** Dimensions of Specimen do not conform to ASTM D 4543-04.

TEST DONE BY: BT
REVIEWED BY: WM

BH10-2 RUN11 UCS.xls



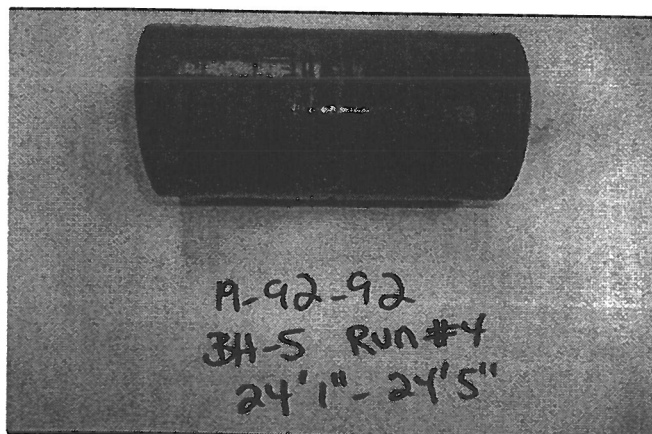
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UNCONFINED COMPRESSION TEST REPORT

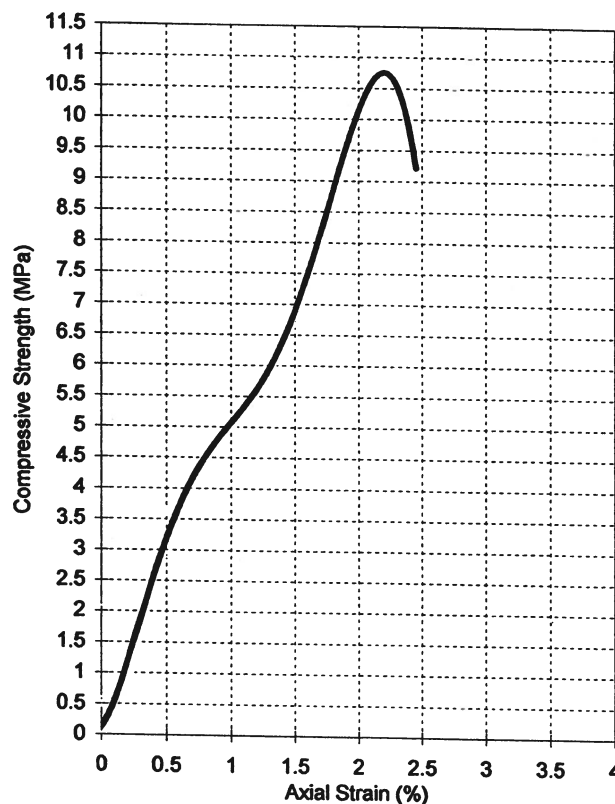
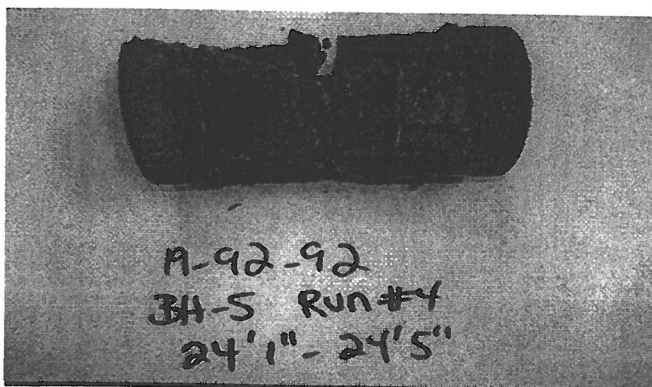
ASTM D 2938 - 95 (with Stress vs. Strain Measurements)

CLIENT:	SNC Lavalin	FILE NUMBER:	19-92-92
PROJECT NAME:	QEW - Credit River	REPORT DATE:	25-Oct-10
BOREHOLE No.:	BH10-5	TEST DATE:	20-Oct-10
SAMPLE No.:	HQ RUN#4		
SAMPLE DEPTH:	24'1"-24'5"		
DESCRIPTION:	Shale, Fresh, Thinly Bedded, Dark Grey, Weak		

Avg. Height (cm):	11.10	Weight (g):	914.9
Avg. Diameter (cm):	6.21	Wet Density (kg/m ³):	2,721
H. to Dia. Ratio**:	1.79 : 1	Dry Density (kg/m ³):	2,607
Cross Sectional Area (cm ²):	30.29	Moisture Content* (%):	4.4
Sample Volume (cm ³):	336.20	Temperature during Test (° C)	20



FRACTURED SPECIMEN



AVG. RATE OF STRAIN TO FAILURE:	1.4 % / min
MAXIMUM COMPRESSIVE LOAD:	32.7 kN
UNCONFINED COMPRESSIVE STRENGTH:	10.8 MPa @ 2.2% strain

Note: * The moisture content was obtained before the test.
** Dimension of Specimen does not conform to ASTM Standard.

TEST DONE BY: EA
REVIEWED BY: WM

UCS Test Report BH10-5 RUN#4.xls

Appendix A

Record of Borehole Sheets

19-92-92



SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT 'N' VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$



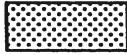


 Water Level
 C_{pen} Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. $(W_L < 30\%)$.
		CI	Inorganic clays of medium plasticity, silty clays. $(30\% < W_L < 50\%)$.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

<u>ROCK WEATHERING CLASSIFICATION</u>		<u>SYMBOLS</u>	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>		
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa) (psi)	Field Estimation of Hardness*
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250 Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m			
Medium bedded	0.2 to 0.6m	Very Strong	100-250 15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m			
Very thinly bedded	20 to 60mm	Strong	50-100 7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm			
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0 3,500 to 7,500	Breaks under single blow of geological hammer.
<u>TERMS</u>		Weak	5.0 to 25.0 750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0 150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0 35 to 150	Indented by thumbnail
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.			
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen			
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.			

RECORD OF BOREHOLE No 10-01

1 OF 1

METRIC

W.P. 2186-07-00 LOCATION QEW - Credit River Access Road (N 4 823 900.11 E 295 760.01) ORIGINATED BY SLL
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/HQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2010.10.04 - 2010.10.04 CHECKED BY SKP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100	W _p	W	W _L		
96.2																
0.0	TOPSOIL, with roots and rootlets: (150mm)		1	SS	12	96										GR SA SI CL
0.2																
95.6	Silty CLAY, with sand, trace gravel, trace shale fragments		2	SS	47											10 46 28 16
0.6	Stiff Brown Moist															
	SHALE, weathered Reddish Brown to Grey Moist		3	SS	56	95										0 16 59 25
			4	SS	86	94										
93.5																
2.7	END OF SPT SAMPLING AT 2.7m. START CORING AT 2.7m. FOR ROCK DETAILS PLEASE REFER TO 10-01R. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Oct.04/10 4.5 91.7 Oct.05/10 9.2 87.0 Oct.12/10 9.3 86.9 Dec.17/10 10.0 86.2															

RECORD OF BOREHOLE No 10-02

1 OF 1

METRIC

W.P. 2186-07-00 LOCATION QEW - Credit River Access Road (N 4 823 966.28 E 295 797.95) ORIGINATED BY SLL
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/HQ Rock Coring COMPILED BY AN
 DATUM Geodetic DATE 2010.09.30 - 2010.09.30 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
94.4																	
0.0	TOPSOIL: (150mm)																
0.2	Silty CLAY, with sand, trace gravel, roots and rootlets Very Stiff to Hard Brown Moist		1	SS	25		94							o			3 59 22 16
			2	SS	29									o			
	With shale fragments		3	SS	39		93							o			3 16 46 35
			4	SS	34		92							o			1 26 47 26
91.2			5	SS	76/ 0.275		91							o			
3.2	SHALE, weathered Grey Moist		6	SS	50/ 0.100									o			
89.7			7	SS	50/ 0.075		90							o			
4.6	END OF SPT SAMPLING AT 4.6m. START CORING AT 4.6m. FOR ROCK DETAILS PLEASE REFER TO 10-02R. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. SHALLOW PIEZOMETER WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Oct.04/10 Dry - Oct.05/10 Dry - Oct.12/10 Dry - Dec.17/10 Dry - DEEP PIEZOMETER WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Oct.04/10 8.5 85.9 Oct.05/10 8.5 85.9 Oct.12/10 8.7 85.7 Dec.17/10 9.3 85.1																

RECORD OF BOREHOLE 10-02R

PROJECT : QEW - Credit River Access Road
 LOCATION : Oakville
 STARTED : September 30, 2010
 COMPLETED : October 1, 2010

Project No. 2186-07-00

INCLINATION: Vertical AZIMUTH: Vertical

SHEET 1 OF 3
 DATUM Geodetic

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	FIELD/LABORATORY TESTING RESULTS																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														
				ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	COLOUR % RETURN	FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX PER 3 m	DIP wrt Core Axis	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	Unconfined Compressive Strength (MPa)	FIELD/LABORATORY TESTING RESULTS ● Point Load Test Diametral ▲ Point Load Test Axial ■ Laboratory UCS Test																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
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Bentonite

85.25

Sand Filter

83.57

Limestone

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
 WATER LEVEL (date)

▽ DEEP/DUAL INSTALLATION
 WATER LEVEL (date)

LOGGED :
 CHECKED :



RECORD OF BOREHOLE 10-02R

PROJECT : QEW - Credit River Access Road
 LOCATION : Oakville
 STARTED : September 30, 2010
 COMPLETED : October 1, 2010

Project No. 2186-07-00

INCLINATION: Vertical AZIMUTH: Vertical

SHEET 2 OF 3
 DATUM Geodetic

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	PENETRATION RATE (mm/min)	COLOUR	FLUSH % RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER 3 m	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec	Unconfined Compressive Strength (MPa)	FIELD/LABORATORY TESTING RESULTS ● Point Load Test Diametral ▲ Point Load Test Axial ■ Laboratory UCS Test
				DEPTH						TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION				
				(m)						80 80								

Slotted Screen

75.95

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
 WATER LEVEL (date)

▽ DEEP/DUAL INSTALLATION
 WATER LEVEL (date)

LOGGED :
 CHECKED :



RECORD OF BOREHOLE 10-02R

PROJECT : QEW - Credit River Access Road
LOCATION : Oakville
STARTED : September 30, 2010
COMPLETED : October 1, 2010

Project No. 2186-07-00

SHEET 3 OF 3
DATUM Geodetic

INCLINATION: Vertical AZIMUTH: Vertical

[illegible]

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
WATER LEVEL (date)

DEEP/DUAL INSTALLATION
WATER LEVEL (date)

LOGGED :
CHECKED :



RECORD OF BOREHOLE No 10-03

1 OF 1

METRIC

W.P. 2186-07-00 LOCATION QEW - Credit River Access Road (N 4 823 973.68 E 295 862.56) ORIGINATED BY GA
HWY QEW BOREHOLE TYPE Tri-pod COMPILED BY AN
DATUM Geodetic DATE 2010.11.04 - 2010.11.04 CHECKED BY SKP

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100		
76.5													
0.8	TOPSOIL: (50mm)		1	SS	11								
	Clayey SILT, some sand, trace gravel, occasional shale fragments, occasional wood fibres												
75.7	Stiff		2	SS	50	76							
0.8	Brown/Grey Dry				0.075								
	END OF BOREHOLE AT 0.8m UPON SPLIT SPOON SAMPLER REFUSAL. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.												

RECORD OF BOREHOLE No 10-03A

1 OF 1

METRIC

W.P. 2186-07-00 LOCATION QEW - Credit River Access Road (N 4 823 979.03 E 295 865.10) ORIGINATED BY GA
 HWY QEW BOREHOLE TYPE Tri-pod COMPILED BY AN
 DATUM Geodetic DATE 2010.11.04 - 2010.11.04 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE										
76.2							20	40	60	80	100	20	40	60		GR SA SI CL		
0.0	Clayey SILT, with sand, trace gravel, trace shale fragments Very Stiff Brown/Grey		1	SS	17	V	76									0 44 36 20		
75.6	Dry																	
0.6	Frequent shale fragments, trace silt and sand		2	SS	24													
75.0								75										3 33 47 17
1.2	Clayey SILT, with sand, trace gravel Stiff to Firm Grey Wet		3	SS	14													
			4	SS	8		74											
73.5			5	SS	10													1 38 46 15
2.7	PEAT, amorphous, occasional rootlets Firm Brown Moist	6	SS	4	73													
72.6																		
3.7	SHALE, weathered Grey Moist	7	SS	35	72													
72.0																		
4.3	END OF BOREHOLE AT 4.3m UPON SPLIT SPOON SAMPLER REFUSAL. BOREHOLE OPEN TO 4.3m AND WATER LEVEL AT 1.5m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.																	

ONTMT-4S 9292.GPJ 17/7/11

METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	20 40 60 80 100	20 40 60						
76.3 0.0	Clayey SILT, with sand, trace gravel Very Stiff Brown Dry		1	SS	21		76							0 36 46 18		
75.7 0.6	Frequent shale fragments		2	SS	28		75									
75.0 1.3	Clayey SILT, with sand Stiff Grey Moist		3	SS	14		74									
73.9 2.4	PEAT, amorphous, occasional rootlets Firm to Stiff Brown Moist		4	SS	11		73									
73.2 3.0	Clayey SILT, some clay, mixed with organics Stiff	5	SS	7									0 25 57 18			
72.6 3.7	SHALE, weathered Grey Wet	6	SS	10												
72.3 4.0	END OF BOREHOLE AT 4.0m UPON SPLIT SPOON SAMPLER REFUSAL. BOREHOLE OPEN TO 4.0m AND WATER LEVEL AT 2.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.	7	SS	50/ 0.150												

RECORD OF BOREHOLE No 10-04

1 OF 1

METRIC

W.P. 2186-07-00 LOCATION QEW - Credit River Access Road (N 4 824 005.69 E 295 823.89) ORIGINATED BY GA
HWY QEW BOREHOLE TYPE Tri-pod COMPILED BY AN
DATUM Geodetic DATE 2010.11.05 - 2010.11.05 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
76.3	TOPSOIL: (50mm)																
76.8	Clayey SILT, some sand, trace gravel Stiff to Very Stiff Brown to Grey Dry to Wet		1	SS	17		76										2 30 46 22
			2	SS	19												
			3	SS	12		75										
			4	SS	8												
			5	SS	10		74										0 23 59 18
72.8			6	SS	26		73										
72.9	SHALE, weathered		7	SS	50												
3.7	END OF BOREHOLE AT 3.7m. BOREHOLE OPEN TO 3.7m AND WATER LEVEL AT 0.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO SURFACE.				0.025												

RECORD OF BOREHOLE No 10-05

1 OF 1

METRIC

W.P. 2186-07-00 LOCATION QEW - Credit River Access Road (N 4 823 937.77 E 295 774.94) ORIGINATED BY SLL
 HWY QEW BOREHOLE TYPE Hollow Stem Augers/HQ Rock Coring COMPILED BY AN
 DATUM Geodetic DATE 2010.10.04 - 2010.10.04 CHECKED BY SKP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
95.2								20	40	60	80	100		
0.0	TOPSOIL: (150mm)													
0.2	Silty CLAY, with sand pockets, trace rootlets Stiff Brown Moist		1	SS	12		95							
			2	SS	13									0 5 37 58
93.9							94							
1.4	SHALE, weathered Reddish Brown to Grey Moist		3	SS	53									0 20 51 29
							93							
92.6			4	SS	84/ 0.225									
2.7	END OF SPT SAMPLING AT 2.7m. START CORING AT 2.7m. FOR ROCK DETAILS PLEASE REFER TO 10-05R.													

RECORD OF BOREHOLE 10-05R

PROJECT : QEW - Credit River Access Road
 LOCATION : Oakville
 STARTED : October 5, 2010
 COMPLETED : October 5, 2010

Project No. 2186-07-00

INCLINATION: Vertical AZIMUTH: Vertical

SHEET 1 OF 2
 DATUM Geodetic

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	COLOUR % RETURN	FR-FRACTURE CL-CLEAVAGE SH-SHEAR VN-VEIN	F-FAULT J-JOINT P-POLISHED S-SLICKENSIDED	SM-SMOOTH R-ROUGH ST-STEPPED PL-PLANAR	FL-FLEXURED UE-UNEVEN W-WAVY C-CURVED	HYDRAULIC CONDUCTIVITY k, cm/sec	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	DIP wrt Core Axis	FRACT. INDEX PER 3 m	R.O.D. %	RECOVERY TOTAL CORE %	SOLID CORE %	FIELD/LABORATORY TESTING RESULTS ● Point Load Test Diametral ▲ Point Load Test Axial ■ Laboratory UCS Test
				92.60 2.70															
	RUN	SHALE, slightly to moderately weathered, reddish brown to grey, fine grained, thinly bedded with some grey LIMESTONE interbeds and occasional clay seams, moist			1	0.22	100												
		Rubble zone (50mm thick) at 2.8m																	
		Vertical joint from 2.8m to 2.9m																	
4	RUN	Limestone interbed (80mm thick) at 2.8m			2	0.22	100												
		Rubble zone from 3.3m to 3.4m and 3.8m to 3.9m																	
		Limestone interbed (100mm thick) at 3.7m																	
6	RUN				3	0.22	100												
	RUN				4	0.19	100												
		Limestone interbed (10mm thick) at 7.7m																	
8	RUN	Limestone interbeds between 50mm and 75mm thick at 9.0m and 9.2m			5	0.21	100												
		Limestone interbeds between 10mm and 50mm thick at 9.6m, 10.1m, 10.3m, 10.9m																	
10	RUN	Limestone interbeds between 15mm and 50mm thick at 11.3m, 11.7m and 12.0m			6	0.16	100												
12	RUN				7	0.13	100												

GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
 WATER LEVEL (date)

▼ DEEP/DUAL INSTALLATION
 WATER LEVEL (date)

LOGGED :
 CHECKED :



RECORD OF BOREHOLE 10-05R

PROJECT : QEW - Credit River Access Road
 LOCATION : Oakville
 STARTED : October 5, 2010
 COMPLETED : October 5, 2010

Project No. 2186-07-00

INCLINATION: Vertical AZIMUTH: Vertical

SHEET 2 OF 2
 DATUM Geodetic

DEPTH SCALE (metres)	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. LOG													FIELD/LABORATORY TESTING RESULTS																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
				ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH % RETURN	RECOVERY		R.O.D. %	FRACT. INDEX PER .3 m	DIP wrt. Core Axis	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY k, cm/sec	Unconfined Strength (MPa)	Compressive Strength (MPa)	Laboratory UCS Test																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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GROUNDWATER ELEVATIONS

▽ SHALLOW/SINGLE INSTALLATION
 WATER LEVEL (date)

▽ DEEP/DUAL INSTALLATION
 WATER LEVEL (date)

LOGGED :
 CHECKED :



Appendix B

Laboratory Test Results

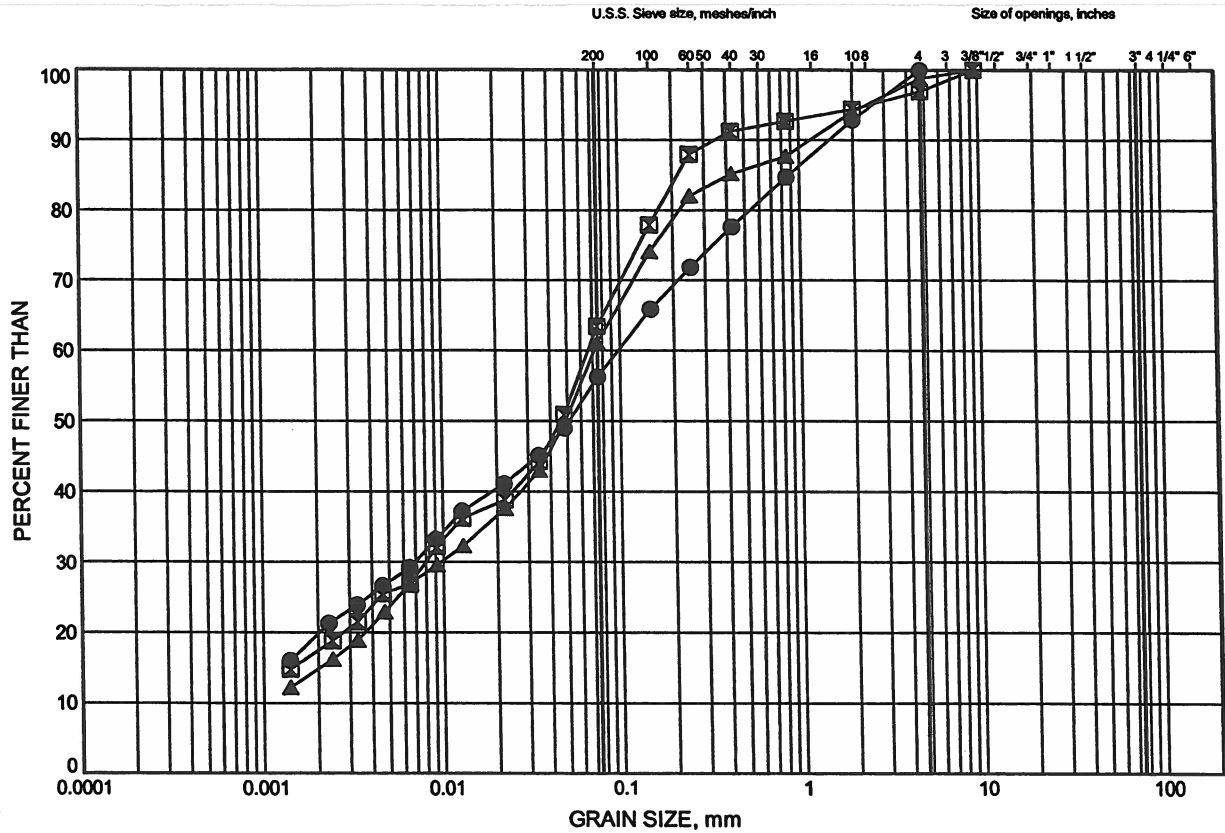
19-92-92



QEW - Credit River Access Road GRAIN SIZE DISTRIBUTION

FIGURE B1

CLAYEY SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-03A	0.30	75.92
◻	10-03A	1.52	74.70
▲	10-03A	2.74	73.48

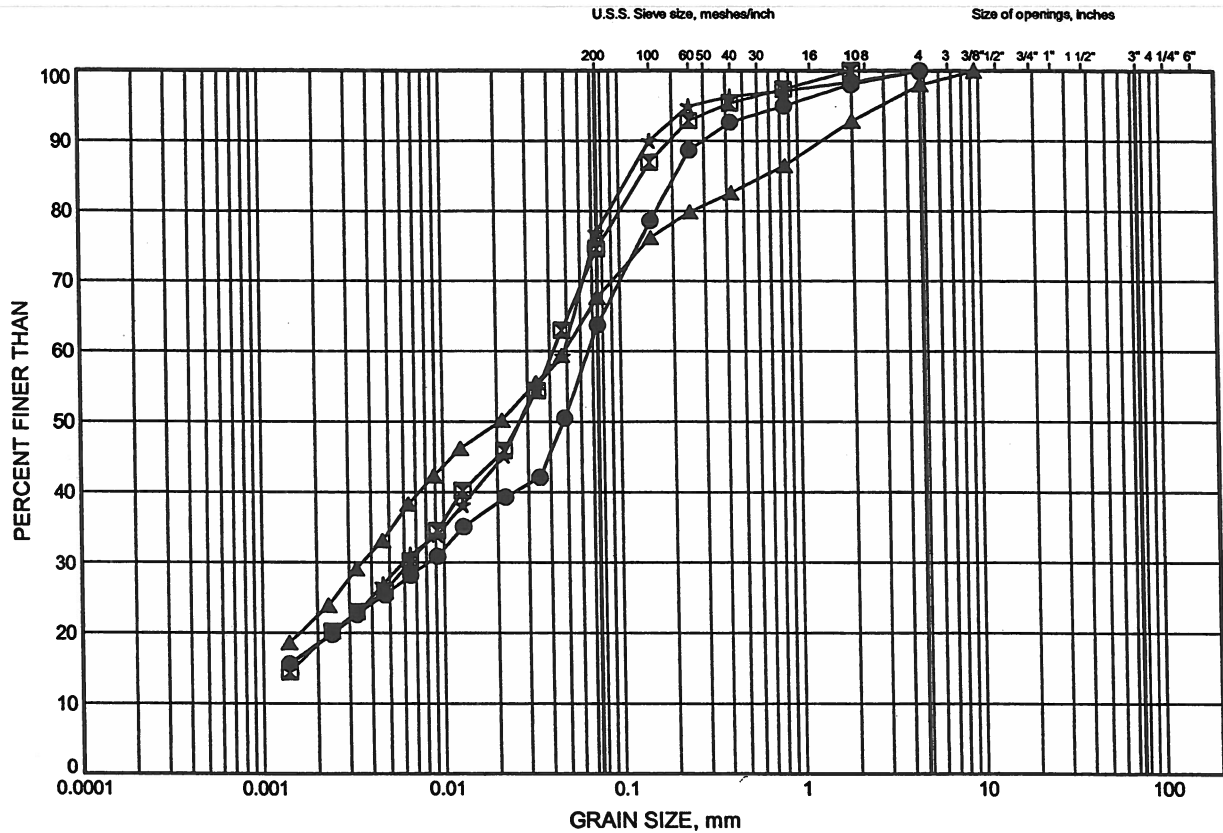


W.P.# .2186-07-00.....
Prepared By .AN.....
Checked By .SKP.....

QEW - Credit River Access Road
GRAIN SIZE DISTRIBUTION

FIGURE B2

CLAYEY SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND		GRAVEL		SIZE	

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-03B	1.52	74.77
⊠	10-03B	3.35	72.94
▲	10-04	0.91	75.43
★	10-04	2.74	73.60

GRAIN SIZE DISTRIBUTION - THURBER 9292.GPJ 1/7/11

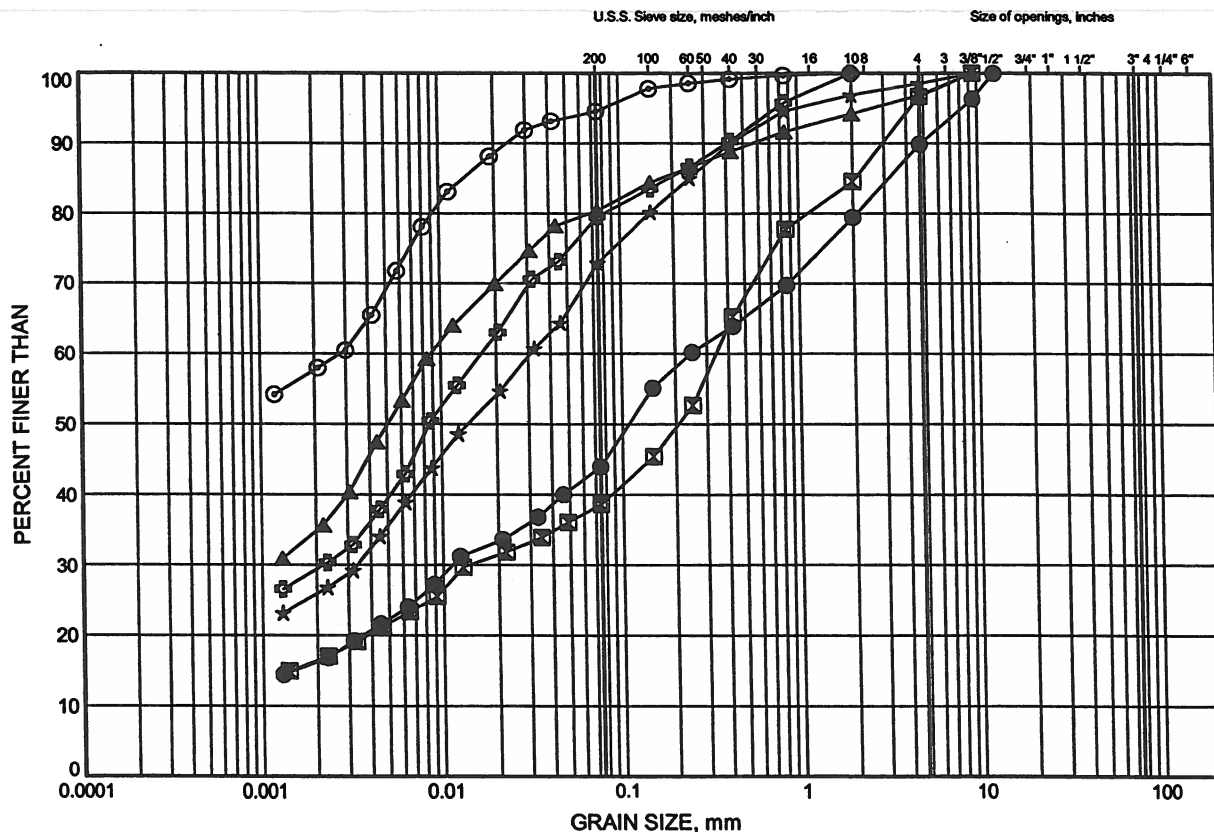
W.P.# 2186-07-00
Prepared By AN
Checked By SKP



QEW - Credit River Access Road
GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND			GRAVEL		SIZE

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-01	0.30	95.88
⊠	10-02	0.30	94.09
▲	10-02	1.83	92.56
★	10-02	2.59	91.80
⊙	10-05	0.91	94.33
⊗	10-05	1.83	93.41

GRAIN SIZE DISTRIBUTION - THURBER 9292.GPJ 1/7/11

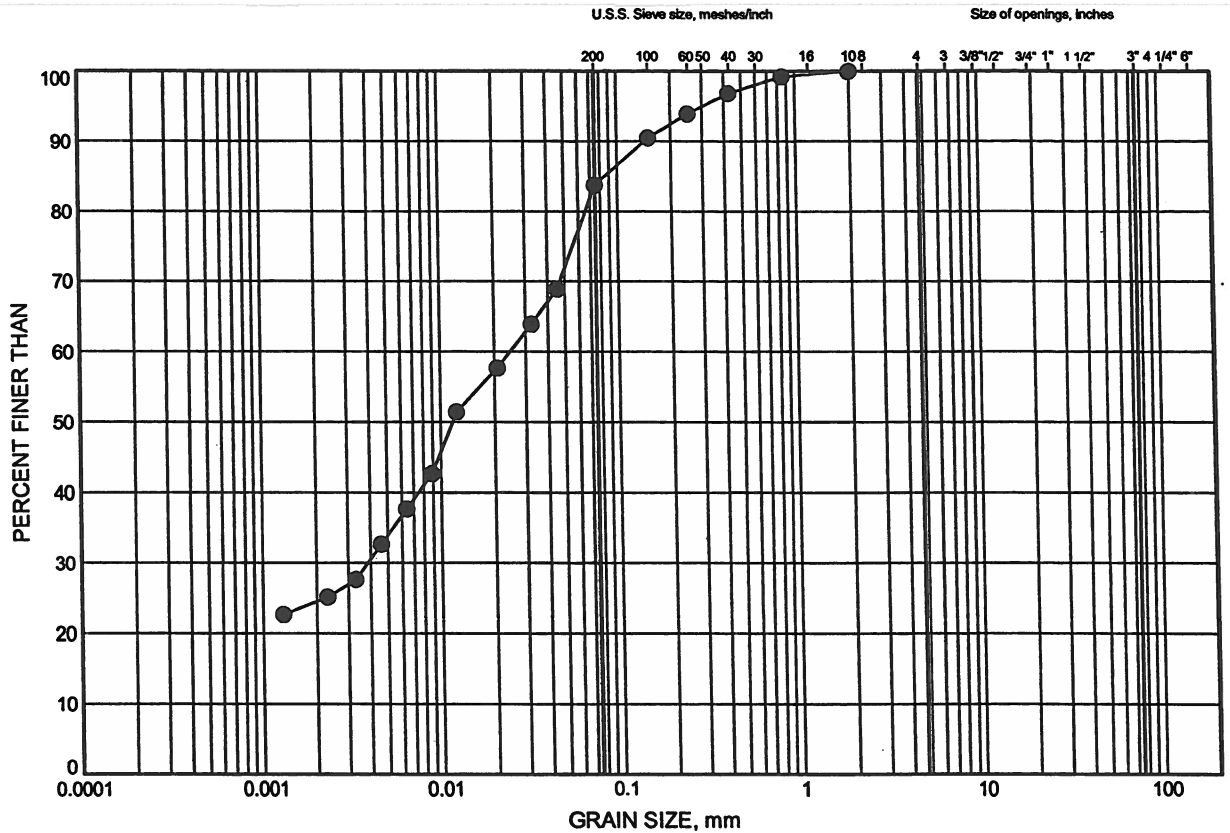
W.P.# 2186-07-00.....
Prepared By .AN.....
Checked By .SKP.....



QEW - Credit River Access Road GRAIN SIZE DISTRIBUTION

FIGURE B4

WEATHERED SHALE



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-01	1.83	94.35

GRAIN SIZE DISTRIBUTION - THURBER 9292.GPJ 17/11

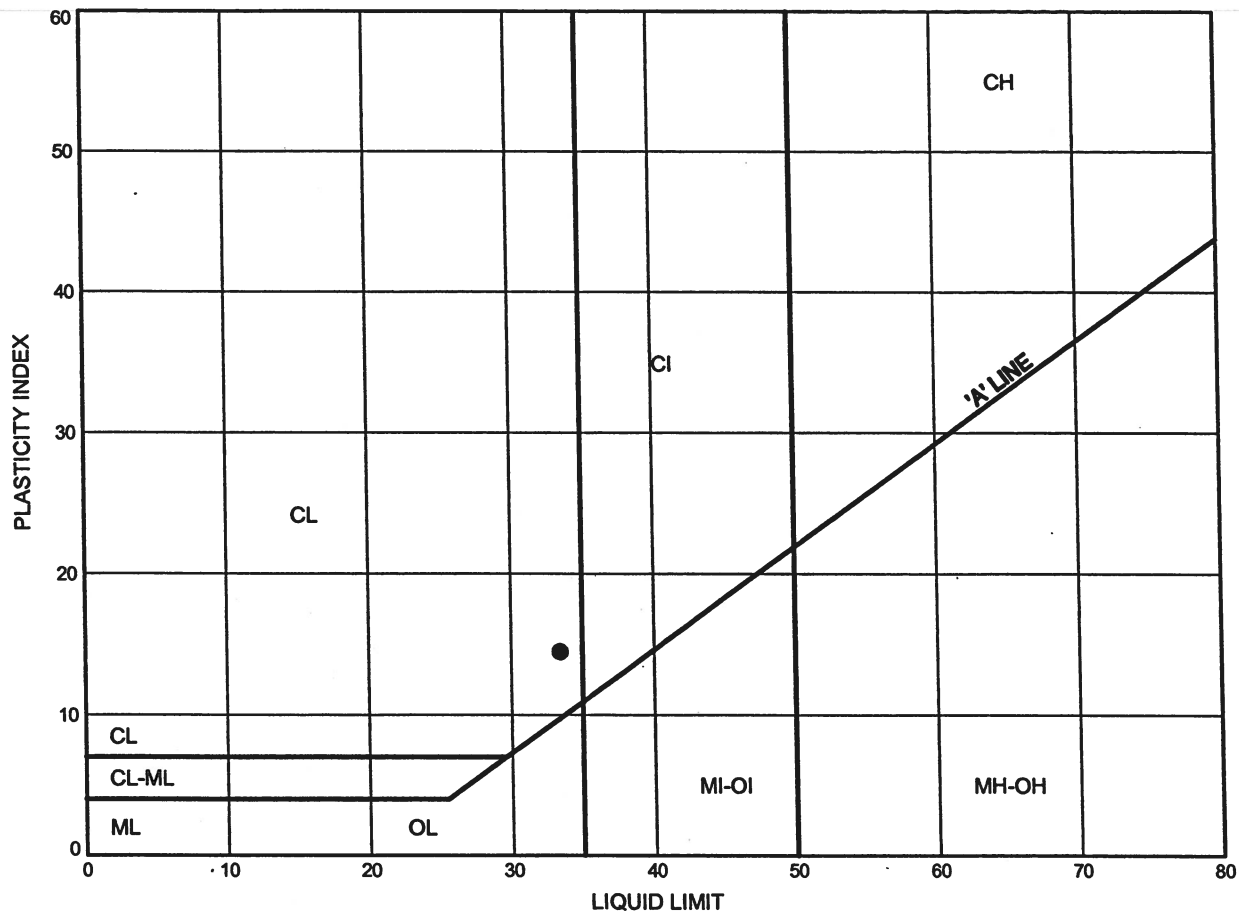
W.P.# 2186-07-00
Prepared By AN
Checked By SKP



QEW - Credit River Access Road
ATTERBERG LIMITS TEST RESULTS

FIGURE B5

CLAYEY SILT



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	10-03A	0.30	75.92

Date January 2011
 Project 2186-07-00

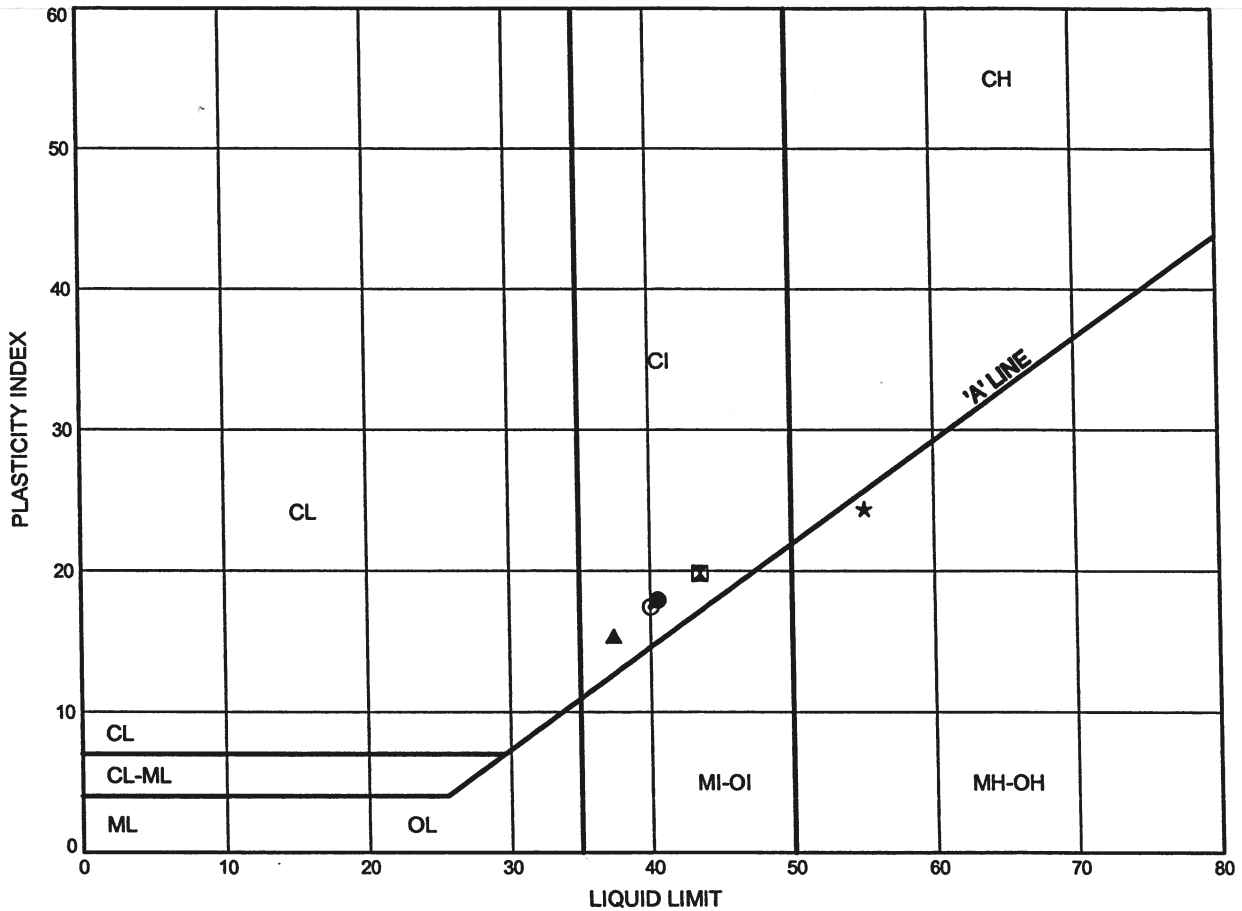


Prep'd AN
 Chkd. SKP

QEW - Credit River Access Road
ATTERBERG LIMITS TEST RESULTS

FIGURE B6

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	10-01	1.83	94.35
⊠	10-02	1.83	92.56
▲	10-02	2.59	91.80
★	10-05	0.91	94.33
⊙	10-05	1.83	93.41

Date January 2011
 Project 2186-07-00



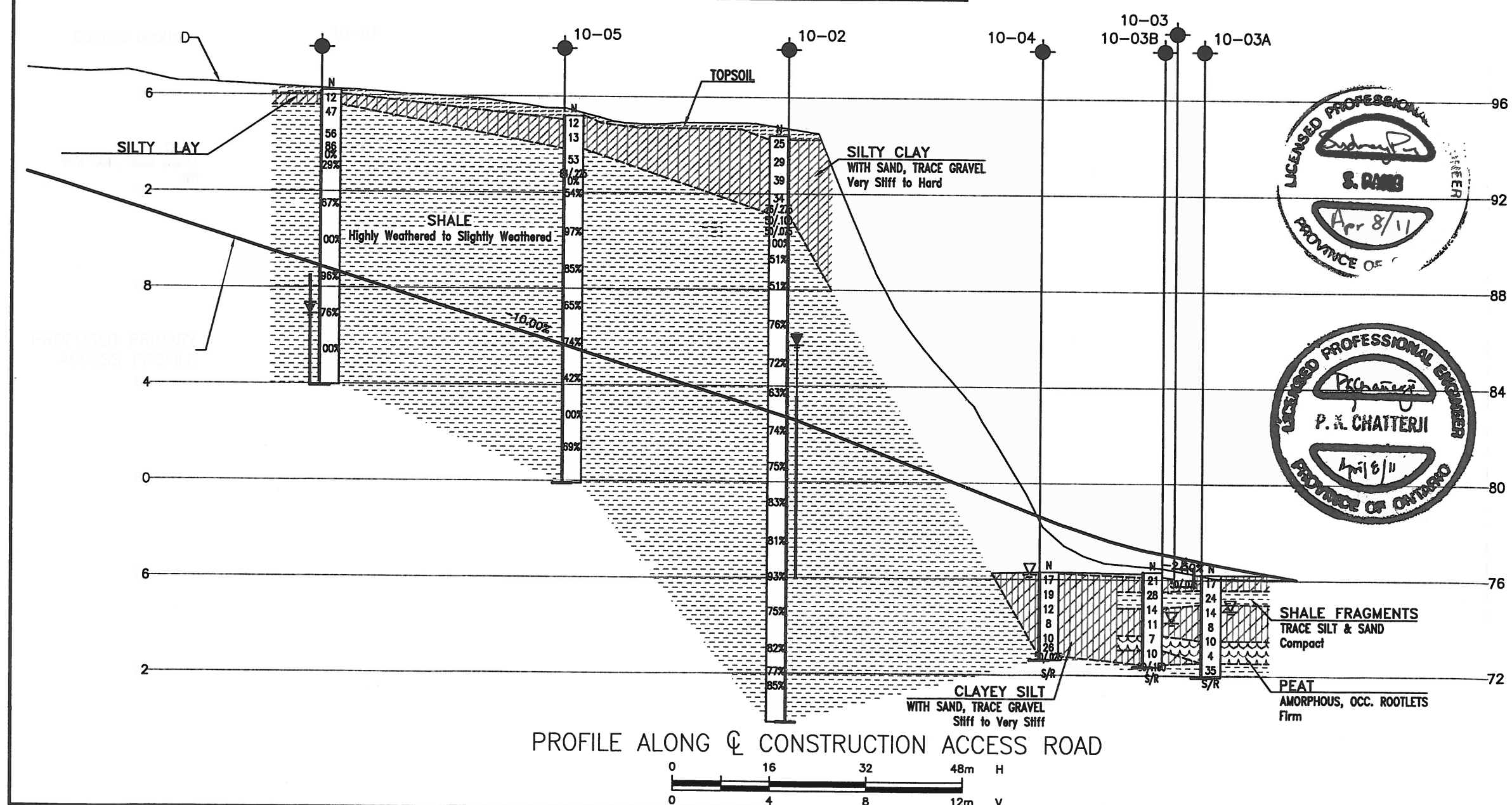
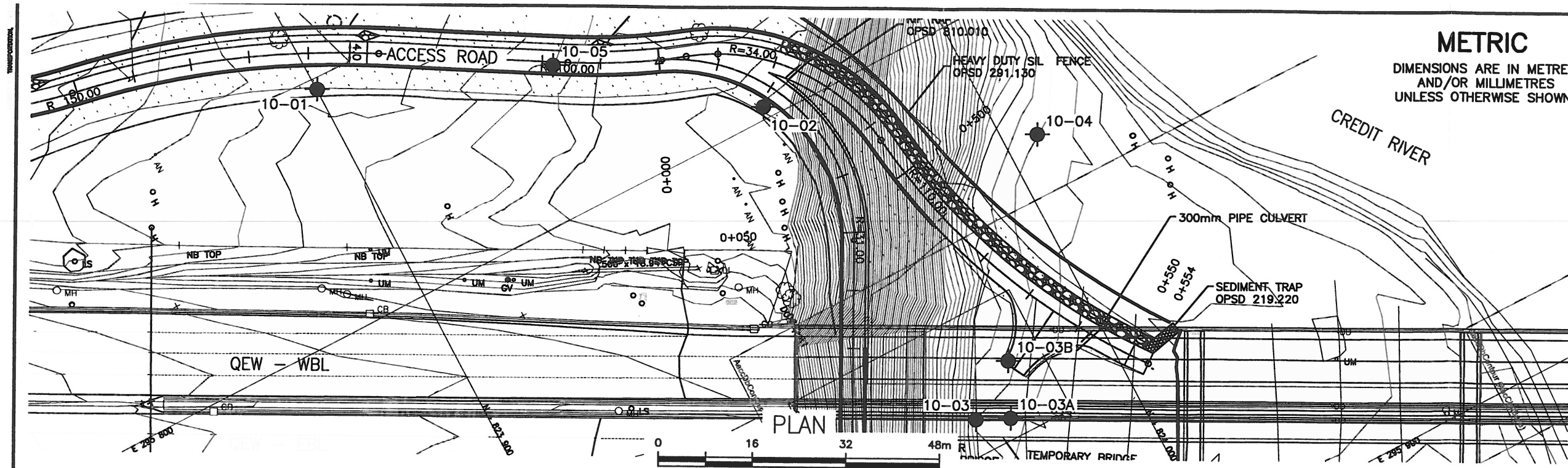
Prep'd AN
 Chkd. SKP



Appendix C
Drawings

19-92-92





QEW
CONT No
WP No 2186-07-00

SHEET

SNC-LAVALIN

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

KEYPLAN

LEGEND

- Borehole
- Borehole and Cone
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- Water Level in Open Borehole
- Water Level in Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal
- S/R Split Spoon Sampler Refusal

NO	ELEVATION	NORTHING	EASTING
10-01	96.2	4 823 900.1	295 760.0
10-02	94.4	4 823 866.3	295 798.0
10-03	76.5	4 823 973.7	295 862.6
10-03A	76.2	4 823 979.0	295 865.1
10-03B	76.3	4 823 983.2	295 856.2
10-04	76.3	4 824 005.7	295 823.9
10-05	95.2	4 823 937.8	295 774.9

NOTES

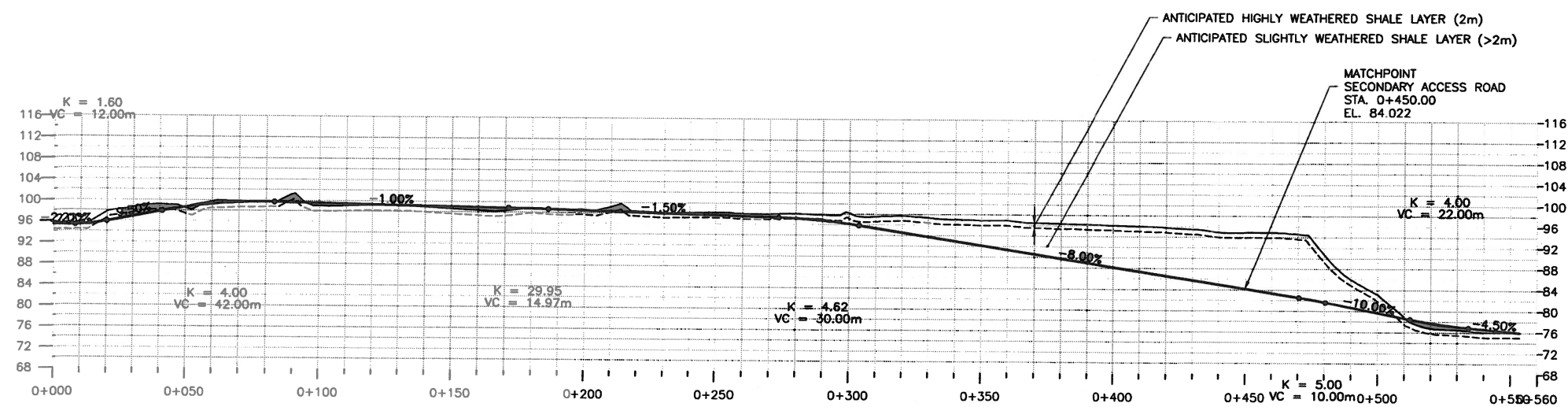
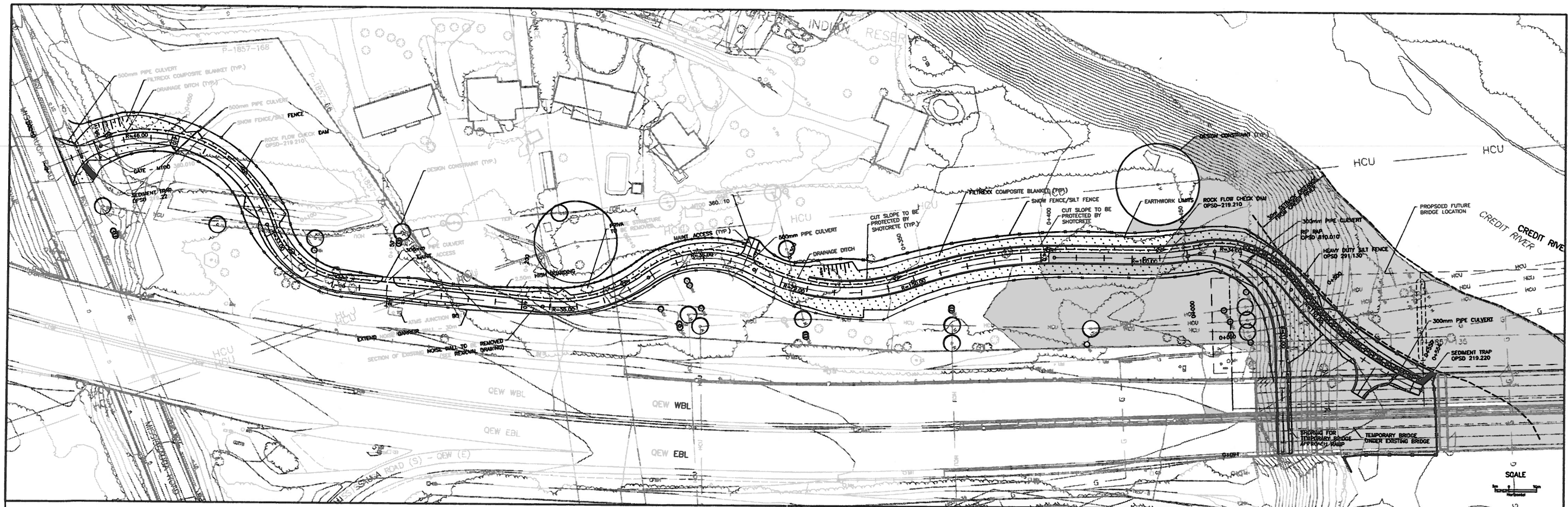
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 30M12-324

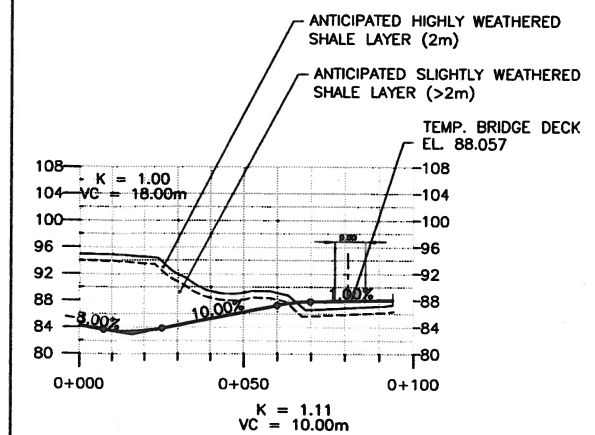
REVISIONS

DATE	BY	DESCRIPTION
DESIGN	SKP	CHK SKP
DRAWN	MFA	CHK PKC

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PRIMARY ACCESS PROFILE



SECONDARY ACCESS PROFILE

QEW/CREDIT RIVER WP 2186-07-00 ACCESS ROAD RECOMMENDED DESIGN

DATE: DECEMBER 15, 2010




PH-D-302 88-10

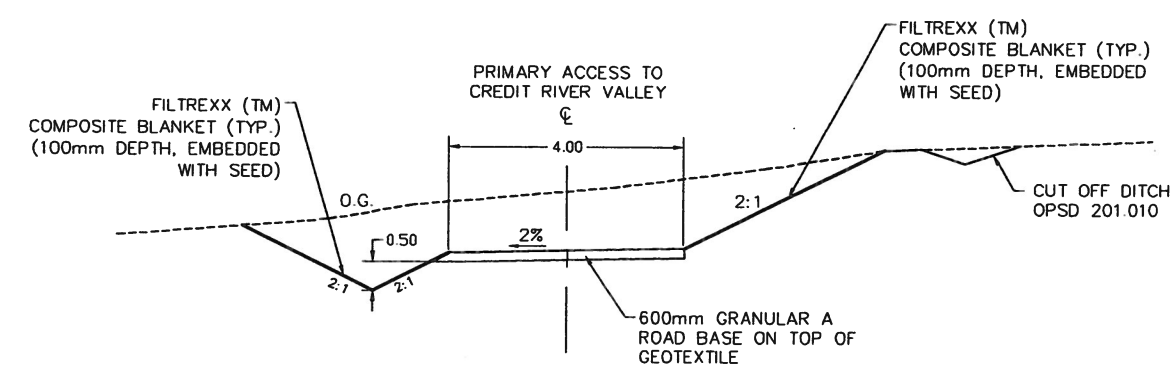
MINISTRY OF TRANSPORTATION, ONTARIO

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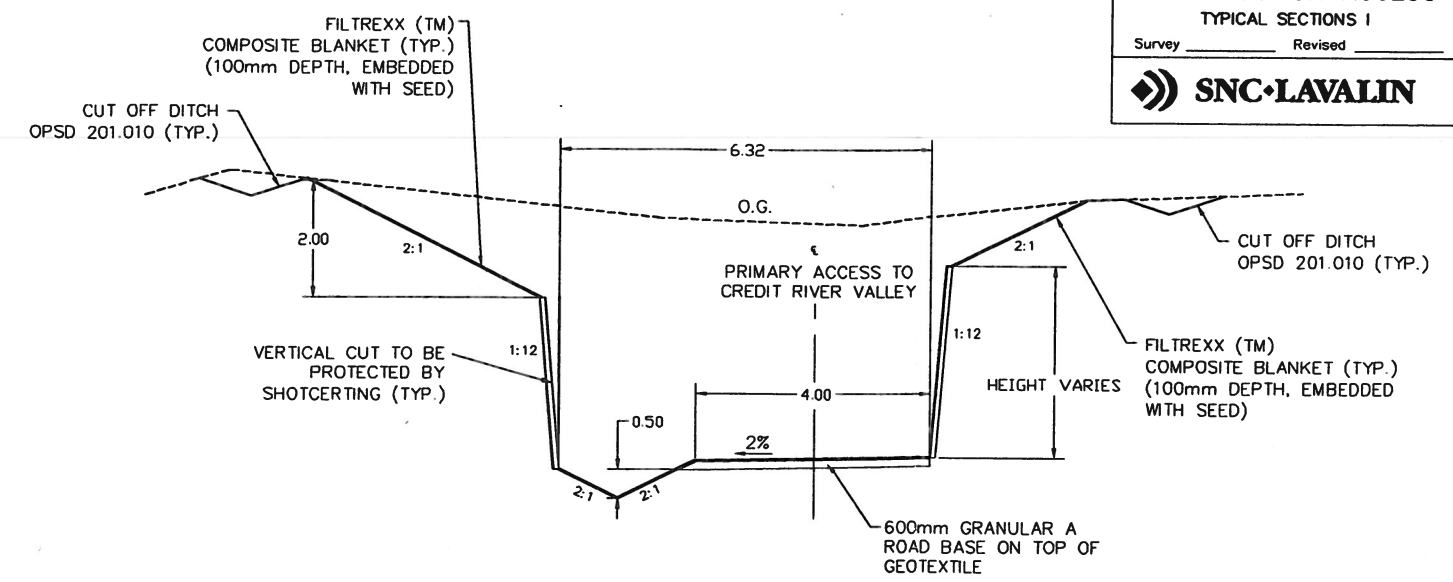
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METRIC
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AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

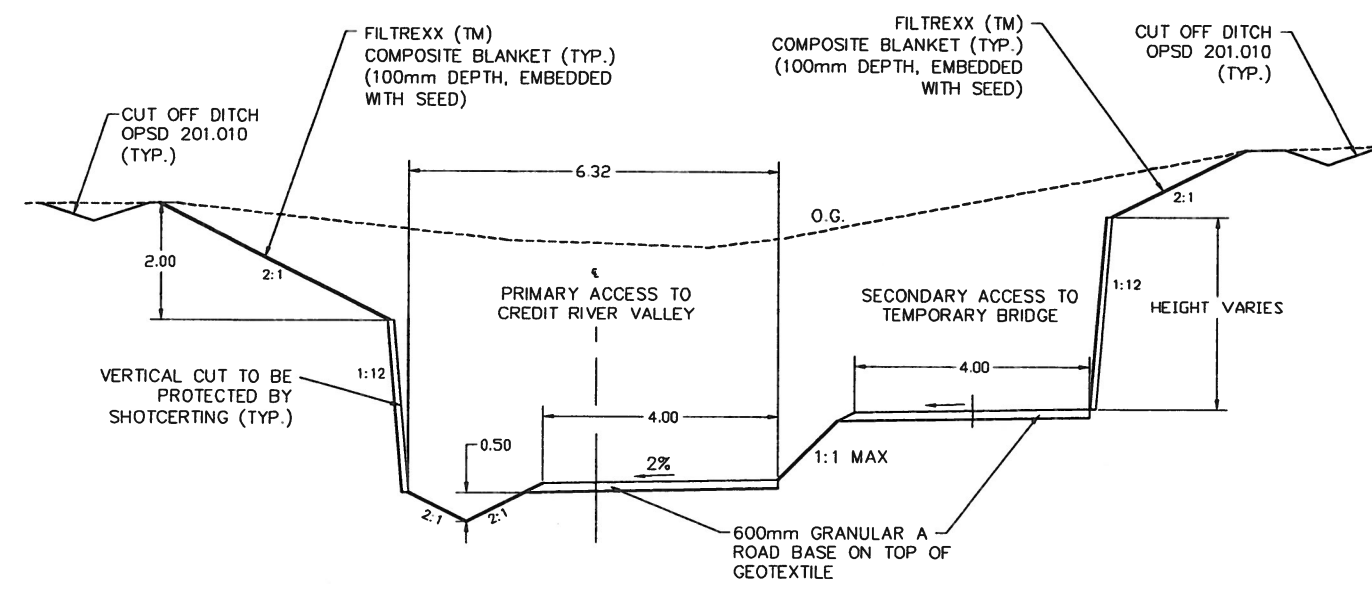
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WP No	2186-07-00
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Survey	Revised
 SNC-LAVALIN	



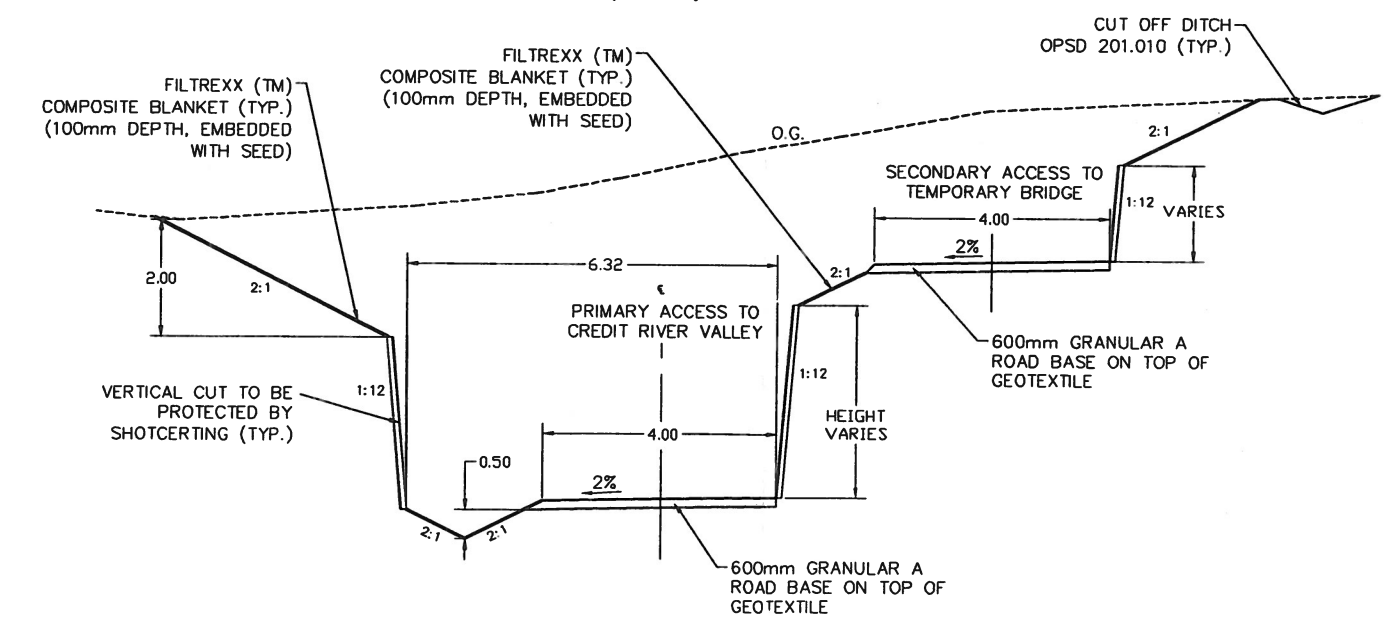
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STA. 0+000 TO STA. 0+310
(N.T.S.)



CONSTRUCTION ACCESS TYPICAL SECTION
STA. 0+310 TO STA. 0+450
(N.T.S.)



CONSTRUCTION ACCESS TYPICAL SECTION
STA. 0+450 TO STA. 0+483
(N.T.S.)



CONSTRUCTION ACCESS TYPICAL SECTION
STA. 0+483 TO STA. 0+500
(N.T.S.)

NOTE:
STATIONING REFERS TO PRIMARY CONSTRUCTION ACCESS
TO CREDIT RIVER VALLEY.
SEE NEW CONSTRUCTION DRAWINGS FOR DETAILS.

PH-2-202 88-10

MINISTRY OF TRANSPORTATION, ONTARIO

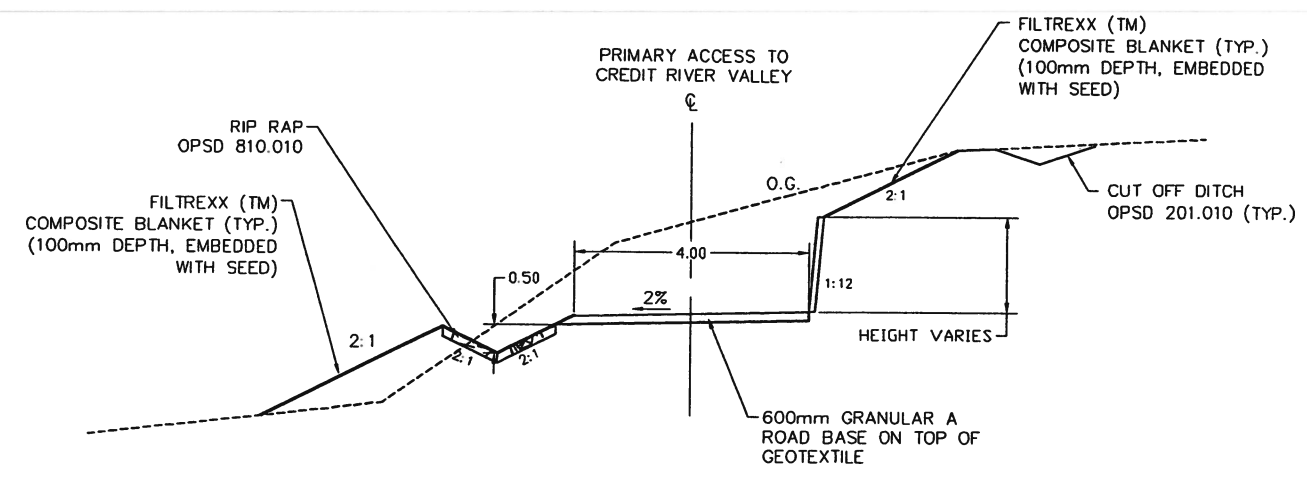
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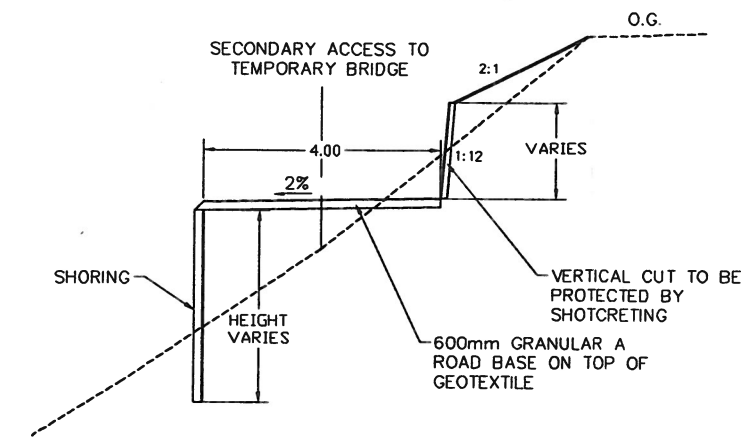
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

PLATE No	
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WP No	2186-07-00
CONSTRUCTION ACCESS	
TYPICAL SECTIONS II	
Survey	Revised
SNC-LAVALIN	

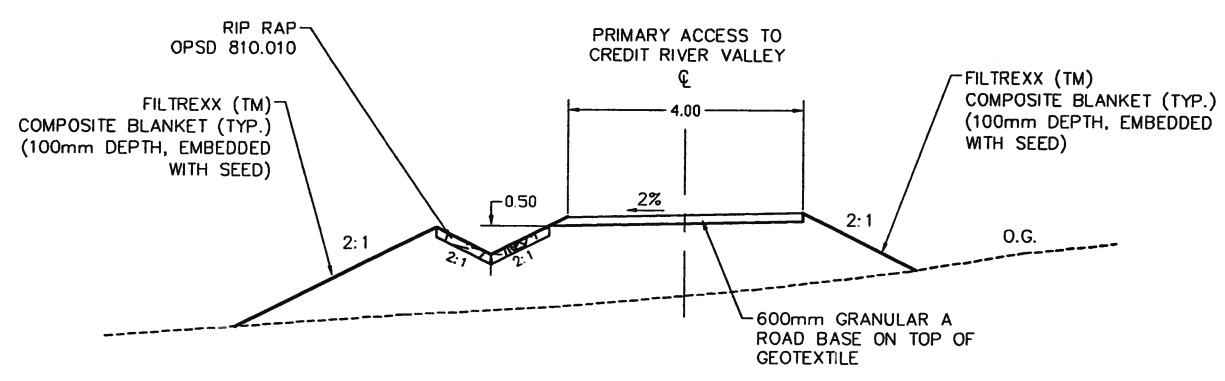
SHEET



CONSTRUCTION ACCESS TYPICAL SECTION
STA. 0+500 TO STA. 0+510
(N.T.S.)



TYPICAL SECTION OF SECONDARY ACCESS
TO TEMPORARY BRIDGE
(N.T.S.)



CONSTRUCTION ACCESS TYPICAL SECTION
STA. 0+510 TO STA. 0+554
(N.T.S.)

NOTE:
STATIONING REFERS TO PRIMARY CONSTRUCTION ACCESS
TO CREDIT RIVER VALLEY.
SEE NEW CONSTRUCTIOTN DRAWINGS FOR DETAILS.

Appendix D

Foundation Comparison

COMPARISON OF CUT DESIGN ALTERNATIVES

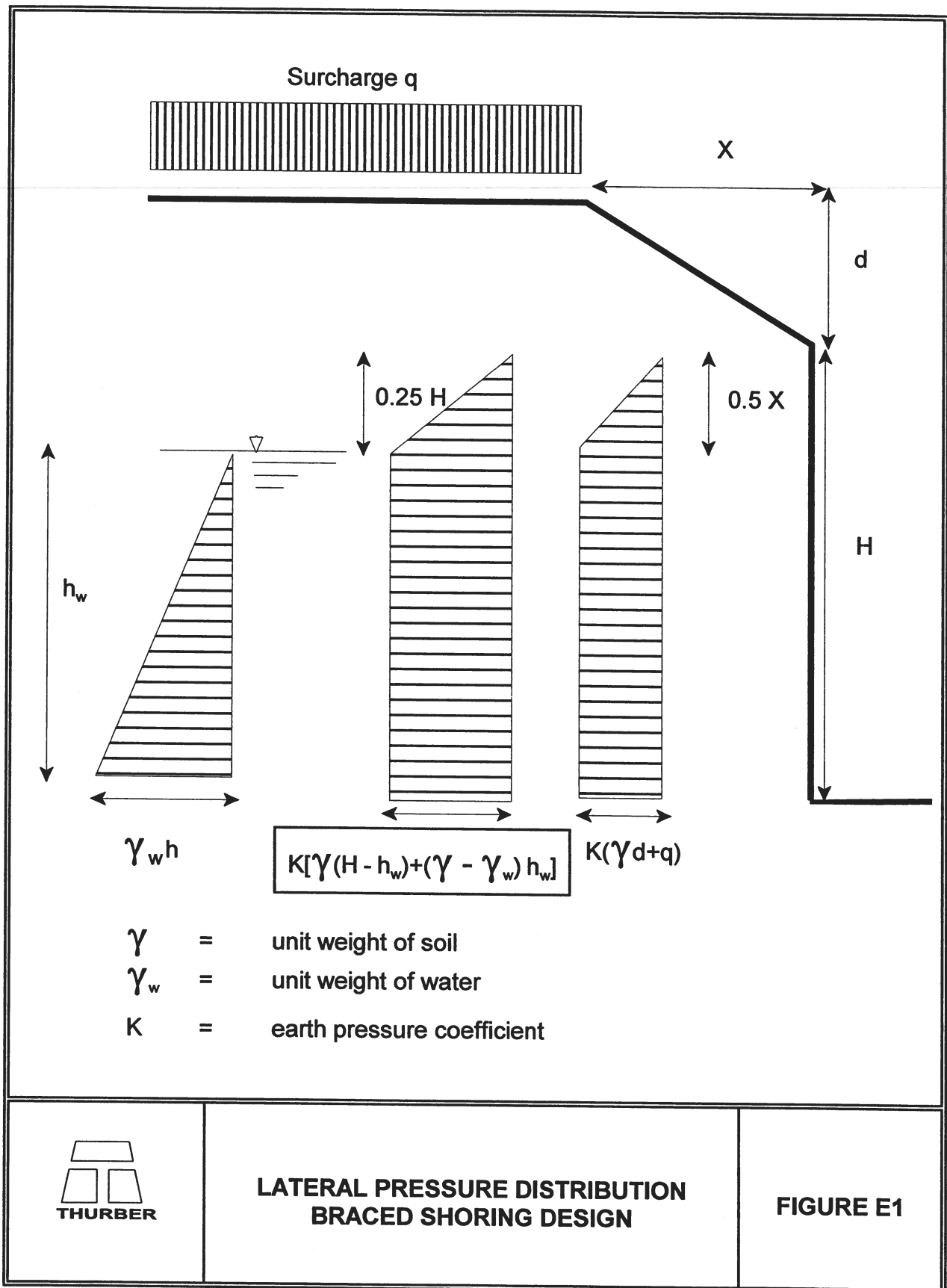
Option 1 Full Depth Inclined Slopes	Option 2 Inclined Slopes on Shotcreted Vertical Cuts in Intact Shale	Option 3 Inclined Slopes on Shored Vertical Cuts in Intact Shale	Option 4 Inclined Slopes on Multi-Tiered Bench Cuts in Intact Shale
<p>Advantages:</p> <ul style="list-style-type: none"> i. 2H : 1V in soils and highly weathered shale would be stable. ii. 1H : 1V or flatter slopes in intact shale would be stable during the design life of the cut. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. 1H : 1V or flatter slopes in intact shale involve extensive excavation and eventual backfilling, erosion protection, site space constraints and periodic maintenance requirements after completion of current rehabilitation work. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. 2H : 1V in soils and highly weathered shale would be stable. ii. Minimal excavation and requirements for rock face maintenance. iii. Rock face is protected from further weathering and deterioration. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Requires involvement of specialist designers and contractors. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. 2H : 1V in soils and highly weathered shale would be stable. ii. Minimal excavation and requirements for rock face maintenance. iii. Rock face is protected. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Requires involvement of specialist designers and contractors. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. 2H : 1V in soils and highly weathered shale would be stable. ii. 1H : 4V intact shale faces are less exposed to rainfall and surface runoff, and do not collect much snow; 3 m wide benches collect ravelling rock fragments. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Involves substantial excavation, erosion protection and maintenance requirements after completion of current rehabilitation work.

Appendix E

Figure

19-92-92





Appendix F

List of SPs and OPSS

Copies of Selected SPs

Suggested Text for Selected NSSPs

1. List of Special Provisions and OPSS Documents Referenced in this Report

- SP 299 S08
- SP 299 S09
- SP 903 S01
- SP 999 S26
- OPSS 206
- OPSS 501
- OPSS 539
- OPSS 572
- OPSS 902
- OPSS 1010

2. Suggested Text for NSSP on “Rock Excavation”

The strength of the shale bedrock increases with depth and there is presence of very hard limestone and/or siltstone interbeds within the shale bedrock. Bulk excavation and pre-drilling through the sound shale and the hard interbeds may be difficult. As such, intensive use of pneumatic rock splitting/breaking equipment, ripping machinery or other methods of loosening the bedrock may be required and should be available on site to assist in rock excavation.

3. Suggested Text for NSSP on “Installation of Pile Sockets”

For pile socket installation in the shale bedrock, in addition to augering equipment, rock coring equipment or pneumatic rock splitting/breaking equipment shall be required to penetrate the hard interbeds.

4. Suggested Text for NSSP on “Installation of Augered Caissons”

Caisson installation through the till may encounter cobbles, boulders or rock slabs and the installation equipment must be capable of dislodging and removing such obstructions. Temporary steel liners shall be available on site to support the sidewalls of the caisson holes where required. The shale bedrock contains hard limestone interbeds. Excavation and augering through the hard interbeds may

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be difficult. As such, rock coring equipment or pneumatic rock splitting/breaking equipment should be available on site to assist in excavation and drilling of caisson sockets. The base of each caisson shall be cleaned of loose and disturbed material prior to placement of concrete. The caisson excavation shall be dewatered (if necessary) to allow cleaning of the base and walls. Concrete shall be placed with minimum delay after the socket is cleaned and approved.