

**FOUNDATION INVESTIGATION AND DESIGN REPORT
SLOPE FAILURE AT W-N/S RAMP
FORMER HIGHWAY 403 (GARDEN AVENUE)/
HIGHWAY 403 INTERCHANGE
GWP 30-00-00, AGREEMENT NO. 3005-E-0017
MINISTRY OF TRANSPORTATION - SOUTHWESTERN REGION**

Submitted to:

Philips Engineering Ltd.
3215 North Service Road
P.O. Box 220
Burlington, Ontario
L7R 3Y2

DISTRIBUTION:

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LIST OF SYMBOLS

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PART A – FOUNDATION INVESTIGATION REPORT

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Philips Engineering Ltd. (Philips) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the detail design work for GWP 30-00-00. The project involves the detail design works along Garden Avenue, formerly Highway 403, from 0.6 km north to 0.9 km south of the Garden Avenue/Highway 403 Interchange in Brantford, Ontario. The proposed works within the project limits will consist of the following:

- (a) Widening of Garden Avenue to accommodate auxilliary lanes.
- (b) Signalization of the W-N/S and E-N/S ramp terminals.
- (c) Widening of the W-N/S and E-N/S ramps to accommodate auxilliary lanes.
- (d) Intersection improvements at the intersection of Garden Avenue and Sinclair Boulevard.
- (e) Provision of a signalized intersection at Garden Avenue and the proposed Industrial Park Access Road.
- (f) Provision for 350 metres of Industrial Park Access Road and a new commuter parking lot with access to the Industrial Park Access Road.
- (g) Signalization at Henry Street.
- (h) Signal interconnect to all signals within the project limits.
- (i) High mast illumination and conventional illumination.
- (j) Pavement rehabilitation/reconstruction of Garden Avenue.
- (k) Pavement rehabilitation of the W-N/S and E-N/S ramps.

The foundation aspects for the high mast light component of item (i) are addressed in the appropriate foundation investigation and design report under separate cover. Items (b), (e), (g) and (h) are addressed by others.

This report addresses the foundation investigation for an area of the slope adjacent to the W-N/S ramp where scour and slope failure has occurred. Remediation of the slope failure is necessary for the protection of the existing ramp and the successful completion of the proposed widening of the W-N/S ramp. As part of the investigation, the channel reach between the 4.0 x 3.1 metre concrete box culvert under the W-N/S ramp and the 5.0 x 3.7 metre concrete box culvert beneath Garden Avenue was visually assessed.

The purpose of the foundation investigation is to determine the subsurface conditions at the locations of the proposed works by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P51-3083 dated August 2, 2005 and a work plan for additional foundation engineering services related to the affected failure zone detailed in Golder Associates' letter dated November 4, 2005 and an email from the MTO dated December 9, 2005. The work was carried out in accordance with our Quality Control Plan for Foundations Engineering dated August 18, 2005. Philips provided Golder Associates with preliminary drawings and topographic mapping for this project in digital format.

2.0 SITE DESCRIPTION

GWP 30-00-00 comprises the design of the widening and improvements of Garden Avenue from 0.6 kilometres north to 0.9 kilometres south of the Garden Avenue/Highway 403 Interchange in the City of Brantford, Ontario. Subsequent to the initiation of the project, it was noted that remediation of toe erosion and slope failure occurring on the bank of a tributary of the Fairchild Creek which flows beneath the W-N/S ramp was required both to maintain the stability of the existing ramp and as a part of the proposed ramp improvements. The location of the project is shown on the Key Plan, Figure 1.

Apart from some industrial areas between Sinclair Boulevard and the Garden Avenue/Highway 403 Interchange, a commercial development in the northwest quadrant of the interchange and off Henry Street and Garden Avenue, the land bordering the project limits is mainly agricultural and consists of vacant fields with shrubs or small woodlots.

Immediately west and south of the interchange, a tributary to the Fairchild Creek flows southeastwards and skirts the south side of the W-N/S ramp. At normal stage, the creek water level is approximately at elevation 205 metres in the vicinity of the W-N/S ramp. The W-N/S ramp is approximately at elevation 212 metres or 7 metres above the creek. The old road bed of the former Highway 403 alignment is located west of the creek. A small pond is situated east of the failure scarp and southwest of Garden Avenue and the ramp.

The failure scarp is situated at approximately station 10+310 on the W-N/S ramp. During a field reconnaissance on October 12, 2005, it was noted that the scarp of the slope failure extends laterally along the north creek bank for a distance of about 20 metres. The slope failure had occurred at a bend in the creek. The creek bank in the affected area is approximately 3.2 metres high and is currently near vertical. The slope inclination above the scarp is approximately 4.5 horizontal to 1 vertical. The overall slope height in this area is about 8 metres. However, only an approximately 9 metre wide area remains between the outer edge of the ramp pavement and the crest of the failure scarp. The failure scarp is generally bare except for vegetation which is present on the top of a mass of soil which slumped after the toe was undercut. The creek bank and slopes are generally well vegetated with grass, shrubs and occasional small trees. Photographs of the failure area are shown in Appendix C.

According to the original design drawings provided by Philips, immediately upstream of the failure scarp, the tributary crosses under the current Highway 403 and the W-N/S ramp through two 4.0 x 3.1 concrete culverts and discharges into an area which is protected by a 300 millimetre thick layer of rip-rap. The design drawings indicate that the rip-rap area extended 5 metres upstream and 25 metres downstream of the outlet. The invert of the ditch between the W-N/S ramp and the former Highway 403 roadbed was protected with a 300 millimetre thick, 4 metre

wide layer of rip-rap for a distance of 27 metres upstream of the discharge area. Downstream of the armoured discharge area, a 70 metre long unlined trapezoidal channel with a 4 metre wide base and 2 horizontal to 1 vertical side slopes was constructed. A 300 millimetre diameter CSP culvert, which is not shown on the design drawings, discharges into the tributary at the end of the trapezoidal channel. This culvert is located approximately 20 metres upstream of the failure.

During the site reconnaissance, the creek bed was observed to typically consist of fine-grained silt and clayey materials. However, it was noted that remnants of a cobble stone and geotextile lining were present in the realigned portion of the creek. Most of the lining had failed and the cobbles had washed downstream. Localized instability was noted at the toes of the unprotected creek channel side slopes both upstream and downstream of the main failure. The creek banks within the former trapezoidal channel were generally steeper than 2 horizontal to 1 vertical.

2.1 Site Geology

This project lies within the physiographic region of southwestern Ontario known as the Norfolk Sand Plain¹. Near the area of the site, the Horseshoe Moraines intersect the sand plain. In the area of the site, a discontinuous veneer of surficial sandy soil deposited in glacial Lakes Whittlesey and Warren overlies extensive deposits of stratified clays and silts associated with the Haldimand Clay Plain.

Based on the Ontario Department of Mines and Northern Affairs Map 2241 entitled “Granular Deposits of the Brantford Area” dated 1972, the soils at the site consist of glaciolacustrine deep water sediments, mainly Lake Warren and younger. These are predominantly stratified to varved silts and clays with minor sand and are locally overlain by a veneer of sand.

Bedrock in the area of the site is considered to consist of shale and dolomite belonging to the Salina Formation of Upper Silurian Age. Bedrock surface topographical mapping indicates that the bedrock surface in the area of the site is likely to be at about elevation 180 metres or some 25 to 40 metres below the existing ground surface.

¹ L.J. Chapman and D.F. Putnam: The Physiography of Southern Ontario, Third Edition. Ontario Geological Survey, Special Volume 2, 1984.

3.0 INVESTIGATION PROCEDURES

The field work for this portion of the investigation was carried out on November 10, 2005 and December 22, 2005. In November 2005, three boreholes, numbered 1 to 3, were drilled in the creek bed adjacent to the failure area using hand operated equipment provided by Golder Associates. In addition, the stratigraphy exposed in the failure scarp and condition of the channel adjacent to the ramp was also documented by our field personnel and soil samples obtained. In December 2005, borehole 4 was drilled to a depth of 15.8 metres on the shoulder of the W-N/S ramp above the failure scarp.

The locations of the boreholes are shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole locations, ground surface elevations and depths.

<u>BOREHOLE</u>	<u>LOCATION (m)</u>		<u>GROUND SURFACE ELEVATION</u>	<u>BOREHOLE DEPTH</u>
	<u>Northing</u>	<u>Easting</u>	(m)	(m)
1	4780879.4	246722.4	204.51	3.05
2	4780882.8	246715.0	205.07	3.05
3	4780868.1	246733.7	204.31	3.05
4	4780908.3	246744.1	212.16	15.85

Boreholes 1 to 3 were advanced by Golder Associates staff using manual drilling techniques to a depth of 3.05 metres. Manual penetration and sampling was carried out utilizing a standard 50 millimetre outside diameter split spoon sampler and a 31.8 kilogram hammer. The recorded drilling resistances have been adjusted to a standard 63.6 kilogram hammer and these are shown on the Records of Boreholes as approximate 'N' values.

Borehole 4 was drilled using a truck mounted mobile power auger supplied and operated by a specialist drilling contractor. In the borehole, samples of the overburden were obtained at suitable intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures. In situ vane shear strength testing was carried out in the softer cohesive strata, where feasible. The borehole was terminated at 15.8 metres below the existing ground surface. Groundwater conditions in the borehole were observed throughout the drilling operations. A piezometer and a standpipe were installed in borehole 4 in order to monitor the long term groundwater levels. Groundwater observations are indicated on the corresponding Record of Borehole sheets and summarized in Section 4. The

borehole was backfilled in accordance with current MTO procedures and Ontario Regulation 128/03.

The field work was supervised on a full-time basis by experienced members of our engineering staff who located the boreholes in the field, directed the drilling, sampling and in situ testing operations and logged the boreholes. The samples were identified in the field, placed in labeled containers and transported to our London laboratory for further examination and testing. Index and classification tests consisting of water content determinations, grain size distribution analyses and Atterberg limits determinations were carried out on selected samples. The results of the testing are shown on the Records of Boreholes and on Figures A-1 to A-4, inclusive, in Appendix A.

The as-drilled borehole locations and ground surface elevations at the borehole locations were determined by members of our staff relative to geodetic datum. The locations of the boreholes are shown on the Record of Borehole sheets and on Drawing 1.

The geotechnical information gathered during the field investigation was supplemented by the addition of borehole 4 which was advanced on March 1, 1988 for the culvert at station 10+273 Garden Avenue. The Record of Borehole 4, reproduced from foundation investigation report Geocres No. 40P1-81, is presented in Appendix B.

4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the in situ testing and the laboratory testing carried out on selected samples, are given on the attached Record of Borehole sheets following the text of this report and in Appendices A and B and on Drawing 1. The stratigraphic boundaries are inferred from non-continuous samples and observations of drilling resistance and, therefore, may represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations.

The soil conditions exposed in the failure scarp generally consisted of surficial topsoil, silt and clayey silt or silty clay with silt layers. The boreholes typically encountered surficial topsoil, underlain by clayey silt and silty clay and layers of silt.

The locations and elevations of the boreholes are shown on the attached Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes is provided on the Record of Borehole sheets and is summarized in the following sections.

4.2 Conditions Exposed in Failure Scarp

The stratigraphy exposed in the currently (December 2005) near vertical failure scarp is presented in Table I.

4.2.1 Topsoil

A topsoil depth of 240 millimetres was noted near the top of the failure scarp.

4.2.2 Silt

The topsoil was underlain by a 2.7 metre thick layer of silt from elevation 207.5 metres. The sample of the silt had a measured water content of 25 per cent. The gradation curve of the silt sample is shown in Figure A-1 in Appendix A.

4.2.3 Silty Clay

Silty clay was observed beneath the silt layer from elevation 204.9 metres. The silty clay deposit contained silt partings and extended to the creek water level.

4.3 Creek Bed and Slope

Boreholes 1, 2 and 3 were advanced in the creek bed adjacent to the failure area. Borehole 4 was drilled near the edge of the W-N/S ramp through the slope immediately above the failure scarp.

4.3.1 Pavement Structure

Ninety millimetres of asphalt from the W-N/S ramp shoulder pavement structure was encountered at the surface of borehole 4.

The asphalt was underlain by a 0.5 metre thick layer of granular roadbase fill from elevation 211.6 metres.

4.3.2 Silt

Layers of silt were intercepted in the boreholes beneath the clayey silt layer in borehole 1 at elevation 203.3 metres, beneath the granular fill in borehole 4 at elevation 211.6 metres and beneath the upper clayey silt in borehole 4 at elevation 200.3 metres. The silt in borehole 4 was noted to contain layers of silty clay. Grain size distribution curves for silt samples are shown on Figure A-1 in Appendix A.

The upper silt layer in borehole 4 is 2.4 metres thick and compact with N values ranging from 14 to 17 blows per 0.3 metres. Water contents of 28 to 31 per cent were measured on samples of the silt retrieved from the standard penetration testing.

The lower silt layers in boreholes 1 and 4 are 0.5 to 1.1 metres thick and very loose to loose with N values of 2 and 8 blows per 0.3 metres. Water contents of 23 and 25 per cent were measured on samples of these layers.

4.3.3 Silty Clay

Silty clay was found in boreholes 1 and 3 of the current investigation and in borehole 4 of the previous investigation.

In the boreholes advanced for the current investigation, silty clay was encountered at the surface of borehole 3 and beneath the silt in borehole 1 at elevation 202.8 metres. Silty clay was also encountered at the surface of borehole 4 of the previous investigation. An extensive deposit of silty clay, 22.4 metres thick, was found in borehole 4 of the previous investigation. The silty clay was noted to be stratified with silt layers or seams. Boreholes 1 and 3 of the current investigation were terminated in the silty clay strata.

Standard penetration test values of 2 to 13 blows per 0.3 metres were measured in the silty clay deposits. The silty clay is firm to very stiff but generally stiff based on undrained in situ vane shear strengths of 26 to 108 kilopascals with an average of 52 kilopascals. The vane sensitivity ranged from 1.0 to 4.1 with an average of 2.6. An unconfined compression test conducted in borehole 4 of the previous investigation indicated an undrained shear strength of 17 kilopascals.

Water contents of the silty clay strata varied between 24 and 34 per cent with an average of 30 per cent. The silty clay found in borehole 4 of the previous investigation is of low to intermediate plasticity based on average plastic and liquid limits of 16 and 31 per cent and an average plasticity index of 15 per cent. The results of the grain size analysis of a single sample from the current investigation are presented on Figure A-2.

4.3.4 Clayey Silt

Clayey silt was encountered from the ground surface of boreholes 1 and 2, below the silty clay in borehole 4 at elevation 209.3 metres and below the lower silt layer from elevation 199.2 metres. The clayey silt layers were stratified with layers of silt and silty clay. Where fully penetrated in boreholes 1 and 4, the upper layers were 1.2 and 6.3 metres thick, respectively. Boreholes 2 and 4 were terminated in the clayey silt after exploring it for some 2.9 to 3.1 metres.

Standard penetration test N values in the clayey silt ranged from 2 to 10 blows per 0.3 metres and averaged 5 blows per 0.3 metres. The clayey silt is stiff to very stiff based on undrained in situ vane shear strengths of 65 to greater than 144 kilopascals with an average of 90 kilopascals. The vane sensitivity was between 2.0 and 3.3 with an average of 2.4. The water content of samples varied between 27 and 35 per cent with an average of 31 per cent. Gradation curves for four samples of clayey silt are presented on Figure A-3. The results of the Atterberg limits testing are shown on the Plasticity Chart on Figure A-4 and indicate a low plasticity clay based on the three samples tested.

4.3.5 Bedrock

Dolomite bedrock of the Salina formation was proven in borehole 4 from the previous investigation. The bedrock was encountered at elevation 184.1 metres. A recovery of 100 per cent for a 1.3 metre length of BXL core was reported.

4.4 Seepage and Groundwater Conditions

Seepage was not observed in the face of the failure scarp at the time of the investigation.

Groundwater conditions in the boreholes were observed during and on completion of drilling and sampling. Details of the groundwater conditions encountered in the boreholes are provided below and on the Record of Borehole sheets.

Groundwater was encountered in boreholes 1, 2 and 3 of the current investigation near the creek between elevations 204.2 and 204.6 metres in boreholes. The water level in the creek was surveyed at elevation 204.57 metres on November 10, 2005.

Groundwater was encountered at elevation 209.1 metres in borehole 4 of the current investigation. The most recent groundwater measurements were taken on January 9, 2005. Groundwater within the upper clayey silt at borehole 4 was measured at elevation 207.4 metres in the standpipe. Within the lower clayey silt, the groundwater was measured at elevation 207.9 metres in the piezometer in borehole 4.

The water level in the pond located downstream of the failure scarp was at elevation 205.67 metres on December 22, 2005. The surface of the pond was frozen at the time of the investigation.

In borehole 4 from the previous investigation, groundwater was encountered at elevation 203.3 metres.

The following table provides a summary of the groundwater levels.

SUMMARY OF GROUNDWATER LEVELS

BOREHOLE	GROUND SURFACE ELEVATION (m)	ENCOUNTERED GROUNDWATER LEVEL		MEASURED GROUNDWATER LEVEL January 9, 2006
		Depth (m)	Elevation (m)	Elevation (m)
1	204.51	0.15	204.36	-
2	205.07	0.46	204.61	-
3	204.31	0.15	204.16	-
4 – Piezometer	212.16	3.05	209.11	207.86
4 – Standpipe	212.16	-	-	207.37
MTO 4	205.5	2.2	203.3	-

The groundwater levels are expected to fluctuate seasonally and are likely to be higher during periods of sustained precipitation and/or spring melt.

5.0 REVIEW OF AERIAL PHOTOGRAPHY

Aerial photographs covering the subject area were examined in order to see if there was any evidence of prior failures and to investigate the history of the site. Photographs dating from 1951, 1964, 1972, 1982 and 1989 were inspected. Failure scarps or significant erosion features were not apparent in any of the photographs.

The 1951 aerial photograph shows that Garden Avenue had been constructed and the tributary to Fairchild Creek appeared to be approximately in its current alignment. Land use around the subject site was primarily agricultural and a cluster of buildings was present immediately northeast of the site. The existing W-N/S and N-E ramps are now situated at the former location of this cluster. The creek passed through open fields or grassland and shrub or trees are noticeable along the banks downstream of the current failure scarp. The existing pond is not evident in this photograph.

By 1964, construction of the former Highway 403 alignment was underway. The surrounding land remained agricultural and the buildings were still present. It is assumed that during this time the tributary west of the cluster of buildings was realigned and channelized as noted in Section 2.0. The pond is now present in the photograph next to a road leading from the nearby cluster of buildings. The topographical mapping for the site indicates that a remnant of the former road bed is still apparent next to the wire fence east of the pond.

The 1972 air photograph showed that construction of the former Highway 403 was complete and the highway was in use. Garden Avenue no longer continued straight through to Henry Street. The channelized section of creek upstream of the current failure scarp is evident in the air photograph. There appeared to be little change in the adjacent land use.

Except for increasing urbanization in the City of Brantford west of the subject site, little had changed in the vicinity of the site by 1982.

By 1989, urban development had spread to Henry Street just south of the site. However, examination of the 1989 air photograph indicates that the site appears to remain relatively unchanged.

6.0 MISCELLANEOUS

The investigation was carried out using hand operated equipment supplied and operated by Golder Associates and power equipment operated by B.U.D Environmental Services Ltd., an Ontario Ministry of Environment licensed well contractor. The field operations were supervised by Mr. Lubo Kosc under the direction of Mr. David J. Mitchell. The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Ms. Dirka U. Prout, P. Eng. under the direction of the Project Manager, Mr. Philip R. Bedell, P. Eng. This report was reviewed by Mr. Fintan J. Heffernan, P. Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

GOLDER ASSOCIATES LTD.

Dirka U. Prout, P. Eng.

Philip R. Bedell, P. Eng.
Principal

Fintan J. Heffernan, P. Eng.
Designated MTO Contact

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PART B – FOUNDATION DESIGN REPORT

**SLOPE FAILURE AT W-N/S RAMP
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7.0 ENGINEERING RECOMMENDATIONS

7.1 General

This section of the report provides our recommendations on the geotechnical aspects of the design of remedial works to address the existing slope failure and to mitigate the impacts of slope erosion in the vicinity of the W-N/S ramp. Although portions of these works will be part of the overall improvements for the Garden Avenue interchange, emergency measures will be required to stabilize the 20 metre section of the creek bank where toe erosion, resulting in a failure of the creek bank which will impact the W-N/S ramp, is most severe.

Given the significance of the existing slope failure and based on the subsurface conditions prevailing at the site together with environmental considerations and the MTO's schedule for any proposed works, the following program of channel/side slope stabilization is recommended:

- i) an immediate response by MTO forces to contain/limit ongoing regression of the existing vertical scarp of the slope failure and stabilize the slope. This work is required to maintain the integrity of the existing W-N/S ramp; and
- ii) a subsequent study/work project for the entire reach of the creek regime to identify and implement erosion control and slope stabilization works for this reach of the waterway.

This report specifically addresses item i). Additional foundation investigation, engineering and input will be required for item ii).

It should be noted that the interpretation and recommendations provided herein are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

7.2 Channel Side Slope

7.2.1 Existing Conditions

The reach of the watercourse between the 4.0 x 3.1 metre concrete box culvert under the W-N/S ramp and the downstream 5.0 x 3.7 metre concrete box culvert under Garden Avenue was visually assessed for signs of erosion and instability. Although the creek banks and slopes were well vegetated with grass, shrubs and occasional small trees, the creek bed and walls were noted to be composed of highly erodible, fine grained granular and cohesive soils. Localized instability was noted at the toes of the unprotected creek channel slopes within the subject reach. The problem was most acute near Station 10+310 of the W-N/S ramp where an approximately 3.2

metre high near vertical failure scarp had developed due to undercutting of the toe and subsequent slumping. The failure scarp extended approximately 10 metres upstream and downstream of Station 10+310.

Further, it was observed that most of the geotextile and rip-rap erosion protection which was placed in the discharge area of the 4.0 x 3.1 metre culvert had failed and the stone had been washed downstream.

The crest of the existing scarp is at elevation 207.8 metres. The conditions exposed in the failure scarp were surficial topsoil over silt from elevation 204.5 metres to the approximate creek water level which was surveyed at 204.6 metres on November 10, 2005. Silty clay with silt layers underlies the silt stratum to elevation 203.9 metres. No groundwater seepage was observed from the face of the failure scarp. Borehole data from the current investigation and the Ministry of Transportation, Ontario (MTO) Geocres files indicates that silty clay layer is extensive and extends to bedrock near elevation 184 metres. The silty clay was found to be firm to stiff with an undrained shear strength of about 50 kilopascals.

7.2.2 Slope Stability

Slope stability analyses considering wheel loading in the ramp area were conducted to assess the global and local stability of the existing creek bank side slopes. Although the global and local slip surfaces in areas not subject to erosion were judged to be stable with factors of safety exceeding 1.3, it should be noted that the existing area of instability presents an immediate danger to the W-N/S ramp. In the failure zone, the global and local slip surfaces had factors of safety of about 1. Therefore, it is critical that emergency repairs to the failure scarp be conducted in order to protect the ramp and maintain the safety of the public and to facilitate future projects which involve the W-N/S ramp and Garden Avenue/Highway 403 Interchange improvements.

It should also be noted that extensive and active toe erosion is occurring along much of subject reach; the potential for further failures similar to the present one is therefore high. Therefore, additional studies including geomorphological and environmental assessments as well as the foundation engineering issues of the entire reach of the waterway will be required in the near future.

7.2.3 Emergency Remedial Works

Based on the results of this investigation, the following emergency remedial works are required:

- The immediate protection of the existing highly erodible, fine grained soils to reduce the risk of failure of the W-N/S ramp;
- Drainage improvements to prevent slope deterioration due to run-off will also be required in the area of the failure scarp and any other location where gullyng of the slope is occurring; and
- stabilization of the slope.

The above-noted remedial works generally constitute remedial works for emergency measures. A detailed engineering and environmental study is required to assess the long term stability and erosion potential(s) of the subject reach.

7.2.3.1 Immediate Response

To correct the ongoing erosion and slope regression, a 0.5 metre thick layer of 200 millimetre rock should be placed on Ontario Provincial Standard Specifications (OPSS) 1860, Type 2 geotextile having a FOS of 70 microns.

Alternatively, a separation layer consisting of a minimum of 300 millimetres of Granular B, Type II, could be used. However, since this material is unlikely to meet filter criteria between the native silt and the rock, it would require removal for future work.

The treatment should extend at least 1 metre into the creek and 1 metre up the slope. The lateral limits should extend at least 1 metre beyond the limits of the current failure as shown on Drawing 1. This work should be carried out as soon as feasible and no later than the spring of 2006.

Stabilization of the existing failed slope is required to preserve the integrity of the W-N/S ramp. Further, the W-N/S ramp cannot be widened if the slope remains in its original configuration or if it is flattened. In order to provide a minimum factor of safety of 1.3, the creek should be realigned and the failed area should be restored to a 2 horizontal to 1 vertical slope using compacted Granular B Type III properly benched into the existing bank. A 2 metre wide bench is required at mid-height should the height of the cut slope be in excess of 6 metres. The slope restoration should extend 3 metres upstream and downstream of the failure scarp in order to properly key in the backfill. The approximate limits of the required slope restoration are shown on Drawing 1. It is recommended that the stabilization measures be implemented early in 2006 or combined with the immediate response work detailed above.

The results of the hydraulic channel assessment provided by Philips indicate that a rock size of 200 millimetres should be provided for erosion protection. Type 2 geotextile (OPSS 1860) having a FOS of 70 microns should be provided between the rock and the Granular B, Type III. Alternatively, the use of suitably graded granular filters could be considered. However, this will necessitate a Granular B gradation close to the fine limit of Type III and a 0.3 metre thick layer of filter material. Figure 2 provides gradation envelopes for the native silts, Granular B Type III filter and the rock. Based on the requirement to properly produce and very carefully place the very small volumes of the two granular filter materials, together with the necessity of stringent construction supervision, this option is not recommended.

Surface drainage, in the form of swales or drainage ditches along the W-N/S ramp, should be provided in order to prevent concentrated flows from the ramp area eroding the face of the channel side slopes.

Following the slope stabilization works, the slopes should be adequately seeded and vegetated.

Consideration was given to the need for staged construction, lightweight fill and/or geosynthetic reinforcement. Due to the limited size and nature of the affected area, none of these treatments are considered appropriate.

7.2.4 Long Term Remedial Measures

Failure of the erosion protection placed at the outlet of the concrete 4.0 by 3.1 metre concrete box culvert, changes in the geometry of the trapezoidal channel and ongoing toe erosion along the entire subject reach were noted during the site reconnaissance. The channel works, which were part of the original construction for the current Highway 403 alignment, have deteriorated since their construction in the 1980's. Detailed engineering, environmental and geomorphological studies of the channel reach should be conducted in order to assess the channel characteristics and predict the potential for channel erosion to adversely affect the future stability of the slope below the W-N/S ramp. The study should assess both short term and long term impacts and identify areas where remediation is required. An evaluation of the stability of the slopes within the channel reach, particularly beneath the ramp, should be required as part of the engineering study.

Subject to the results of the detailed assessments, completion of the following remedial works is recommended:

- Armouring of the channel reach between the 4.0 x 3.1 metre culvert to the culvert under Garden Avenue, where necessary, in order to minimize or prevent excessive toe erosion;
- Erosion and scour protection at, and immediately downstream of, the outlet of the concrete 4.0 by 3.1 metre concrete culvert should be reestablished;

- Erosion protection at the outlet of 300 millimetre diameter CSP culvert should be provided or improved, as necessary;
- The configuration of the original trapezoidal channel should be restored;
- Provision of adequate surface drainage in order to prevent slope deterioration due to run-off.

Due to the fine-grained nature of the native soils, materials placed for armouring or erosion protection should be laid on a geotextile or granular layers as discussed in Section 7.2.5 which will provide sufficient filtration.

7.3 Construction Considerations

All surficial topsoil, organic, loose, soft and otherwise deleterious materials should be stripped from areas of proposed slope reconstruction. Fill materials should be placed in maximum 300 millimetre thick loose lifts and properly benched into the existing embankments or natural slopes in accordance with Ontario Provincial Standard Drawing (OPSD) 208.010 and compacted. Upon completion of filling to the pavement subgrade level, the embankment side slopes should be trimmed to a final inclination of two horizontal to one vertical or flatter. Embankment fills greater than 6 metres in height should be provided with a minimum 2 metre wide bench at mid-height.

All excavation should be carried out in accordance with the guidelines outlined in the current Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The fill and silts beneath the groundwater level would be classified as Type 3 soils. Compact silt above the groundwater table, the clayey silt and silty clay will generally be classified as Type 2 soils.

8.0 MISCELLANEOUS

This report was prepared by Ms. Dirka U. Prout, P.Eng. under the direction of the Project Manager, Mr. Philip R. Bedell, P.Eng. This report was reviewed by Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment.

GOLDER ASSOCIATES LTD.

Dirka U. Prout, P. Eng.

Philip R. Bedell, P. Eng.
Principal

Fintan J. Heffernan, P. Eng.
Designated MTO Contact

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split spoon sampler for a distance of 300 mm (12 in.)

(b) Cohesive Soils

Consistency

	c_u, s_u	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	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)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_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
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
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 $= (\text{compressive strength})/2$
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

PROJECT 051130139
G.W.P. 30-00-00 LOCATION N 4780879.4 ; E 246722.4 ORIGINATED BY DJM
DIST HWY 403 BOREHOLE TYPE MANUAL DRILLING COMPILED BY WDF
DATUM GEODETIC DATE November 10, 2005 CHECKED BY

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								WATER CONTENT (%)		
								20	40	60						80	100	10
204.51	GROUND SURFACE																	
0.00	CLAYEY SILT, with silt layers Soft Brown and Grey		1	SS	3		204								0 1 69 30			
203.29	SILT, trace sand, some clay Very Loose Grey		2	SS	2		203								0 0 88 12			
202.83	SILTY CLAY, with silt layers Soft to Stiff Grey						202								0 2 38 60			
201.46	END OF BOREHOLE		3	SS	3													
3.05	Ground water encountered at elev. 204.36m during drilling Nov. 10, 2005.																	

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>051130139</u>		RECORD OF BOREHOLE No 2		1 OF 1		METRIC	
G.W.P. <u>30-00-00</u>		LOCATION <u>N 4780882.8 ;E 246715.0</u>		ORIGINATED BY <u>DJM</u>			
DIST <u> </u> HWY <u>403</u>		BOREHOLE TYPE <u>MANUAL DRILLING</u>		COMPILED BY <u>WDF</u>			
DATUM <u>GEODETIC</u>		DATE <u>November 10, 2005</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	w _p	w		w _L			
205.07 0.00	GROUND SURFACE CLAYEY SILT, with silt layers Soft to Stiff Grey					▽													
			1	SS	2		204												
			2	SS	3		203												
			3	SS	4														
202.02 3.05	END OF BOREHOLE Ground water encountered at elev. 204.61m during drilling Nov. 10, 2005.																		

+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT <u>051130139</u>		RECORD OF BOREHOLE No 3		1 OF 1		METRIC	
G.W.P. <u>30-00-00</u>		LOCATION <u>N 4780868.1 ; E 246733.7</u>		ORIGINATED BY <u>DJM</u>			
DIST <u> </u> HWY <u>403</u>		BOREHOLE TYPE <u>MANUAL DRILLING</u>		COMPILED BY <u>WDF</u>			
DATUM <u>GEODETIC</u>		DATE <u>November 10, 2005</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED	● QUICK TRIAXIAL	+	×	FIELD VANE	LAB VANE	w _p	w		w _L				
204.31	GROUND SURFACE																				
0.00	SILTY CLAY, with silt layers Soft to Stiff Grey		1	SS	3																
			2	SS	2																
201.26	END OF BOREHOLE		3	SS	3																
3.05	Ground water encountered at elev. 204.16m during drilling Nov. 10, 2005.																				

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

1 OF 2

METRIC

PROJECT 051130139

G.W.P. 30-00-00

LOCATION N 4780908.3 ; E 246744.1

ORIGINATED BY DJM/LK

DIST HWY 403

BOREHOLE TYPE POWER AUGER

COMPILED BY WDF

DATUM GEODETIC

DATE December 22, 2005

CHECKED BY

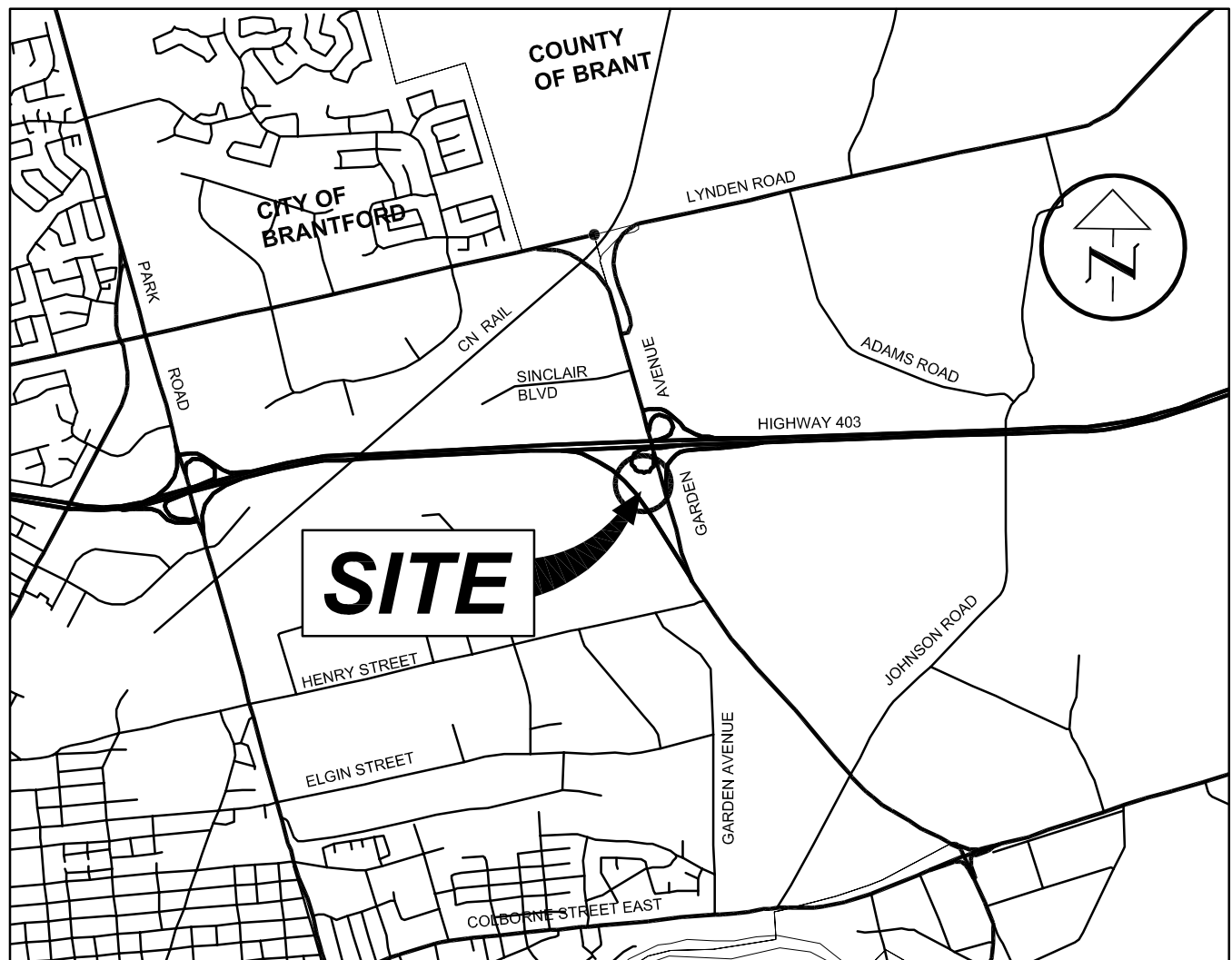
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p W W _L			
212.16	GROUND SURFACE							20 40 60 80 100		10 20 30			
0.09	ASPHALT FILL, Granular roadbase						212	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					
211.61								20 40 60 80 100					
0.55	SILT, trace clay, trace sand, with silty clay layers Compact Brown		1	SS	17		211						
			2	SS	16		210						
			3	SS	14		209						
209.26													
2.90	CLAYEY SILT, with silt and silty clay layers Stiff to Very Stiff Grey		4	SS	5		208						
			5	SS	5		207						
			6	SS	4		206						
							205						
			7	SS	10		204						
							203						
							202						
			8	SS	4		201						
							200						
			9	SS	5		199						
							198						
			10	SS	6								
200.27													
11.89	SILT, trace to some clay, with silty clay layers Loose Grey		11	SS	8								
199.21													
12.95	CLAYEY SILT, with silt and silty clay layers Stiff to Very Stiff Grey		12	SS	7								

Continued Next Page


+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

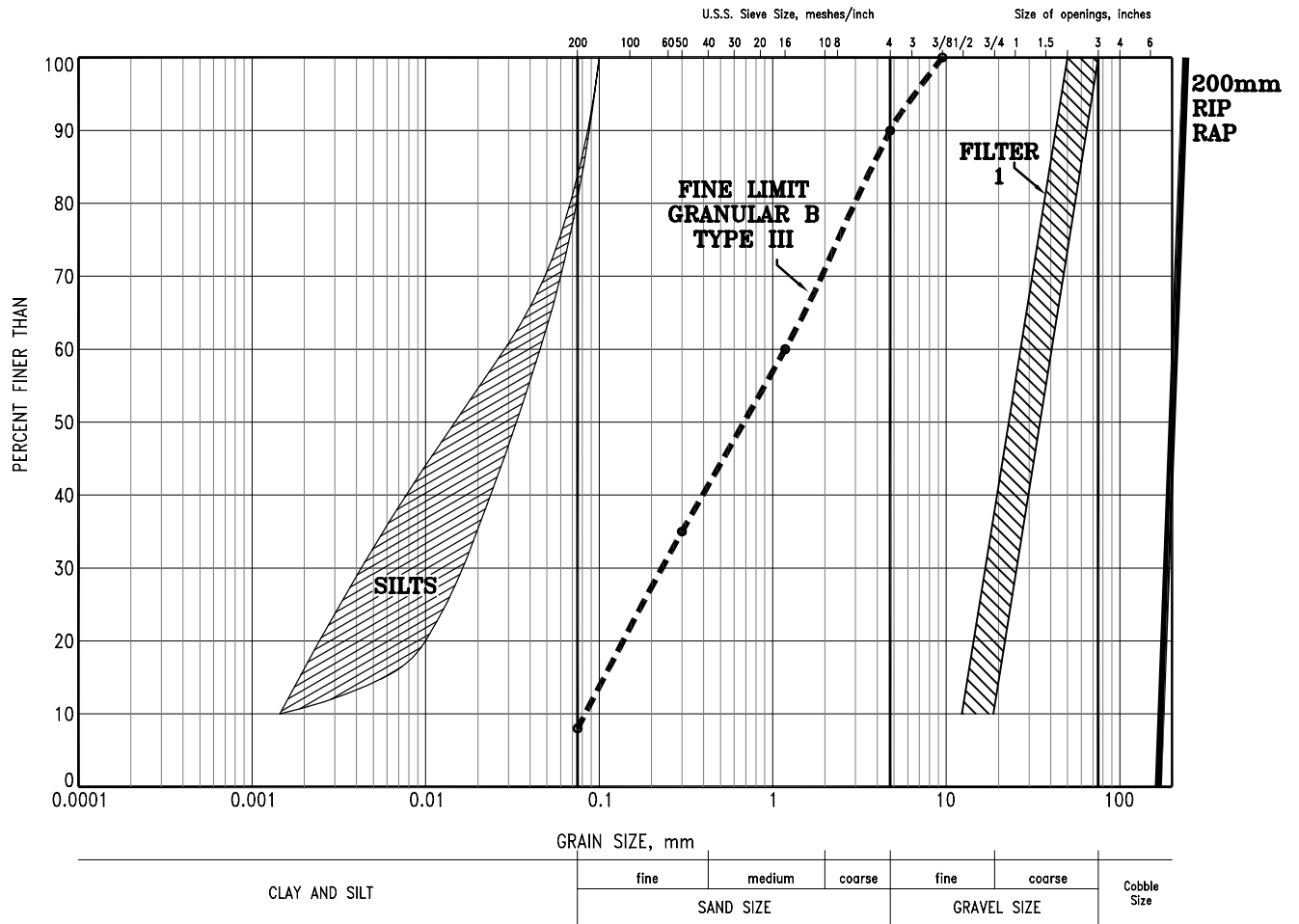
PROJECT <u>051130139</u>		RECORD OF BOREHOLE No 4		2 OF 2		METRIC	
G.W.P. <u>30-00-00</u>		LOCATION <u>N 4780908.3 ; E 246744.1</u>		ORIGINATED BY <u>DJM/LK</u>			
DIST <u> </u> HWY <u>403</u>		BOREHOLE TYPE <u>POWER AUGER</u>		COMPILED BY <u>WDF</u>			
DATUM <u>GEODETIC</u>		DATE <u>December 22, 2005</u>		CHECKED BY <u> </u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT			LIQUID LIMIT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					w _p	w	w _L	GR	SA	SI	CL
								20	40	60	80	100	○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL									
196.31			13	SS	7		197																	
15.85	END OF BOREHOLE																							
	Ground water encountered at elev. 209.11m during drilling Dec. 22, 2005.																							
	Water level measured in standpipe at elev. 207.37m Jan. 09, 2006.																							
	Water level measured in piezometer at elev. 207.86m Jan. 09, 2006.																							



SCALE
1 0 1 km

PROJECT		SLOPE FAILURE GARDEN AVENUE / HIGHWAY 403 WP 30-00-00			
TITLE		KEY PLAN			
 Golder Associates LONDON, ONTARIO		PROJECT No. 05-1130-139		FILE No. 051130139F001	
		CADD	WDF	JAN. 11/06	SCALE AS SHOWN
		CHECK			REV. 0
FIGURE 1					



NOTE

THIS DRAWING IS APPROXIMATE ONLY AND IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING TEXT.

PROJECT		SLOPE FAILURE GARDEN AVENUE / HIGHWAY 403 WP 30-00-00			
TITLE					
GRADATION ENVELOPES					
PROJECT No.		05-1130-139		FILE No.	
CADD		WDF		051130139F002	
CHECK		FEB 08/06		SCALE AS SHOWN REV. 0	
Golder Associates LONDON, ONTARIO		FIGURE 2			

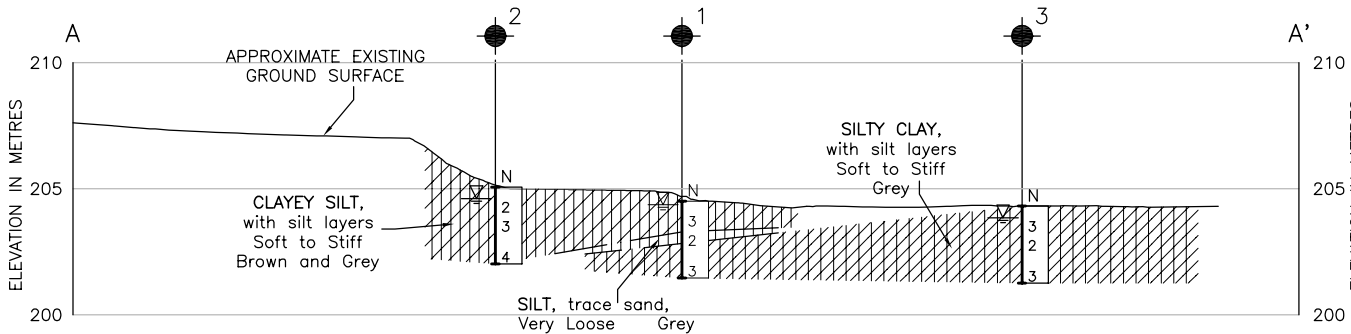
1 = 1 metric
D size dwg 22" x 34" 11" x 17" plot half scale

PLOT DATE: February 10, 2006
FILENAME: N:\Active\2005\1130 - Geotechnical\1130-105\05-1130-139 PHILIPS - GARDEN AVE FENCE - HWY 403 Drafting\Drafting - Slope Failure 05-10511301390001.dwg



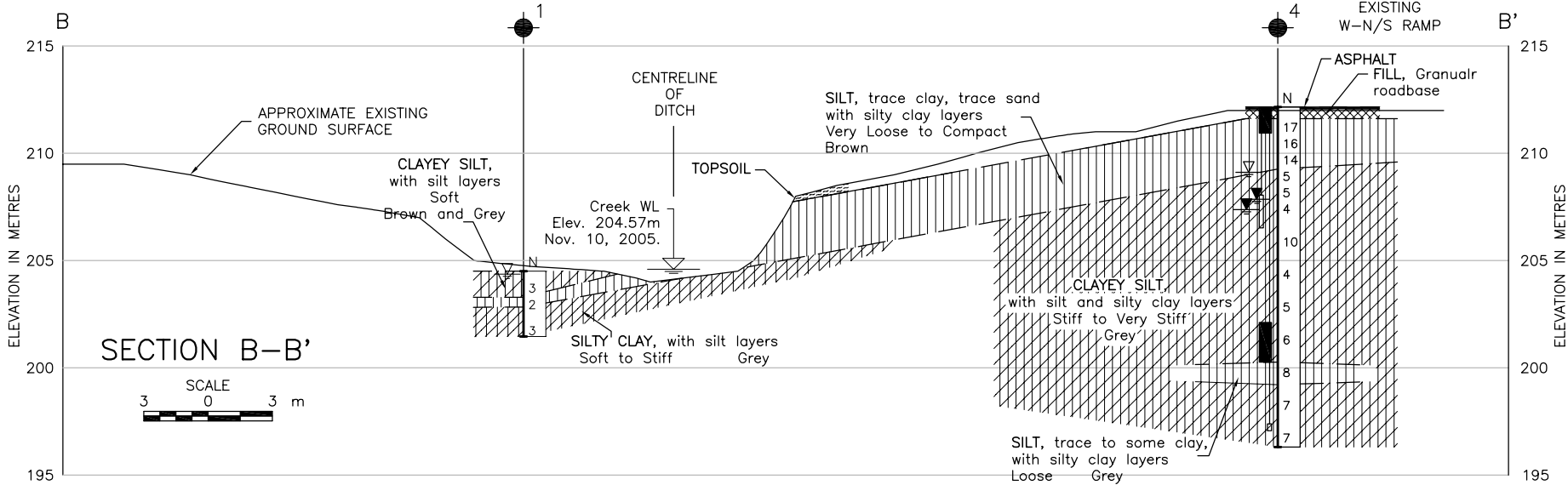
PLAN

SCALE
20 0 20 m



SECTION A-A'

SCALE
3 0 3 m



SECTION B-B'

SCALE
3 0 3 m

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST 1 HWY. 403
CONT. No.
WP No. 30-00-00

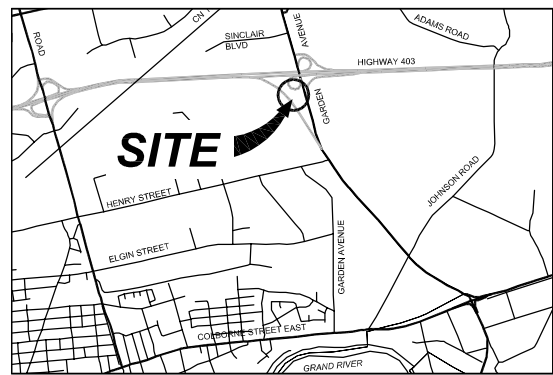


SLOPE FAILURE
GARDEN AVENUE / HIGHWAY 403
BOREHOLE LOCATIONS & SOIL STRATA

SHEET



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN

SCALE
1 0 1 km

LEGEND

- Borehole
- Borehole (Previous Investigation by Others)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL encountered during drilling
- WL in piezometer, JAN. 09, 2006.
- Failure Area (Approximate)

No.	ELEVATION (metres)	CO-ORDINATES	
		NORTH	EAST
1	204.51	4 780 879.4	246 722.4
2	205.07	4 780 882.8	246 715.0
3	204.31	4 780 867.0	246 728.7
4	212.16	4 780 908.3	246 744.1
(Previous Investigation by Others)	205.5	4 780 823.8	246 839.5

NOTES

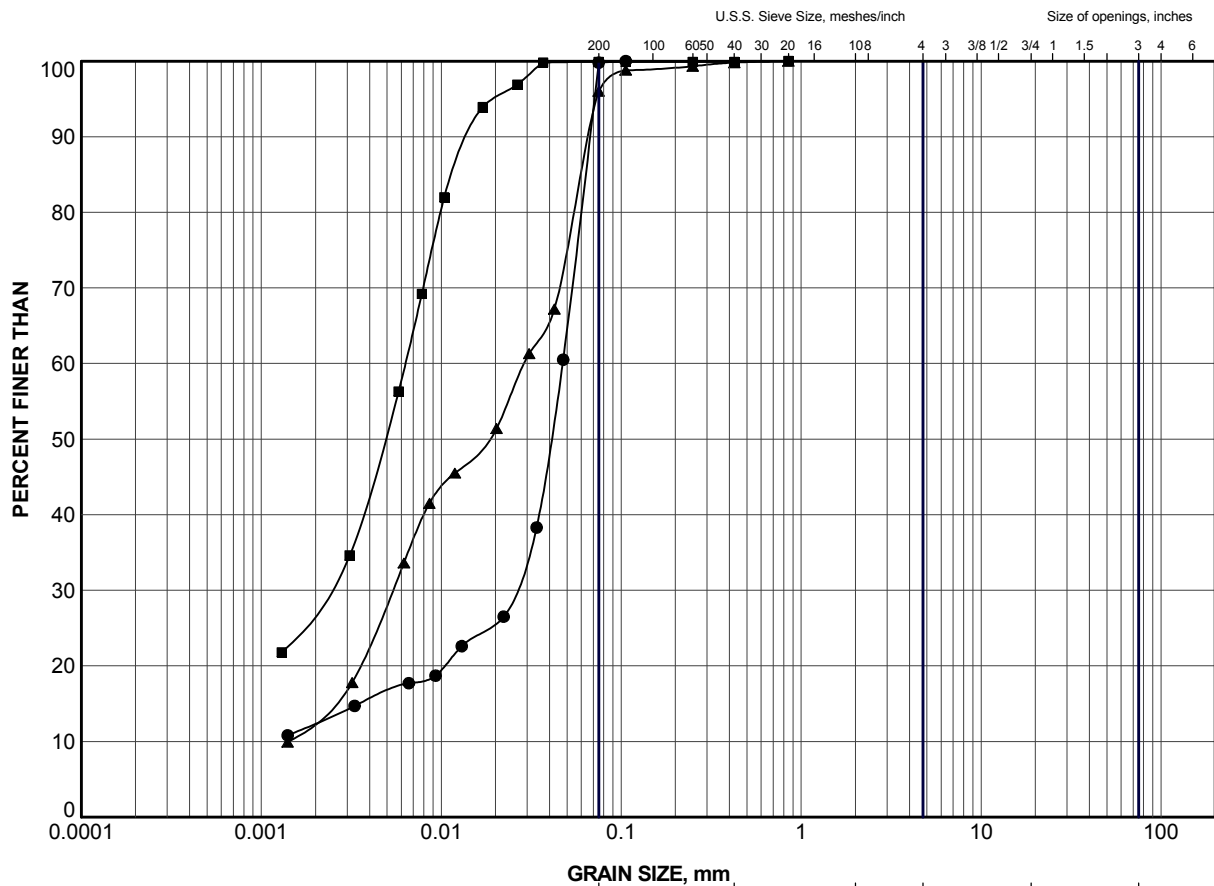
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
This drawing is for subsurface information only.

REFERENCE

REFERENCE :
DRAWING SUPPLIED BY PHILIPS ENGINEERING LTD.
ENTITLED XSURV_MTO.DWG

NO.	DATE	BY	REVISION
Geocres No.	40P1-92		
HWY. No.	403	PROJECT NO.:	05-1130-139
SUBM'D.	-	CHKD:	DATE: NOV 15/05
DRAWN:	WDF	CHKD:	APPD.
			DWG. 1


APPENDIX A
LABORATORY TEST DATA

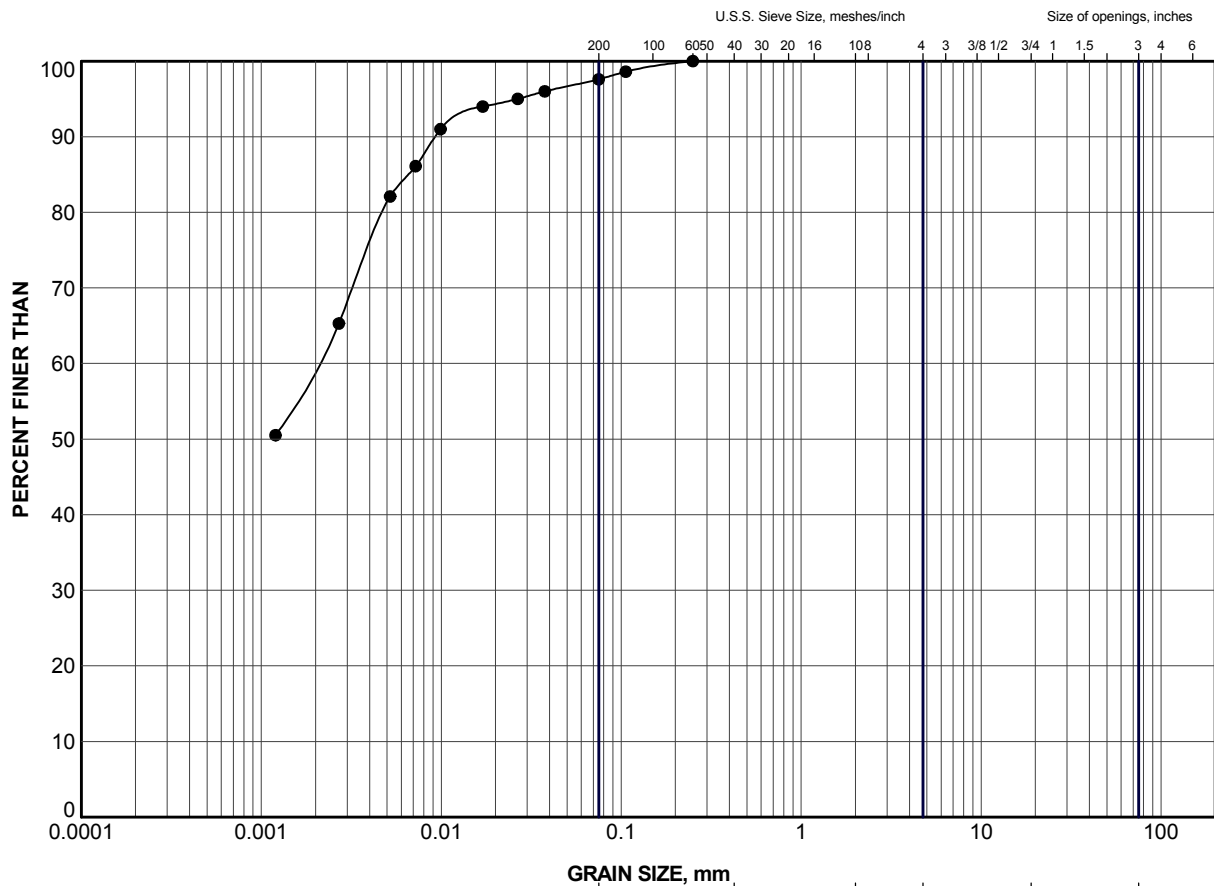


GRAVEL SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	2a	203.0
■	4	11	199.7
▲	Slope Face	1	205.6

PROJECT				SLOPE FAILURE GARDEN AVENUE/HIGHWAY 403 GWP 30-00-00			
TITLE				GRAIN SIZE DISTRIBUTION SILT (with clayey layers)			
PROJECT No.		051130139		FILE No.		051130139.GPJ	
DRAWN		WDF		SCALE		N/A	
CHECK		Jan 09/06		REV.			
 Golder Associates LONDON, ONTARIO				FIGURE A-1			

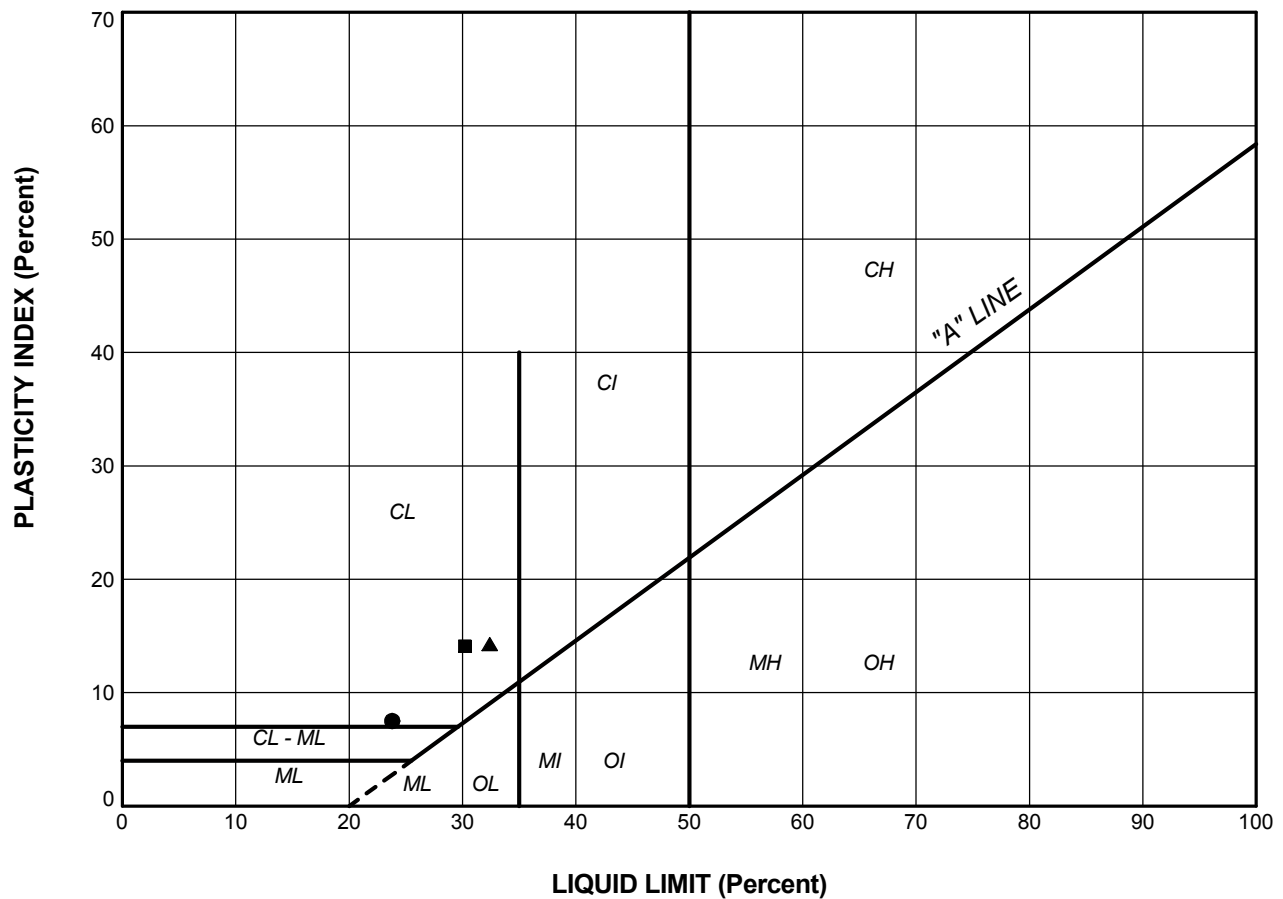



GRAVEL SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	1	2b	202.8

PROJECT				SLOPE FAILURE GARDEN AVENUE/HIGHWAY 403 GWP 30-00-00			
TITLE				GRAIN SIZE DISTRIBUTION SILTY CLAY			
PROJECT No.		051130139		FILE No.		051130139.GPJ	
DRAWN		WDF		SCALE		N/A	
CHECK				REV.			
Golder Associates LONDON, ONTARIO		Jan 09/06		FIGURE A-2			



PROJECT		SLOPE FAILURE GARDEN AVENUE/HIGHWAY 403 GWP 30-00-00	
TITLE		PLASTICITY CHART	
PROJECT No. 051130139		FILE No. 051130139.GPJ	
DRAWN	WDF	Jan 09/06	SCALE N/A
CHECK			REV.
 Golder Associates LONDON, ONTARIO			FIGURE A-4

APPENDIX B

**RECORD OF PREVIOUS BOREHOLES –
MTO GEOCRES NO. 40P1-81**



RECORD OF BOREHOLE No 4

METRIC

W P 66-67-08 LOCATION Sta. 10 + 2740 32 m Right of E Garden Avenue ORIGINATED BY DG
DIST 4 HWY 403 BOREHOLE TYPE Cone Test, Hollow Stem Auger COMPILED BY CB
DATUM Geodetic DATE 88 03 01 CHECKED BY

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 20 40 60 80 100	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
205.5	Ground Surface									
0.0										
	Silty Clay		1	SS	12					
			2	SS	13					
			3	SS	8					
	Stratified Trace of Sand		4	SS	8					
			5	SS	12					
			6	SS	7					
			7	SS	13					
	Occasional Silt Seams		8	SS	12					
			9	SS	12					
			9b	TW	PH					
			10	SS	12					
	Firm to Stiff		11	SS	9					
			12	SS	8					
			13	SS	7					
			14	SS	8					
			15	SS	12					
184.1	Trace of Gravel		16	SS	13					
22.4	Moderately Fractured		17	RC	REC					
182.8	Dolomite Bedrock			BXL	100%					
23.7	End of Borehole									

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX C
SITE PHOTOGRAPHS

SITE PHOTOGRAPHS



Photo 1 View of failure scarp below W-N/S ramp from west bank of creek. Cobbles washed downstream from original channel works are evident in left side of photo.



Photo 2 View of failure scarp looking upstream from west bank of creek.