REPORT ON FOUNDATION INVESTIGATION AND DESIGN EMBANKMENTS AND RETAINING WALLS CP RAIL SUBWAY RECONSTRUCTION HIGHWAY 401 AND STEVENSON ROAD G.W.P. 127-99-00 PURCHASE ORDER NO. 2005-A-000490 GEOCRES NO. 30M15-101

Submitted

To

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PART 1 FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION

This report presents the factual data from a foundation investigation carried out by Thurber Engineering Ltd. (Thurber) for the detailed design of embankments and retaining walls along the alignment of the proposed CP Rail Subway reconstruction in Oshawa, Ontario. This work is part of the project involving construction of a new Stevenson Road interchange at Highway 401. Thurber has been retained by McCormick Rankin Corporation (MRC) to carry out this additional scope of investigation under the Ministry of Transportation Ontario (MTO) Purchase Order No. 2005-A-000490.

The purpose of this investigation was to determine the subsurface conditions at selected locations along the proposed alignments of the embankments and retaining walls and, based on this and previously obtained data, to provide borehole location and soil strata drawings, records of boreholes, laboratory test results and a written description of the subsurface conditions.

Reference is made to the following documents in the preparation of this report.

- Thurber Engineering Ltd. report titled "Draft Report on Foundation Investigation and Design, CP Rail Subway Structure, Highway 401 and Stevenson Road, G.W.P. 127-99-00, Site 22-185, Purchase Order No. 2005-A-000490, GEOCRES No. 30M15-101, File: 19-1351-46-CP, submitted to McCormick Rankin Corporation and dated November 10, 2003 (Reference 1).

- Golder Associates Ltd. report titled "Addendum A, Preliminary Foundation Investigation and Design Report, CP Rail Bridge, Highway 401 and Stevenson Road, Oshawa, Ontario", W.P. 127-99-00, Agreement No. 2005-A-000179, The Ministry of Transportation, Central Region, 001-8033F-1, July 2002 (Reference 2).

2.0 SITE DESCRIPTION

The existing CPR bridge is located at approximately 400 m west of the existing Stevenson Road underpass bridge, within the southwestern quadrant of the City of Oshawa. Bloor Street runs parallel to Highway 401 on its south side, and Champlain Avenue runs parallel to Highway 401 on its north side. The General Motors Canada Auto Plant complex is situated to the southeast of the site. The site location is shown on Drawing 19-1351-46-CPER1.

The terrain at the site is generally flat-lying with a gentle southerly sloping trend towards Lake Ontario. Industrial and commercial buildings currently occupy the southeast and southwest quadrants, while residential buildings occupy the northeast quadrant. Fox Street, which runs in a north-south orientation, intersects the existing Champlain Avenue to the east of the existing north approach. The northwest quadrant is largely an open field with a hydro tower situated at some 30 m west of the existing north approach.

At the site, Highway 401 runs in an east-west orientation with three lanes in each direction. The existing subway structure carries the tracks of a CP Rail spur line over Highway 401, Bloor Street and Champlain Avenue. The north approach embankment to the existing CP bridge extends from its north abutment over open fields for about 400 m. The south embankment extends from the existing south abutment to beyond the grade separation with the CN tracks. The approaches near the abutments are up to about 6 m in height.

3.0 INVESTIGATION PROCEDURES

3.1 Field Investigation

Preliminary plans and sections for the proposed works were provided to Thurber as attachments to a letter dated October 8, 2003 by McCormick Rankin Corporation (MRC).

The original borehole investigation program for the new CP structure was carried out during the period of April 2 to June 11, 2003, inclusive. The current additional investigation program was carried out on November 24 and 25, 2003. The table below lists all seven boreholes from the current investigation and three selected boreholes from the previous investigation. All of these boreholes are located at or near the alignments of the proposed embankments and retaining walls covered in this report. The date of drilling and depth of each of these boreholes are listed as follows :

Borehole	Drilling Date	Depth (m)
03-62	November 24, 2003	6.3
03-63	November 24, 2003	6.7
03-64	November 24, 2003	8.1
03-65	November 24, 2003	8.2
03-66	November 24, 2003	6.7
03-67	November 25, 2003	8.1
03-68	November 25, 2003	6.4
03-27	April 15, 2003	6.4
03-27A	June 11, 2003	21.6
03-32	April 23, 2003	6.6

* Reported previously in Reference 1.

Borehole F8 from Reference 2, drilled previously by others, was located near the north abutment. This borehole is also incorporated in this report to provide relevant subsurface information.

The approximate locations of all boreholes covered in this report are shown on Drawing 19-1351-46-CPER1. The investigation was carried out using track and truck mounted drill rigs supplied and operated by specialist drilling contractors.

In these boreholes, a majority of the soil samples were obtained with a 50 mm outside diameter split spoon sampler driven in accordance with the Standard Penetration Test (SPT). Pocket penetrometer readings were obtained on selected cohesive samples for qualitative strength correlation purposes. Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in selected boreholes to permit longer term groundwater level monitoring.

The field work was supervised on a full-time basis by a member of our field staff who located the boreholes in the field with reference to survey stakes and/or markings laid out in the field by others, cleared borehole locations of underground utilities, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in appropriately labelled containers and transported back to Thurber's laboratory in Oakville for further examination and testing.

Upon completion of drilling and piezometer installation, the boreholes were appropriately backfilled and sealed, and capped with cold patch asphalt where they were located on paved surfaces. In boreholes where piezometers were installed, bentonite "holeplug" was used as seals directly above the sand filter and immediately below the ground surface.

For the current additional investigation, the as-drilled locations of Boreholes 03-62 to 03-68 coincide with the locations established by the survey stakes/markings. For the previous investigation, Boreholes 03-27, 03-27A and 03-32 were relocated due to conflicts with sloping ground and traffic conditions. The ground surface elevations and plan co-ordinates (northings and eastings) at the staked/marked locations for Boreholes 03-27, 03-27A and 03-32 have been established in the field and the survey data forwarded to Thurber by J.D. Barnes Ltd. Elevations and co-ordinates of the relocated boreholes were established by Thurber based on the survey information. Similar survey information for Boreholes 02-62 to 03-68 was provided by MRC.

Results of the field sampling and testing are presented on the Records of Boreholes in Appendix A.

3.2 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all soil samples in accordance with the current MTO standards. Grain size distribution analysis and Atterberg Limits tests were conducted on selected samples in the previous investigation. The results were reported in Reference 1 and are repeated in Appendices B and C for completeness.

4.0 GENERAL SITE GEOLOGY AND SUBSURFACE STRATIGRAPHY

4.1 General Site Geology

Based on published geological information, the general area of the project is located within the physiographic region named Iroquois Plain. This region forms part of the lakebed of the former glacial Lake Iroquois. The Iroquois Plain is covered by glacial till with a predominantly clayey silt to sandy silt matrix with numerous cobbles and boulders. More recent tills were deposited in the form of drumlins. Recent glaciolacustrine and glacialfluvial deposits occupy the depressions between the drumlins. Below the extensive till deposits lies shale bedrock of the Whitby Formation (Chapman and Putnam, "The Physiography of Southern Ontario", Third Edition, Ontario Geological Survey, 1984).

4.2 Subsurface Stratigraphy

This section presents a generalized summary of the subsurface conditions along the alignments and/or footprints of the high embankments and retaining walls. The detailed subsurface soil and groundwater conditions encountered in the relevant boreholes covered in this report are presented on the Records of Boreholes in Appendix A. Record of Borehole F8 from Reference 2 has also been included.

In general, the subsurface conditions encountered in Boreholes 03-62, 03-63, 03-66, 03-27, 03-27A, and F8 (north approach embankment areas) consist of topsoil, asphalt and/or fill overlying predominantly native sand and silt till. In Boreholes 03-32, 03-64, 03-65, 03-67 and 03-68 (south approach and embankment areas), topsoil overlies surficial silty clay and clayey silt till, which in turn overlies sand and silt till. Measured groundwater levels range from within 0.5 m depth of the existing ground surface at the north approach, north and south abutment areas, to about 4 m depth near the southerly limit of the project. Drawing No. 19-1351-46-CPER1 titled "Borehole Location Plan" illustrates the approximate locations of the boreholes covered in this report.

4.2.1 Topsoil and Asphalt

Topsoil ranging between 150 mm and 300 mm in thickness was encountered in Boreholes 03-62, 03-63, 03-64, 03-65, 03-66, 03-67 and 03-68. Topsoil thickness may vary between and beyond borehole locations.

Boreholes 03-27 and F8 encountered asphalt in the order of 25 mm in thickness.

4.2.2 Fill

Sand to sand and gravel fill was encountered below topsoil or asphalt in Boreholes 03-62, 03-63, 03-66 and F8 to approximate depths of 0.6 m to 0.7 m. Where measured, SPT 'N' values ranging between 4 blows and 8 blows per 0.3 m penetration indicate that the fill is typically loose. Measured moisture contents of the fill ranged between 13% and 18%.

4.2.3 Silty Clay and Clayey Silt Till

Deposits of native, cohesive silty clay and clayey silt till was encountered below the topsoil, fill, or at ground surface in Boreholes 03-64, 03-65, 03-67, 03-68 and 03-32. These deposits were fully penetrated in all of these boreholes, and were encountered to depths varying from 0.7 m to 2.6 m. Measured SPT 'N' values ranging between 4 blows and 26 blows per 0.3 m penetration, and correlations with pocket penetrometer test results, indicate that these cohesive deposits have a firm to very stiff consistency. Figure B1 is a plasticity chart showing results of Atterberg Limits tests carried out on a sample of clayey silt till. Measured moisture contents of these cohesive deposits ranged between 13% and 35%. Although not encountered in the boreholes, glacial till inherently contains cobbles and boulders.

4.2.4 Sand and Silt Till

Sand and silt till was encountered below the silty clay, clayey silt till, fill or asphalt in Boreholes 03-62 to 03-68, 03-27/27A, 03-32 and F8. This deposit was not fully penetrated in any of these boreholes. The SPT 'N' values measured within the main body of this till were typically greater than 50 blows for less than 0.3 m penetration indicating a very dense state, except in Boreholes 03-62 to 03-65, and F8 where the upper zones of this till was in a compact to dense state, i.e. 'N' values of between 13 and 32 blows per 0.3 m penetration. Figure C1 shows the grain size distribution curve of a sample of the sand and silt till. Measured moisture contents of the till samples typically ranged between 5% and 12%, with occasional higher values of up to 25% near the surface of this deposit. Although not encountered in the boreholes, glacial till inherently contains cobbles and boulders.

Interlayers of sand, and occasional gravelly sand, was encountered within the sand and silt till in Boreholes 03-62, 03-64, 03-65 and 03-68. Measured SPT 'N' values of these interlayers also exceed 50 blows per 0.3 m penetration, indicating a very dense state which possibly indicate the presence of cobbles and boulders. Measured moisture contents ranged between 8% and 22%.

4.2.5 Groundwater Conditions

Free water was observed in Boreholes 03-62, 03-64, 03-65, 03-67, 03-68, 03-27A and 03-32 upon completion of drilling. The remaining boreholes were dry upon completion of drilling. One piezometer was installed and sealed near the bottom of each of Boreholes 03-64, 03-67, 03-32 and F8. The depths, elevations and dates of water level readings taken in these piezometers are presented in the following table.

Borehole	Water Level Depth (m)	Water Level Elevation (m)	Date of Reading
03-64	0.9	107.2	November 27, 2003
	0.7	107.4	December 3, 2003
	0.6	107.5	December 22, 2003
03-67	3.8	101.6	November 27, 2003
	3.9	101.5	December 3, 2003
	4.1	101.3	December 22, 2003
03-32	0.6	104.7	April 24, 2003
	0.0	105.3	May 27, 2003
	0.2	105.1	June 17, 2003
F8	0.2	105.8	March 20, 2002
	0.2	105.8	April 1, 2002

It should be noted that these piezometric levels are based on short term observations and the groundwater levels are subject to seasonal fluctuations. It is also anticipated that there is a regional flow in a southerly direction towards Lake Ontario.

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PART 2 FOUNDATION DESIGN REPORT

5.0 FOUNDATION EVALUATION AND RECOMMENDATIONS

5.1 General

This section of the report presents the foundation recommendations for the design of the proposed approach embankments and associated retaining walls for the CP Subway structure. It is understood that Highway 401, Stevenson Road, Bloor Street and Champlain Avenue will be realigned as part of the interchange construction.

We understand that the proposed works will include the following :

- New fills will be placed on the east side to widen the north and south approaches to the CPR bridge. The final embankment heights will be up to 7.5 m.
- Two sections of retaining wall extending from approximately Stations 9+650 to 9+800 (about 150 m in length) at the north approach, and from Stations 10+075 to 10+150 (about 75 m in length) at the south approach. The retaining walls are up to 3 m in height.

The general layout and typical cross sections (Stations 9+900 and 10+100) of the proposed works were shown on drawings provided by MRC.

5.2 Approach Embankment Analysis and Design

The design of the approach embankments must take into account stability and settlement issues. In particular, the following issues are addressed in this report.

- Placement of new fill on native ground and on existing fill will be required as part of the widening of the existing approaches. Consideration has been given to achieving and maintaining embankment stability throughout fill construction and in the long term.

- The magnitude of total and differential settlements of foundation soils under the load of the new fills adjacent to the existing fills, and settlement of the new fill itself, have been assessed.

5.2.1 Stability Analysis Methodology

For the purpose of embankment stability assessment, a commercially available slope stability analysis program GSLOPE developed by Mitre Software Inc. was used. To assess long term stability conditions, effective stress (drained) analyses were carried out for typical embankment cross-sections using the Bishop's simplified method for stability assessment. Analyses in terms of total stresses were carried out to confirm short term stability conditions for embankment construction.

The following variables have been considered in the stability analyses :

- Fill materials – Select Subgrade Material (SSM) has been assumed in the analyses for new fill. Silt borrow should not be used for new embankment construction.
- Embankment geometry – 2H : 1V for all side slopes as long as SSM is used.
- Embankment subgrade – all surficial organics, soft and deleterious soils to be removed from the subgrade areas.
- Groundwater table – at existing ground surface.

In view of the above and based on past experience of embankment construction on similar subsurface conditions, a Factor of Safety (F.S.) of 1.3 has been selected for use in the analysis and design of new fill embankments at this site.

5.2.2 Settlement Assessment Methodology

Immediate (elastic) settlements due to compression of cohesionless foundation soils under the load of the new fills have been estimated based on elastic analysis.

Anticipated settlements due to primary consolidation, where applicable, of the foundation silty clay and clayey silt till deposits, under the load of the new fills, have been estimated based on the methods described in the CHBDC (2000) Commentary Section C6.6.3.6, which involves the use of drained and undrained moduli of deformation. Methods based on conventional one dimensional consolidation theory have also been employed, where applicable, for confirmatory purposes.

Correlation between SPT 'N' values, soil index properties and field pocket penetrometer test results, where applicable, have been used to estimate the compressibility and deformability characteristics of the foundation soils.

5.2.3 Stability Analysis Results

5.2.3.1 North Approach

It is understood that new fills will be placed on old fills to construct embankments reaching about 7.5 m in total height above existing ground surface to widen the north approach embankment on the east for the new CP structure. Where there is space restriction imposed by the property line, it is understood that retaining walls of up to 3 m in height will be required to retain the new approach fills. Foundation recommendations for design of the retaining walls are contained in a later section of this report.

Based on existing subsurface information from Boreholes 03-62, 03-63, 03-64, 03-65, 03-66, 03-27, 03-27A and F8, the subgrade of the widened embankments will range from the native, firm to very stiff clayey silt till to the compact to very dense sand and silt till. New fills are to be placed on existing fills. Emphasis was placed on carrying out stability analyses using drained conditions, given that the foundation soils are the predominantly cohesionless sand and silt till. A selected section involving surficial clayey silt till layers was analysed for undrained conditions. Figures D1 to D3 show that for a new fill height of 7.5 m, the calculated Factors of Safety for selected cases at the north approach are greater than 1.3 for embankment side slopes not steeper than 2 H : 1 V.

Figure D4 shows the stability analysis results of an assumed case where a 3 m high concrete retaining wall (assumed 1.5 m thick) was used to retain 7.5 m of new fill. A F.S. in the order of 1.3 for global stability was achieved, provided that the retaining wall was designed and constructed as recommended in this report.

Provided that the subgrade is stripped of organics, soft and deleterious soils, and is uniformly stiff/dense and competent, and the fill properly placed and compacted in accordance with the recommendations of this report, earth embankments at this location with slope inclination of 2 H to 1 V, and up to 7.5 m in height, would be stable at the end of construction and over the long term. Mid-height benches for addressing surficial stability are not required for embankments lower than 8 m in height.

5.2.3.2 South Approach

It is understood that new fills will be similarly placed on old fills to construct embankments reaching 6.5 m in total height above existing ground surface. Where there is space restriction imposed by the property line, retaining walls will be required to retain the new approach fills. Foundation recommendations for design of the retaining walls are contained in a later section of this report.

Based on existing subsurface information from Boreholes 03-67, 03-68 and 03-32, the subgrade of the widened embankment will consist of the surficial native, firm silty clay and stiff to very stiff clayey silt till, underlain by very dense sand and silt till. Stability analyses were carried out for long term (drained) and end of construction (undrained) conditions. Figures D5 and D6 show that the calculated Factors of Safety for these selected cases are greater than 1.3 for embankment side slopes not steeper than 2 H : 1 V.

Provided that the subgrade is stripped of organics, soft and deleterious soils, and is uniformly stiff/dense and competent, and the fill properly placed and compacted in accordance with the recommendations of this report, earth embankments at this location with slope inclination of 2 H to 1V, and up to 7 m in height, would be stable during construction and over the long term.

5.2.4 Settlement Analysis Results

It is understood that the existing approach fills for the CPR bridge have been in place for many years. Accordingly, it is considered reasonable to assume that any settlement of the existing fills themselves, and settlement due to consolidation of the foundation soils under the existing fills, have been completed.

Immediate (elastic) settlements of the foundation soils as well as a substantial portion of settlement of the new fill itself is expected to occur during construction, provided that the new fill is composed of Select Subgrade Materials (SSM) and compacted as per the OPSS requirements discussed later in this report.

5.2.4.1 North Approach

At the north approach where up to 6 m of new fill will be placed on the east side of the existing embankment to straddle existing fill and the adjacent native ground, immediate settlement of the predominantly cohesionless foundations soil consisting of sand and silt till, and settlement of the new fill itself are expected to be complete by the end of construction. Within the area of the widening, it is estimated that the magnitude of immediate foundation settlement by the end of construction could be in the range of 25 mm to 50 mm. It is anticipated that post construction settlement will be practically negligible.

5.2.4.2 South Approach

At the south approach where up to 5 m of new fill will be placed on the east side of the embankment to straddle existing fill and the adjacent native ground, immediate settlement of the surficial silty clay, clayey silt till and the underlying sand and silt till, as well as settlement of the new fill itself are expected to be complete by the end of construction. Within the area of the widening, it is estimated that the magnitude of immediate foundation settlement by the end of construction could be in the range of 50 mm to 75 mm. It is anticipated that post construction settlement will be practically negligible.

At both approaches, some settlement will be induced on the existing fill as new fill is placed. However, the anticipated settlement will be less than those quoted above and will decrease towards the tracks, where settlement will be limited to the magnitude permissible by Level 1 Track Protection recommended in the following Section 5.5.

5.2.5 Subgrade Preparation and Embankment Construction

In general, surface vegetation, topsoil, organic deposits and soft soils should be stripped from the subgrade areas within the plan limits of the proposed new fill. It is recommended that a minimum 150 mm depth be stripped from the footprint of the new fill (embankment widening) in all areas, except from Stations 9+800 to 9+900, and from Stations 10+100 to 10+200, where the minimum stripping depth should be 300 mm. All subgrade should be proof-rolled prior to fill placement to identify soft or otherwise disturbed areas. These soft or disturbed areas should be sub-excavated, replaced with new fill and recompacted.

Where new fill is to be placed on old fill, the existing slope surfaces should be appropriately benched, as per OPSD 208.010, after stripping of topsoil/organics and prior to placement of new fill. The benching procedures would allow adequate keying in of the new fill. However, it is not uncommon for old railway embankments to have an outer shell (immediately below the slope surface) consisting of loose or soft, saturated soils. The fill composition of the old rail embankment may also be variable along the alignment of the embankment, and may contain organics and other deleterious material. Benching into these loose and possibly saturated soils on the slope may adversely affect the stability of the existing railway embankment. Placement of new fills over these loose soft embankment fill may also adversely affect the stability of the existing railway embankment and induce additional settlements of the new fill. It is, therefore, critical to undertake the benching in a careful manner so that the stability of the existing rail embankments is not jeopardized. In this regard, the following are recommended:

- Benches should be excavated one level at a time and the compacted fill brought up before the next bench is excavated.
- Each bench height should not be more 0.3 m and the bench width should not be more than 0.6 m.
- The benching should be carried out in short lengths of 25 m along the alignment of the existing railway embankment. Each benched section should be backfilled before

excavating the next section of the bench. The purpose of this is not to have a long unsupported bench length that may adversely affect the stability of the existing rail embankment.

- Careful inspection and monitoring of benching of the old rail embankment and placement of new fill should be carried out. If there is any sign of instability of the existing rail embankment caused by either the benching or new fill placement operation, such areas must first be stabilized, the benching and fill construction procedures should subsequently be reviewed and amended as appropriate.

If the schedule permits, it is recommended that new embankment fills be placed and compacted during the early stages of construction in areas within the footprint of the proposed embankments where there is little to no existing fill. This will allow much of the settlement of the fill itself to take place before the end of construction.

Construction of the embankment above the prepared subgrade should be carried out (with reference to OPSS 206) using Select Subgrade Material (SSM) at +2% or –2% of optimum moisture content in accordance with Special Provision No. 110F13, Amendment to OPSS 1010, March 1993. The embankment side slopes should not be steeper than 2 H : 1 V for fills constructed with SSM at +2% or –2% of optimum moisture content. Silt and clayey soils should not be used as new fill. Use of such soils will require flatter side slopes to maintain embankment stability and will result in additional settlement of the fill.

All new embankment fill should be placed in regular lifts and be compacted in accordance with OPSS 501, except that the degree of compaction should be at least 98 per cent of the material's Standard Proctor maximum dry density, at its optimum moisture content, to minimize fill settlement.

Vegetation cover should be established on all exposed embankment slopes to protect the fill against surficial erosion. Reference may be made to SP 572S01 for more detailed requirements.

5.3 Retaining Walls and Toe Walls

It is understood that retaining walls are proposed along three sections at the east property line, between the CP right-of-way and the adjacent fields.

At the north approach, the proposed north retaining wall alignment extends approximately from Stations 9+650 to 9+800 (150 m in length). At the northwest quadrant of Fox Street and Champlain Avenue, a proposed retaining wall extends approximately from Stations 9+890 to 9+935 (45 m in length).

At the south approach, the proposed retaining wall alignment extends approximately from Stations 10+075 (near proposed south bridge abutment) to 10+150 (75 m in length).

Consideration may be given to using conventional cantilevered type gravity walls and/or concrete toe walls. Design of the retaining walls may be carried out in general accordance with the AREMA code or the CHBDC (2000). Design of the retaining walls should include checking for resistance against sliding, overturning and global stability.

5.3.1 Footings

It is recommended that the wall footings be founded at a minimum 1.2 m below the final grade. Based on Boreholes 03-64 and 03-65, the footing subgrade along the north wall alignment consists of very stiff clayey silt till, or compact to very dense sand and silt till. Based on Boreholes 03-32 and 03-67, the footing subgrade along the south wall alignment consists of very stiff clayey silt till, underlain by the very dense sand and silt till. The footings should not be founded on the surficial silty clay layer.

At the north wall alignment, the footings may have to be stepped down in a north to south direction in order to accommodate the sloping ground surface. It is recommended that the footings be founded at or below Elevation 106.8 m near the northerly limit and at or below Elevation 105.8 m near the southerly limit of the wall. At the south wall alignment, it is recommended that the footings be founded at or below Elevation 104.0 m.

For working stress design (AREMA code), it is recommended that footings founded on the very stiff clayey silt till or the upper, dense portion of the sand and silt till, at the elevations recommended above, be designed assuming an allowable bearing capacity of 200 kPa. For limits states design (CHBDC), footings founded at the same elevations may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa and a geotechnical resistance at Serviceability Limit States (SLS) of 200 kPa.

Should higher bearing values be required for design, the footings may be founded at lower elevations. At the north alignment, footings founded on the very dense sand and silt till, at or below Elevations 105.6 m (north limit) and 104.8 m (south limit), may be designed using an allowable bearing pressure of 400 kPa (working stress design), or a factored geotechnical resistance at ULS of 750 kPa and a geotechnical resistance at SLS of 400 kPa (limits states design). At the south wall alignment, the same bearing capacities may be used for footings founded on the very dense sand and silt till at or below Elevation 102.5 m near the north limit and Elevation 103.0 m near the south limit.

Along the Fox Street wall alignment, it is recommended that footings founded on the dense sand and silt till at about Elevation 105.5 m near the north limit, and at about Elevation 104 m near the south limit, be designed using an allowable bearing capacity of 200 kPa (AREMA code). For limits states design (CHBDC), footings founded at the same elevations may be designed for a factored geotechnical resistance at ULS of 300 kPa and a geotechnical resistance at SLS of 200kPa. For deeper founding elevations, an allowable bearing capacity of 400 kPa (working stress) and factored ULS and SLS values of 750 kPa and 400 kPa, respectively, may be used.

The above values are for vertical concentric loads only. Effects of load inclination and eccentricity need to be taken into account as per the AREMA code or the CHBDC. The design of stepped footings should also be in accordance with the requirements in the AREMA code or the CHBDC (2000).

Resistance to lateral forces/sliding resistance between the concrete footings and undisturbed clayey silt till subgrade should be calculated in accordance with the AREMA code or the CHBDC

2000 assuming an ultimate (or unfactored) coefficient of friction of 0.45. For a sand and silt till subgrade, an ultimate (or unfactored) coefficient of friction of 0.55 may be used.

For frost protection purposes, it is recommended that a minimum earth cover of 1.2 m, or its thermal equivalent, be provided to all footings.

The underside of the footings may be up to 1 m to 2 m below the groundwater level. The clayey silt till, or sand and silt till, contains fines that will temporarily impede water seepage during construction. However, the footing subgrade must be properly prepared as described below to avoid prolonged exposure. Where water-bearing, less dense, cohesionless interlayers are exposed at subgrade level, localized groundwater control measures may be required.

Once the desired founding subgrade level is reached, careful inspection should be carried out to delineate any loose/softened or otherwise disturbed areas. Such areas should be sub-excavated down to very stiff or dense to very dense native soils, and the sub-excavation backfilled with mass concrete. It is recommended that a working mat of lean mix concrete of at least 150 mm thick be placed on the prepared and approved competent subgrade to provide protection from deterioration due to ponding water and construction traffic. The clayey silt till is prone to softening upon exposure to water. The sand and silt till is a low to non-plastic deposit that is susceptible to disturbance and loss of bearing support if exposed to water seepage or construction traffic.

5.3.2 Lateral Earth Pressures

The retaining walls may be designed in general accordance with the AREMA code or the CHBDC. Select free-draining granular fill meeting the specifications of OPSS Granular A or Granular B, Type I (modified) (Special Provision No. 110F13, 2002, Amendment to OPSS 1010, March 1993) should be used as backfill behind the walls. It is recommended that the fill be placed in accordance with OPSS 501.

If the wall support allows lateral yielding of the wall stem (unrestrained structure), active earth pressures may be used in design. If the wall does not allow lateral yielding (restrained structure), at-rest earth pressures should be assumed for design. The following table lists the unfactored parameters recommended for design.

Conditions Behind Wall	**Earth Pressure Coefficient (K)**					
	OPSS Granular A $\phi = 35°$; $\gamma = 22\ kN/m^3$		OPSS Granular B, Type I And Existing Fill $\phi = 30°$; $\gamma = 21\ kN/m^3$		Native Clayey Silt Till and Silty Clay $\phi = 30°$; $\gamma = 19\ kN/m^3$	
	Horizontal Ground Behind Wall	Ground Sloping at 2H : 1V Behind Wall	Horizontal Ground Behind Wall	Ground Sloping at 2H : 1V Behind Wall	Horizontal Ground Behind Wall	Ground Sloping at 2H : 1V Behind Wall
"Active" Coefficient, K_a	0.27	0.40	0.33	0.54	0.33	0.54
"At-Rest" Coefficient, K_0	0.43	0.62	0.50	0.76	0.50	0.76

Note: The earth pressure coefficients in the table above do not include potential compaction effects which must be included in the design.

Compaction effects may be considered in the design as per the AREMA code. For any component designed in accordance with the CHBDC (2000), additional lateral pressures shall be included as per Sections 6.9.3 and C6.9.3 to account for compaction effects. Heavy compaction equipment should not be used adjacent to the walls.

Perforated sub-drains and weep holes should be installed, where applicable, to provide positive drainage of the granular backfill behind the retaining walls. The sub-drains may consist of 150mm diameter perforated PVC pipes surrounded by clear stone and wrapped in geotextile filter cloth (e.g. Terrafix 270R or equivalent). Flexible perforated pipes wrapped in filter socks may also be used. Other aspects of the abutment granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD-3501.00.

5.3.3 Anchored Wall

Where necessary, consideration could be given to using an anchored wall. For design of an anchored/tied back wall, the lateral pressure distribution shown on Figure E1 in Appendix E may be used. This diagram may be used in conjunction with the recommended geotechnical parameters as follows.

γ	=	20 kN/m^3
γ'	=	10 kN/m^3
K_a	=	0.33 (free draining granular backfill)
K_p	=	3.0 (native soils)
h_w	=	assume groundwater level at existing ground surface

It is recommended that the bond length of the soil anchors be formed within the native stiff clayey silt till or the dense sand and silt till below the base of the embankment. The free stressing length should not be less than 4.5 m. Typical bond lengths of soil anchors range between 5m and 8 m, and nominal drill hole diameters range between 150 mm and 200 mm. For low pressure gravity grouted anchors with the bond (fixed) length formed immediately below the embankment base, an allowable soil to grout bond stress of 50 kPa (F.S. = 2) may be assumed. In order to achieve higher anchor capacity, post grouting of the bond length under higher pressures may be carried out to increase the allowable soil to grout bond stress. For design purposes, an allowable soil to grout bond stress up to 75 kPa (F.S. = 2) may be assumed for post grouted anchors.

The design unconfined compressive strength of the grout should not be less than 30 MPa. No tendon shall be stressed at any time beyond 80% of the specified minimum tendon strength (F_{pu}).

The allowable geotechnical anchor capacity, P, may be estimated by the following expression :

$$P = \tau \cdot A_s \cdot L$$

where τ = allowable soil to grout bond stress, kPa
A_s = surface area per metre of bond length, m^2/m
L = bond length, m

The above information provides means to estimate the anchor capacity for design purposes only. It is recommended that selected anchors be performance tested and all remaining production anchors on site be proof tested to confirm their carrying capacities. Double corrosion protection should be provided for all permanent anchors. Corrosion protection is normally not required for temporary anchors. In addition, it is recommended that vertical and lateral ground movement be monitored during pressure grouting. Should anchors be selected for use at this site, provisions should be incorporated into the contract to temporarily terminate grouting and to have the situation assessed if excessive movement is observed. All anchors should be installed in cased holes. Anchor testing, corrosion protection, pressure grouting and other relevant details should be in accordance with applicable guidelines such as those recommended in OPSS 942 (November 2003) "Construction Specification for Prestressed Soil and Rock Anchors" and the Post-Tensioning Institute (1996) "Recommendations for Prestressed Rock and Soil Anchors".

5.4 Retained Soils Systems (RSS) Walls

Consideration could be given to using RSS walls as retaining walls. The founding elevations of the RSS walls are governed, in part, by the existing grade and slope configurations. The RSS walls may be in the form of a rectangular, reinforced block extending from shallow depths below ground surface to the full retained height.

Design for internal stability of an RSS wall should be carried out by the proprietary designer/supplier.

Prior to construction, all topsoil, organics, soft or loose soils should be removed from the subgrade below the RSS wall. All subgrade should be proof-rolled prior to fill placement to identify soft or otherwise disturbed areas. These soft or disturbed areas should be sub-excavated, replaced with new fill and recompacted. The RSS wall should be founded on prepared, native typically stiff silty clay to clayey silt till subgrade, or on dense sand and silt till subgrade .

It is recommended that the following values be used for foundation design of an RSS wall:

- Factored geotechnical resistance at ULS of 300 kPa, and geotechnical resistance at SLS of 200 kPa, for RSS block founded on undisturbed, stiff clayey silt till or dense sand and silt till. Along the north wall, it is recommended that the design founding level be at or below approximate Elevation 107.3 m near the north limit, and at or below approximate

Elevation 106.4 m near the south limit. Along the Fox Street wall, it is recommended that the design founding level be at or below Elevation 104.5 m. Along the south wall, it is recommended that the design founding level be at or below Elevation 104.5 m.

A properly designed and constructed RSS wall up to about 5 m high founded on undisturbed, native subgrade described above will be stable against localized bearing failure. Such a wall is also stable against global, rotational type failure. The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning.

Resistance to lateral forces / sliding resistance between the RSS mass and the undisturbed native soils should be taken into account using the following values:

- Ultimate coefficient of friction between cast in-situ concrete levelling pad on Granular A is 0.7.
- Ultimate coefficient of friction of between RSS mass and native stiff or dense soils is 0.5.

The actual design must be checked for global stability prior to finalization.

The contract documents should include information on the longitudinal alignments of the wall in plan, top and base elevations of the wall in profile, cross-sectional space constraints, and an NSSP for the RSS wall. The RSS wall should also be specified to have high performance and high appearance.

5.5 Excavation and Groundwater Control

Excavation will be required for retaining wall footing construction. All excavations will likely extend to 1 m to 2 m below the groundwater table at this site. In order not to have adverse effects on the global stability of the existing railway embankments, it is recommended that footing construction be carried out in short sections of, say, 5 m to 10 m in length. Consideration may be given to staggering these lengths of footing construction such that any shoring system and/or open cut slopes associated with a section would not intersect any features of another section.

Excavation will also be required for benching into the existing embankment in order to key in the new fill. Care must be taken during the excavating and backfilling processes, especially at the toe of the slopes, to maintain global stability of the embankments at all times. As discussed earlier, the excavating and backfilling procedures for benching should be carried out in short sections of, say, maximum 25 m in length, and in 0.3 m heights at a time.

It is anticipated that temporary shoring will be required to retain the existing fill during construction. An item titled "Track Protection" as per SP 539S01 will have to be included in the contract documents. It is recommended that performance Level 1 as per Clause 539.04.02.01 be specified for this site. Open cutting may be possible, such as along the east side of the excavations, provided that adequate groundwater and surface water control measures are implemented. Temporary cut slopes may generally be formed through native soils above the groundwater level with inclinations not steeper than 1 H : 1 V. Flatter inclinations may be required at depths below the groundwater level, where the exposed soils on the slope face are loose or soft, and where water seepage occurs.

All excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and safety Act (OHSA), its regulations and other applicable regulations. For the purposes of assessing slope inclination and excavation support requirements in compliance with OHSA, the following soil types would apply to the subsurface stratigraphy encountered at the borehole locations :

Existing Embankment Fill (above groundwater)	Type 3
Silty Clay, and Clayey Silt Till	Type 2
Sand and Silt Till (above groundwater)	Type 2
Sand and Silt Till (below groundwater)	Type 3

Excavation equipment should be appropriate for excavating the very stiff clayey silt till and the very dense sand and silt till. Clauses should be included in the contract documents alerting the contractor that the tills are expected to contain cobbles and/or boulders.

All wall footings must be constructed in the dry. Pumping from properly filtered sumps may be suitable for controlling perched water from the fill and the surficial cohesive soils. Some form of groundwater control measures may be required to prevent potential "boiling" of the excavation base where it is formed in the sand and silt till. Surface run-off should be diverted away from any excavation at all times.

Decisions regarding shoring methods, groundwater control and construction sequencing should be made by the contractor. Any required shoring system must be designed by a licensed Professional Engineer experienced in such designs, whereas any dewatering system should be designed by specialists experienced in such designs.

5.6 Construction Concerns

Concerns during construction of the new embankments and retaining walls are primarily related to maintaining stability of the existing railway embankments at all times during excavation, benching and new fill placement. Track protection (temporary shoring) may be required at some locations. Particular attention should also be paid to controlling the groundwater and surface water during footing construction.

It is recommended that survey monitoring of the existing embankments be carried out during construction of the embankment widenings. Should any evidence of settlement and lateral movement of the existing embankment be identified, modification to construction procedures and/or remedial measures may be necessary.

5.7 Construction Inspection and Testing

Subgrade inspection and field density testing should be carried out by qualified geotechnical personnel during all excavation and fill placement operations to ensure that the foundation recommendations are correctly implemented and material specifications are met.

All slopes should be inspected after construction for surficial instabilities. Where necessary, remedial measures such as re-vegetation and/or placement of gravel sheeting should be implemented.

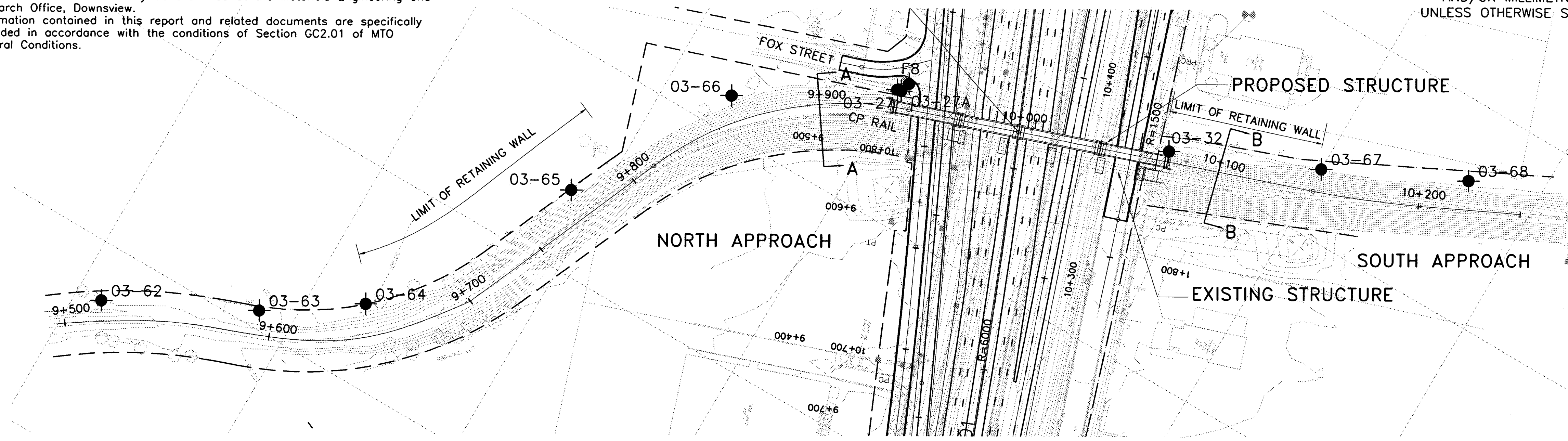
Engineering Analysis and Report Preparation by:
Sydney Pang, P.Eng.,
Senior Geotechnical Engineer

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

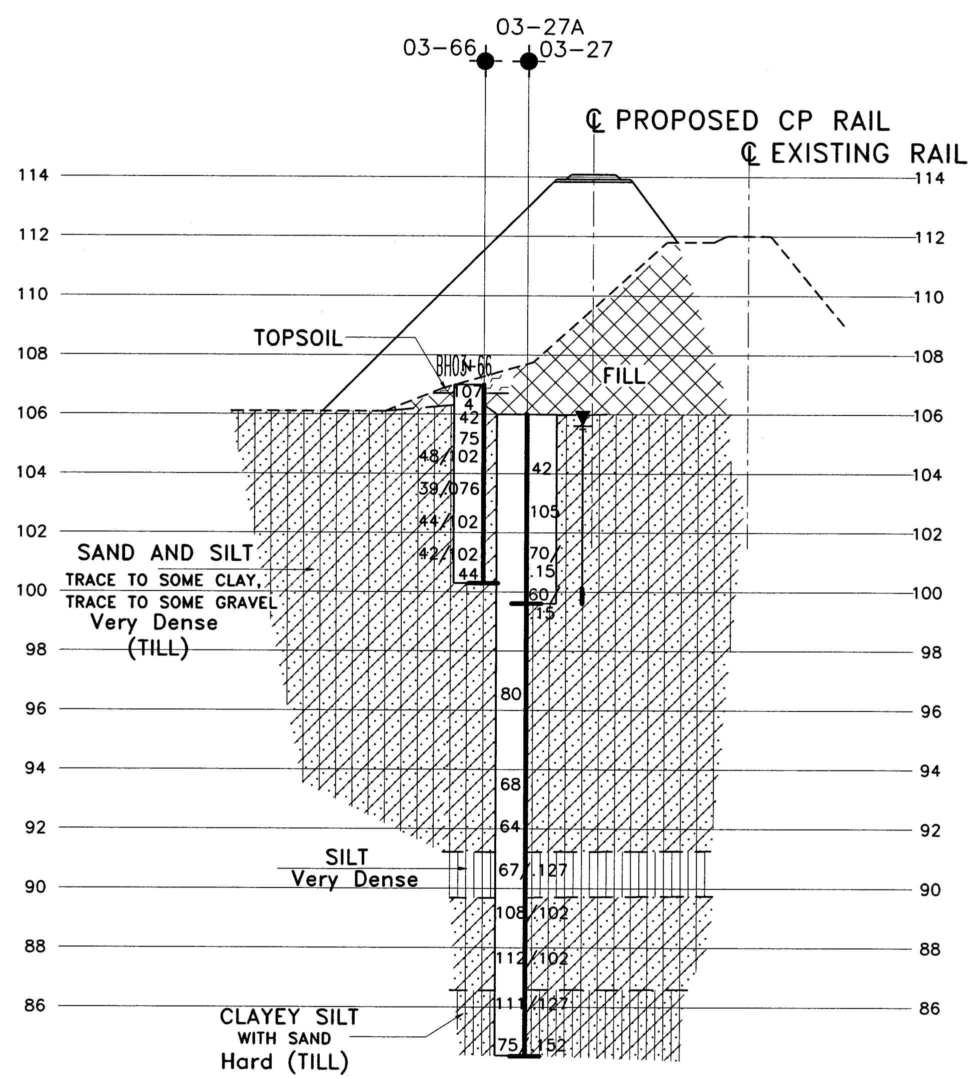
NOTE:
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview.
Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC2.01 of MTO General Conditions.

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

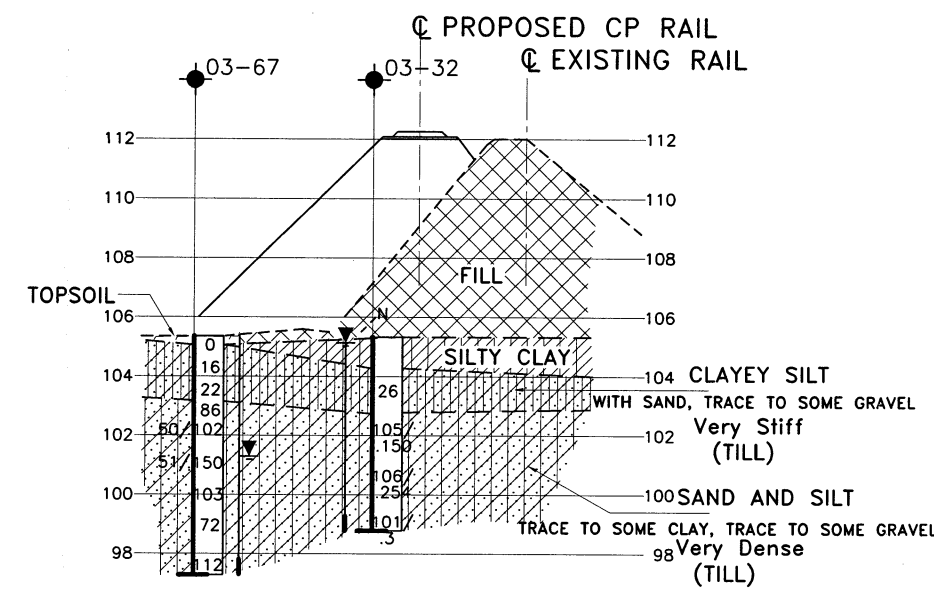
HIGHWAY 401	
GWP No. 127-99-00	
HWY. 401/ STEVENSON ROAD CP Embankments and Retaining Walls	SHEET
BOREHOLE LOCATIONS AND SOIL STRATA	
McCORMICK RANKIN CORPORATION	
THURBER ENGINEERING LTD.	



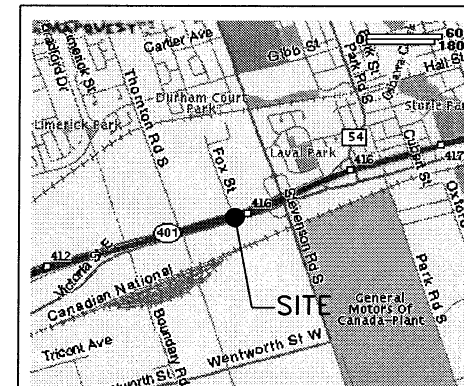
PLAN



SECTION A-A



SECTION B-B



KEYPLAN

LEGEND

Symbol	Description
	Bore Hole
	Dynamic Cone penetration Test (cone)
	Bore Hole & Cone
N	Blow/ 0.3m (std pen Test, 475J/blow)
CONE	Blows/ 0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	WL at time of investigation (as noted)
	Head Artesian Water
	Piezometer

NO	ELEVATION	NORTHING	EASTING
03-62	110.0	4859987.1	354273.9
03-63	109.1	4859917.3	354308.7
03-64	108.1	4859873.3	354338.2
03-65	107.2	4859814.1	354437.4
03-66	107.0	4859768.8	354517.4
03-67	105.4	4859500.0	354632.4
03-68	105.4	4859434.6	354663.9
03-27	106.0	4859700.0	354560.7
03-27A	106.0	4859698.0	354555.0
03-32	105.3	4859569.2	354602.2
F8	106.0	4859696.9	354566.0

— NOTE —
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS			
Jul/04	SKP	FINAL	
Oct/03	SKP	ISSUED AS DRAFT FOR REVIEW	
		DESCRIPTION	

DESIGN SKP	CHK PKC	CODE CHBDC 2000	LOAD	DATE JUL.2004
DRAWN SS	CHK SKP	SITE	STRUCT / SCHEME	DWG CPER1

APPENDIX A

Records of Boreholes

RECORD OF BOREHOLE No 03-62

METRIC

W.P. 19-1351-46 LOCATION N 4 859 987.1 E 354 273.9 ORIGINATED BY GA

DIST HWY 401 BOREHOLE TYPE 108mm Solid Stem Augers COMPILED BY SS

DATUM Geodetic DATE 24.11.03 - 24.11.03 CHECKED BY SKP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
110.0					
109.8 / 0.0	**TOPSOIL** (175mm)				
0.2	**SAND**, occ. silt, occ. rootlets, Loose, Brown, Moist (FILL) (SW)	1	SS	7	
109.3 / 0.7	**SAND and SILT**, trace to some gravel, trace clay, occ. iron oxide staining, Compact to Very Dense, Brown, Moist (TILL) (ML-nonplastic)	2	SS	15	109
		3	SS	48	108
		4	SS	59	107
		5	SS	102	
106.3 / 3.7	**SAND**, fine grained, Very Dense, Grey, Wet (SW)	6	SS	65	106
		7	SS	72	105
	possible cobbles from 5.49m to 5.79m				104
103.9 / 6.1	**SAND and SILT**, trace gravel, Very Dense, Grey, Moist (TILL)	8	SS	106/.150	
103.7 / 6.3	END OF BOREHOLE AT 6.25m. BOREHOLE OPEN TO 5.18m. WATER LEVEL IN OPEN BOREHOLE AT 4.9m DEPTH UPON COMPLETION. BOREHOLE BACKFILLED AND SEALED WITH DRILL CUTTINGS AND BENTONITE.				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × 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/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity   20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

RECORD OF BOREHOLE No 03-63

METRIC

W.P. 19-1351-46 LOCATION N 4 859 917.3 E 354 308.7 ORIGINATED BY GA

DIST HWY 401 BOREHOLE TYPE 108mm Solid Stem Augers COMPILED BY SS

DATUM Geodetic DATE 24.11.03 - 24.11.03 CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
109.1						
109.0 / 0.0	**TOPSOIL** (150mm)				109	
0.2	**SAND**, occ. silt, occ. rootlets, Loose, Brown, (FILL) (SM)	1	SS	8		○
108.4 / 0.7	**SAND and SILT**, trace to some gravel, trace clay, occ. iron oxide staining, Dense to Very Dense, Brown, Moist to Dry, (TILL) (ML-nonplastic)	2	SS	30	108	○
		3	SS	50		○
		4	SS	89	107	○
		5	SS	115	106	○
		6	SS	60/ .102	105	○
		7	SS	50/ .102	104	○
		8	SS		103	○
102.4 / 6.7	END OF BOREHOLE AT 6.71m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE BACKFILLED AND SEALED WITH DRILL CUTTINGS AND BENTONITE.					

Column headings: 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 × 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/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ONTMT4 5146-A.GPJ 07/01/04

$+^3$, $\times^3$: Numbers refer to Sensitivity   15-20-5-10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 03-64

METRIC

W.P. 19-1351-46	LOCATION N 4 859 873.3 E 354 338.2	ORIGINATED BY GA
DIST HWY 401	BOREHOLE TYPE 108mm Solid Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 24.11.03 - 24.11.03	CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
108.1					108
108.0 0.2	**TOPSOIL** (150mm)				
107.4 0.7	Silty **CLAY**, occ. sand, occ. rootlets, Firm, Brown, Moist (CL)	1	SS	4	
	SAND and SILT, trace to some gravel, trace clay, occ. iron oxide staining, Compact to Very Dense, Brown, Moist (TILL) (ML-nonplastic)	2	SS	20	107
		3	SS	33	106
		4	SS	60/ .127	
	becoming grey	5	SS	101	105
		6	SS	55/ .102	104
		7	SS	101/ .150	103
	possible cobbles from 5.49m to 5.79m				102
		8	SS	40/ .076	
100.9 7.2	**SAND**, fine to medium grained, trace silt, Very Dense, Grey, Wet (SM)				101
100.0 8.1		9	SS	104	100
	END OF BOREHOLE AT 8.08m. BOREHOLE OPEN TO BOTTOM UPON COMPLETION. WATER LEVEL IN OPEN BOREHOLE AT 6.4m DEPTH UPON COMPLETION.				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × 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

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 03-64

METRIC

W.P.	19-1351-46	LOCATION	N 4 859 873.3 E 354 338.2	ORIGINATED BY	GA
DIST		HWY 401	BOREHOLE TYPE 108mm Solid Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	24.11.03 - 24.11.03	CHECKED BY	SKP

SOIL PROFILE: ELEV DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × 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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. DATE DEPTH(m) ELEVATION(m) 27/11/03 0.9 107.2 03/12/03 0.7 107.4 22/12/03 0.6 107.5										

ONTMT4 5146-A.GPJ 07/01/04

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 03-65

METRIC

W.P. 19-1351-46 LOCATION N 4 859 814.1 E 354 437.4 ORIGINATED BY GA

DIST ___ HWY 401 BOREHOLE TYPE 108mm Solid Stem Augers COMPILED BY SS

DATUM Geodetic DATE 24.11.03 - 24.11.03 CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
107.2						
107.0 / 0.0	**TOPSOIL** (150mm)					
0.2	Clayey **SILT**, with sand, trace gravel, occ. iron oxide staining Firm to Very Stiff Brown Moist (TILL) (CL-ML)	1	SS	6	107	o
		2	SS	21	106	o
105.8 / 1.5	**SAND and SILT**, trace to some gravel, trace clay, occ. iron oxide staining Dense to Very Dense Brown Moist (TILL) (ML-nonplastic)	3	SS	32	105	o
		4	SS	48/ .102		o
		5	SS	40/ .076	104	o
	becoming grey	6	SS	35/ .076	103	o
		7	SS	50/ .127	102	o
		8	SS	48/ .150	101	o
100.1 / 7.2	**SAND**, fine grained, trace silt Very Dense Grey Wet (SM)				100	
		9	SS	68		o
99.0 / 8.2	END OF BOREHOLE AT 8.23m. BOREHOLE OPEN TO BOTTOM UPON COMPLETION. WATER LEVEL IN OPEN BOREHOLE AT 7.9m DEPTH UPON COMPLETION. BOREHOLE BACKFILLED AND SEALED WITH DRILL CUTTINGS AND BENTONITE.				99	

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × 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/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 03-66

METRIC

W.P. 19-1351-46
LOCATION N 4 859 768.8 E 354 517.4
ORIGINATED BY GA
DIST HWY 401
BOREHOLE TYPE 108mm Solid Stem Augers
COMPILED BY SS
DATUM Geodetic
DATE 24.11.03 - 24.11.03
CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
107.0					107
0.0 / 106.7	**TOPSOIL** (300mm)				
0.3 / 106.3 / 0.7	**SAND**, trace to some silt, trace gravel, occ. rootlets, Loose, Brown, (FILL) (SW)	1	SS	4	
	SAND and SILT, trace to some gravel, trace clay, occ. iron oxide staining, Dense to Very Dense, Brown, Moist, (TILL) (ML- nonplastic)	2	SS	42	106
		3	SS	75	105
		4	SS	48/ .102	104
		5	SS	39/ .076	
		6	SS	44/ .102	103
	becoming grey	7	SS	42/ .102	102
					101
100.3 / 6.7	occ. clay seams, Dense	8	SS	44	
	END OF BOREHOLE AT 6.71m. BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION. BOREHOLE BACKFILLED AND SEALED WITH DRILL CUTTINGS AND BENTONITE.				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × 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/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 03-67

METRIC

W.P. 19-1351-46	LOCATION N 4 859 500.0 E 354 632.4		ORIGINATED BY GA
DIST	HWY 401	BOREHOLE TYPE 108mm Solid Stem Augers	COMPILED BY SS
DATUM Geodetic	DATE 25.11.03 - 25.11.03		CHECKED BY SKP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) (approx.)
105.4						
0.0 / 105.1	**TOPSOIL** (300mm)					
0.3	Clayey **SILT**, with sand, trace gravel Very Stiff Brown Moist to Wet (TILL) (CL-ML)	1	SS	0	105	
		2	SS	16		o
		3	SS	22	104	o
103.2 / 2.2	**SAND and SILT**, trace to some gravel, trace clay, occ. iron oxide staining Very Dense Brown Moist (TILL) (ML-nonplastic)	4	SS	86	103	o
		5	SS	50/.102	102	o
	becoming grey	6	SS	51/.150	101	o
		7	SS	103	100	o
		8	SS	72	99	o
97.3		9	SS	112	98	o
8.1	END OF BOREHOLE AT 8.08m. WATER LEVEL IN OPEN BOREHOLE AT 3.1m DEPTH UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.					

DATE	DEPTH(m)	ELEVATION(m)
27/11/03	3.8	101.6
03/12/03	3.9	101.5
22/12/03	4.1	101.3

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × 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/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ONTMT4 5146-A.GPJ 07/01/04

+ 3, × 3: Numbers refer to Sensitivity — 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 03-68

METRIC

W.P. 19-1351-46 LOCATION N 4 859 434.6 E 354 663.9 ORIGINATED BY GA

DIST ____ HWY 401 BOREHOLE TYPE 108mm Solid Stem Augers COMPILED BY SS

DATUM Geodetic DATE 25.11.03 - 25.11.03 CHECKED BY SKP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
105.4					
105.2 / 0.2	TOPSOIL (150mm)				
0.2	Silty CLAY, trace sand, occ. rootlets, wood fibres and organic staining	1	SS	6	105
104.7 / 0.7	Firm, Dark Brown				
	Clayey SILT, with sand, trace gravel, Stiff to Very Stiff	2	SS	15	
103.9 / 1.5	Brown, Moist (TILL)				104
	SAND and SILT, trace to some gravel, trace clay, occ. iron oxide staining, Very Dense, Brown, Moist (TILL)(CL-ML), becoming grey	3	SS	105	
		4	SS	40/.076	103
		5	SS	56/.150	102
101.7 / 3.7	SAND, fine grained, trace silt, occ. gravel, Very Dense, Grey, Wet (SM)	6	SS	54	101
		7	SS	72	100
99.6 / 5.8	Gravelly SAND, occ. shale fragments, Very Dense	8	SS	50/.127	
99.0 / 6.4	Grey, Wet (SW)				

END OF BOREHOLE AT 6.38m.
BOREHOLE OPEN TO 6.1m.
WATER LEVEL IN OPEN BOREHOLE AT 4.3m DEPTH UPON COMPLETION.
BOREHOLE BACKFILLED AND SEALED WITH DRILL CUTTINGS AND BENTONITE.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × 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/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+ ³, × ³: Numbers refer to Sensitivity; 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 03-27

METRIC

W.P. 127-99-00	LOCATION N 4 859 700.0 E 354 560.7		ORIGINATED BY GA
DIST ____ HWY 401	BOREHOLE TYPE 108mm Solid Stem Augers		COMPILED BY SS
DATUM Geodetic	DATE 15.04.03 - 15.04.03		CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
106.0 / 106.0 / 0.0	**ASPHALT** (25mm) **SAND and SILT**, trace to some gravel, trace clay, occ. iron oxide staining Very dense Brown Moist (TILL) (ML-nonplastic)				106
					105
		1	SS	42	104
					103
		2	SS	105	102
101.7 / 4.3	Resistance to augering Becoming grey				
		3	SS	70/ .15	101
100.4 / 5.6	Occ. cobbles (inferred)				100
99.6 / 6.4		4	SS	60/ .15	

Column headings: 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 × POCKET PEN 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

END OF BOREHOLE AT 6.4m.
BOREHOLE OPEN AND DRY TO BOTTOM UPON COMPLETION.
Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEVATION(m)
24/04/03	5.9	100.1
05/27/03	0.4	105.6
06/17/03	0.4	105.6

$+^3, \times^3$: Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 03-27A

METRIC

W.P. 127-99-00 LOCATION N 4 859 698.0 E 354 555.0 ORIGINATED BY GA

DIST ____ HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS

DATUM Geodetic DATE 11.06.03 - 11.06.03 CHECKED BY SP

SOIL PROFILE: ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × POCKET PEN) (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
106.0 0.0	Auger without sampling to 9.1m						106				
							105				
							104				
							103				
							102				
							101				
							100				
							99				
							98				
96.9 9.1	**SAND and SILT**, trace to some gravel, trace clay Very dense Grey (TILL) (ML-non-plastic)		1	SS	80		97		○ (approx. 6)		
							96				

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

15 –○– 5 (20 above, 10 below): (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 03-27A

METRIC

W.P.	127-99-00	LOCATION	N 4 859 698.0 E 354 555.0	ORIGINATED BY	GA
DIST	HWY 401	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	11.06.03 - 11.06.03	CHECKED BY	SP

Column headings:

- SOIL PROFILE: ELEV / DEPTH; DESCRIPTION; STRAT PLOT
- SAMPLES: NUMBER; TYPE; "N" VALUES
- GROUND WATER CONDITIONS
- ELEVATION SCALE
- DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100
- SHEAR STRENGTH kPa: ○ UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × 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/m³
- REMARKS & GRAIN SIZE DISTRIBUTION (%): GR, SA, SI, CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	SAND and SILT, trace to some gravel, trace clay Very dense Grey Wet (TILL) (ML-non-plastic)				96 95 94
	Free water encountered	2	SS	68	93
	Resistance to augering Occ. sand seams, gravel seam	3	SS	64	92
91.2 / 14.8	**SILT**, occ. sand Very dense Grey Wet (ML-non-plastic)	4	SS	67/ .127	91 90
89.7 / 16.3	Resistance to augering **SAND and SILT**, trace to some gravel, occ. cobbles and /or boulders, trace clay Very dense Grey Moist (TILL) (ML-non-plastic) High resistance to augering below 17m depth	5	SS	108/ .102	89 88
		6	SS	112/ .102	87
86.0		7	SS	111/.127	86

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

ONTMT4 5146.GPJ 07/11/03

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 03-27A

METRIC

W.P.	127-99-00	LOCATION	N 4 859 698.0 E 354 555.0	ORIGINATED BY	GA
DIST	HWY 401	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	SS
DATUM	Geodetic	DATE	11.06.03 - 11.06.03	CHECKED BY	SP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
20.0	Clayey **SILT** with sand, trace gravel Hard Grey (TILL) (CL-ML)				86	o
					85	
84.4		8	SS	75/.152		o
21.6	END OF BOREHOLE AT 21.64m. BOREHOLE OPEN TO 12.8m UPON COMPLETION. WATER LEVEL IN OPEN BOREHOLE AT 2.1m DEPTH UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS SEALED WITH HOLE PLUG/CONCRETE AND PATCHED WITH ASPHALT AT SURFACE.					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × 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/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

ONTMT4 5146.GPJ 10/11/03

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 03-32

METRIC

W.P. 127-99-00 | LOCATION N 4 859 569.2 E 354 602.2 | ORIGINATED BY GA

DIST ___ HWY 401 | BOREHOLE TYPE 108mm Solid Stem Augers | COMPILED BY SS

DATUM Geodetic | DATE 23.04.03 - 23.04.03 | CHECKED BY SP

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
105.3 / 0.0	Silty **CLAY**, trace sand Brown Moist (CL)	1	GS					
104.2 / 1.1	Clayey **SILT** with sand, trace gravel Very stiff Brown Moist to wet (TILL) (CL-ML)	1	SS	26				
102.7 / 2.6	**SAND and SILT**, trace clay, trace to some gravel Very dense Grey Wet (TILL) (ML-non-plastic)	2	SS	105/.150				
		3	SS	106/.254				
	Some clay	4	SS	101/.3	4	45	36	15
98.8 / 6.6	END OF BOREHOLE AT 6.55m. BOREHOLE OPEN TO BOTTOM UPON COMPLETION. WATER LEVEL IN OPEN BOREHOLE AT 1.2m DEPTH UPON COMPLETION. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.							

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEVATION(m)
24/04/03	0.6	104.7
05/27/03	0.0	105.3
06/17/03	0.2	105.1

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (105, 104, 103, 102, 101, 100, 99); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × 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

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Golder Associates

Foundation Design

RECORD OF BOREHOLE No F8

1 OF 1 METRIC

PROJECT 001-8033

W.P. 127-99-00 LOCATION Sta. 10+834 o/s 51m Lt. ORIGINATED BY SB

DIST 6 HWY 401 BOREHOLE TYPE 101mm Solid Stem Augers COMPILED BY DKB

DATUM GEODETIC DATE March 7, 2002 CHECKED BY ASP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
106.0	GROUND SURFACE					
0.0	Asphalt					
	Sand and Gravel, trace silt (Granular Fill)					
105.4 / 0.6	Brown Moist					
	Sand and Silt, some gravel, trace clay, occasional cobbles and/or boulders (Till) Compact to very dense Brown becoming grey below 3.7m depth Moist to wet at 9.1m depth	1	SS	13	105	
	Note: Non-plastic Atterberg limits results measured for samples 3 and 8.	2	SS	41	104	
		3	SS	65	103	
		4	SS	109		
		5	SS	65/.15	102	
		6	SS	75/.15	101	
		7	SS	73/.15	100	
					99	
		8	SS	39	98	
					97	
96.2 / 9.8		9	SS	65		

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

END OF BOREHOLE

Note:

1. Open borehole dry upon completion of drilling.

2. Water level measured in piezometer at 0.2m depth (El. 105.8m) on March 20, 2002.

3. Water in piezometer purged on March 27,2002.

4. Water level measured in piezometer at 1.8m depth (El. 104.2m) on March 28,2002.

5. Water level measured in piezometer at 0.2m depth (El. 105.8m) on April 01,2002.

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

APPENDIX B

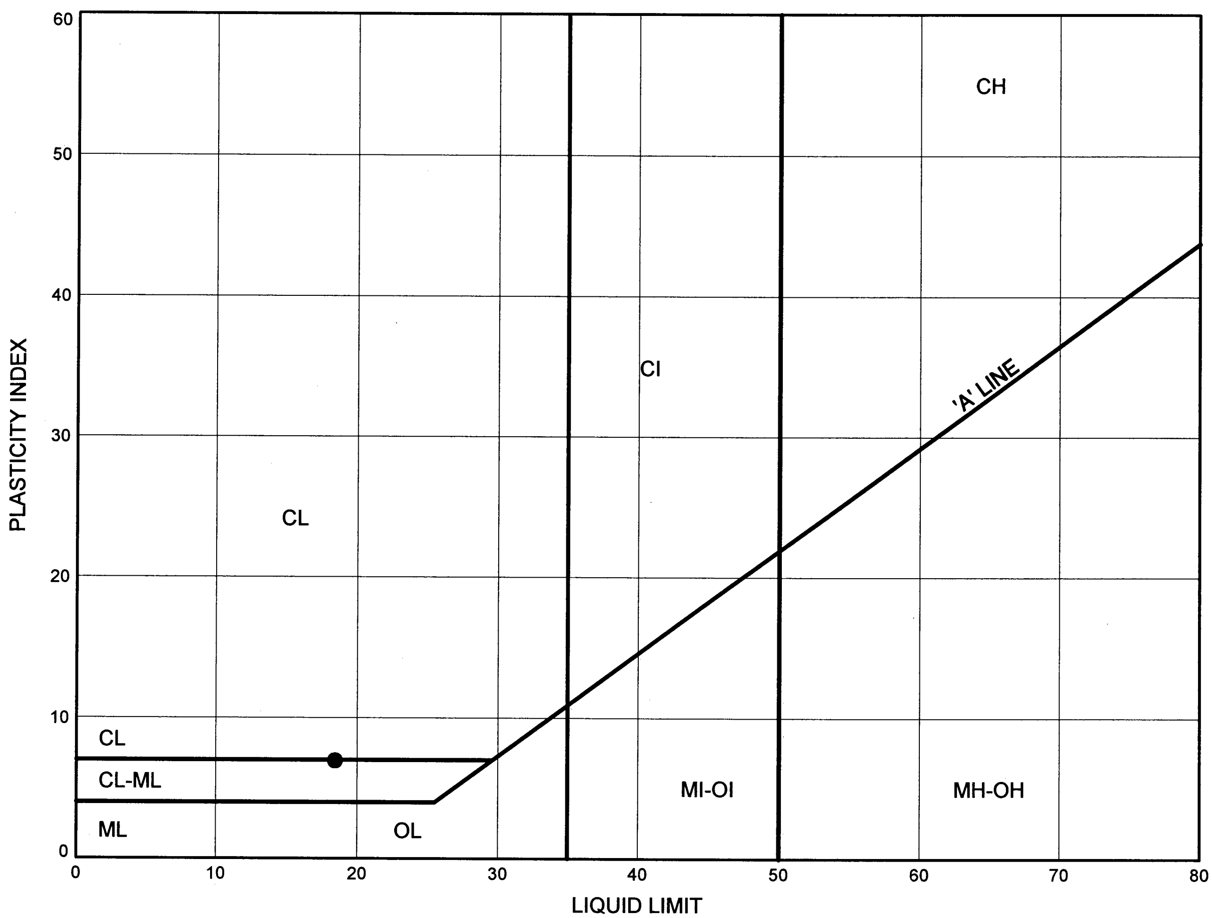
Plasticity Chart

Stevenson Road Interchange

ATTERBERG LIMITS TEST RESULTS

FIGURE B1

CLAYEY SILT (TILL)



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	03-32	1.83	103.47

THURBALT 5146.GPJ 14/01/04

Date January 2004

Project 127-99-00

THURBER

Prep'd SS

Chkd. SKP

APPENDIX C

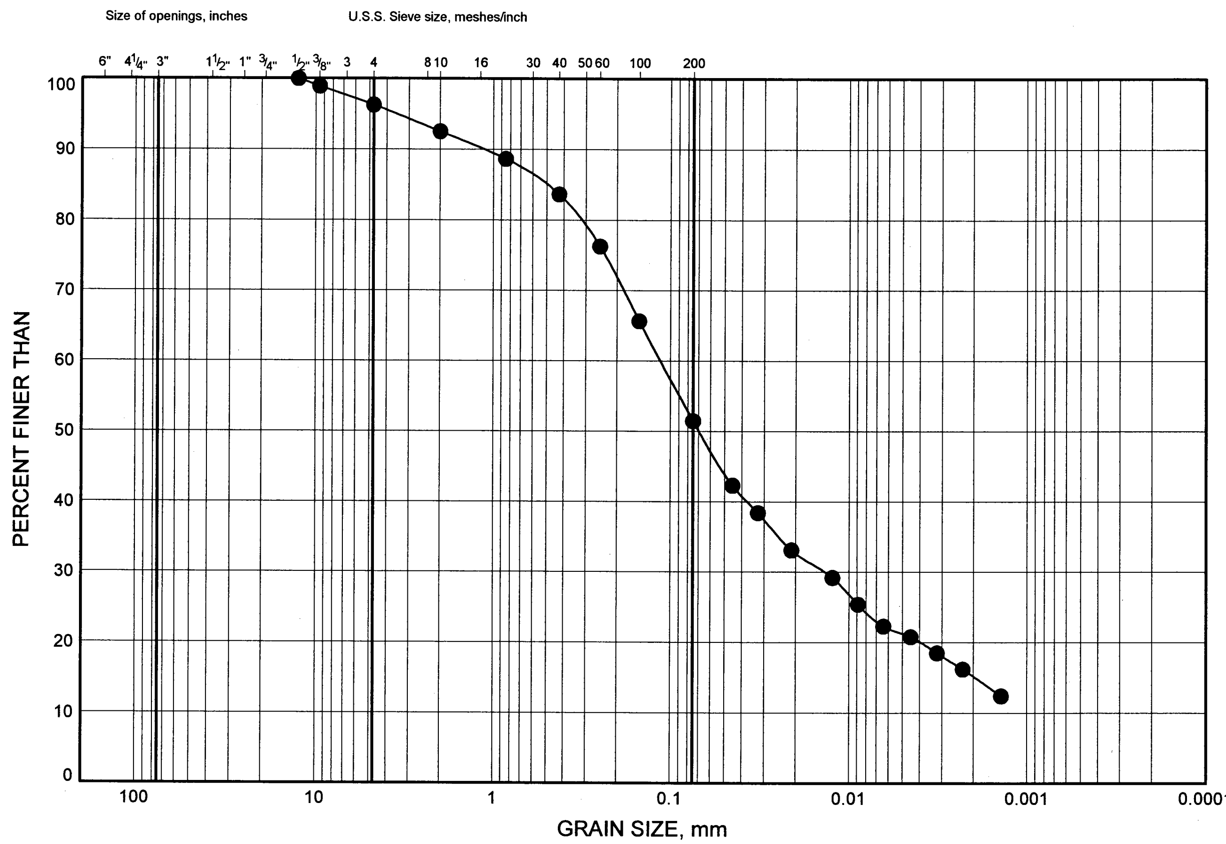
Grain Size Distribution Curve

Stevenson Road Interchange

GRAIN SIZE DISTRIBUTION

FIGURE C1

SAND AND SILT (TILL)



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	03-32	6.40	98.90

THURBGSD 5146.GPJ 14/01/04

Date January 2004

Project 127-99-00

THURBER

Prep'd SS

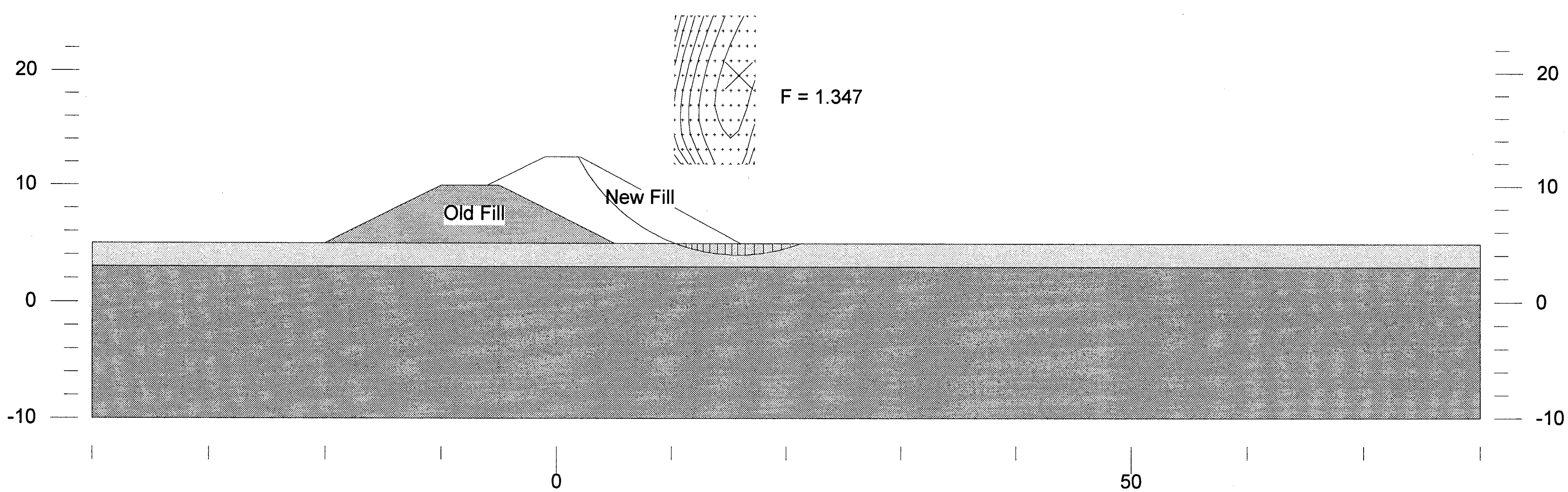
Chkd. SKP

APPENDIX D

Stability Analyses Results

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
New Fill (SSM)	21	0	30	0	0
Old Fill	20	0	29	0	0
Sand & Silt Till	20	0	32	0	1
Sand & Silt Till	21	0	35	0	1

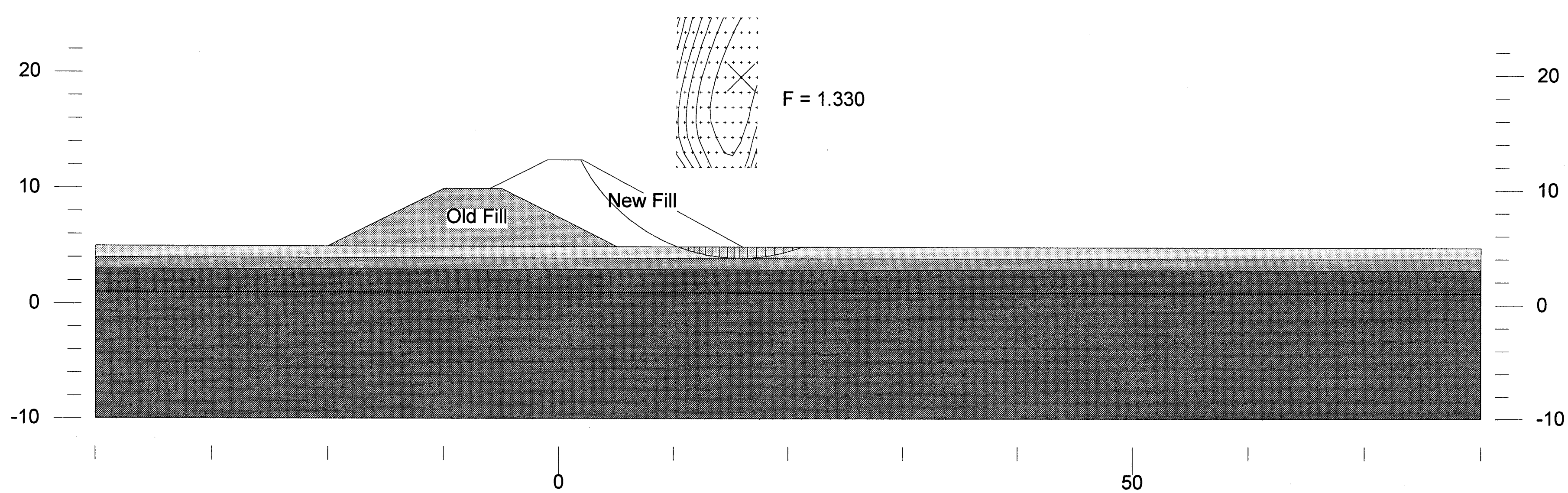
Thurber Engineering Ltd. - Toronto
19-1351-46
Stevenson / Hwy 401 Interchange - CP Embankments
January 6, 2004
Stability of New Fill on Old Embankment - Sta.9+900 North Approach
Figure D1 Drained Analysis



13/01/2004 5:54:29 PM C:\SPANG\THURBE~8\19-135~2\CPEMBA~1.WAL\FIGD1.GSL Thurber Engineering Ltd. - Toronto F = 1.347

Thurber Engineering Ltd. - Toronto
19-1351-46
Stevenson / Hwy 401 Interchange - CP Embankments
January 6, 2004
Stability of New Fill on Old Embankment - Sta.9+900 North Approach
Figure D2 Drained Analysis

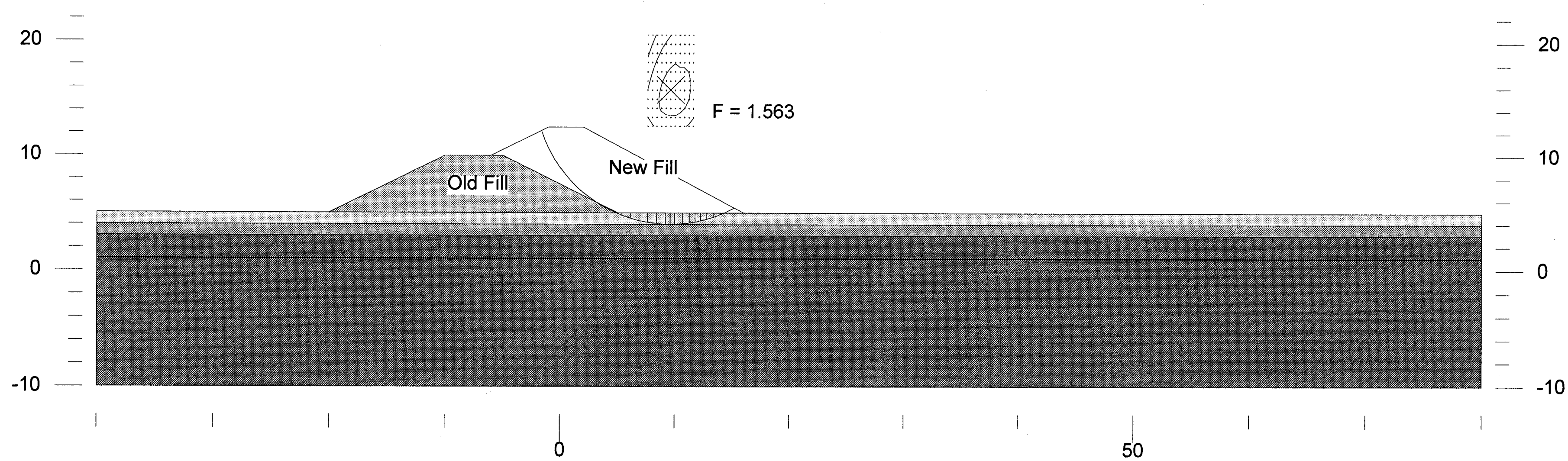
	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
New Fill (SSM)	21	0	30	0	0
Old Fill	20	0	29	0	0
Clayey Silt Till	19	1	30	0	1
Clayey Silt Till	19	1	30	0	1
Sand & Silt Till	20	0	32	0	1
Sand & Silt Till	21	0	35	0	1



13/01/2004 5:55:18 PM C:\SPANG\THURBE~8\19-135~2\CPEMBA~1.WAL\FI9900B1.GSL Thurber Engineering Ltd. - Toronto F = 1.330

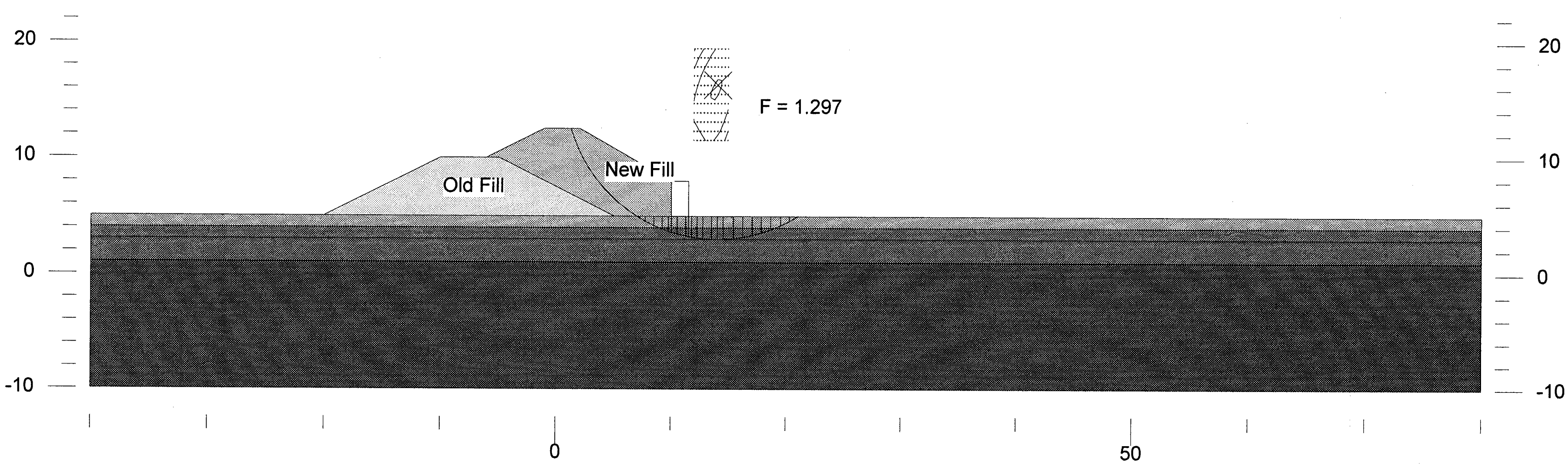
	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
New Fill (SSM)	21	0	30	0	0
Old Fill	20	0	29	0	0
Clayey Silt Till	19	35	0	0	1
Clayey Silt Till	19	100	0	0	1
Sand & Silt Till	20	0	32	0	1
Sand & Silt Till	21	0	35	0	1

Thurber Engineering Ltd. - Toronto
19-1351-46
Stevenson / Hwy 401 Interchange - CP Embankments
January 7, 2004
Stability of New Fill on Old Embankment - Sta.9+900 North Approach
Figure D3 Short Term Analysis



	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Concrete Wall	24.5	500	0	0	0
New Fill (SSM)	21	0	30	0	0
Old Fill	20	0	29	0	0
Clayey Silt Till	19	1	30	0	1
Clayey Silt Till	19	1	30	0	1
Sand & Silt Till	20	0	32	0	1
Sand & Silt Till	21	0	35	0	1

Thurber Engineering Ltd. - Toronto
19-1351-46
Stevenson / Hwy 401 Interchange - CP Embankments
January 7, 2004
Stability of New Fill on Old Embankment - Sta.9+900 North Approach (with retaining wall)
Figure D4 Drained Analysis



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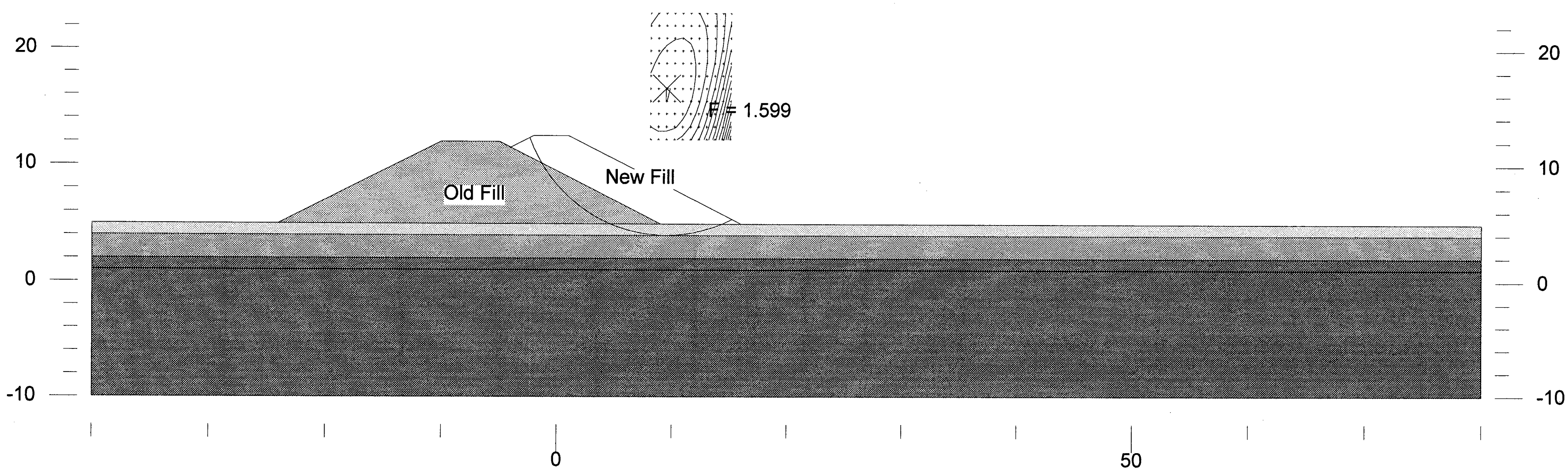
	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
New Fill (SSM)	21	0	30	0	0
Old Fill	20	0	29	0	0
Silty Clay	19	35	0	0	1
Clayey Silt Till	19	100	0	0	1
Sand & Silt Till	20	0	32	0	1
Sand & Silt Till	21	0	35	0	1

Thurber Engineering Ltd. - Toronto
19-1351-46
Stevenson / Hwy 401 Interchange - CP Embankments
January 6, 2004
Stability of New Fill on Old Embankment - Sta. 10+100 South Approach
Figure D6 Short Term Analysis



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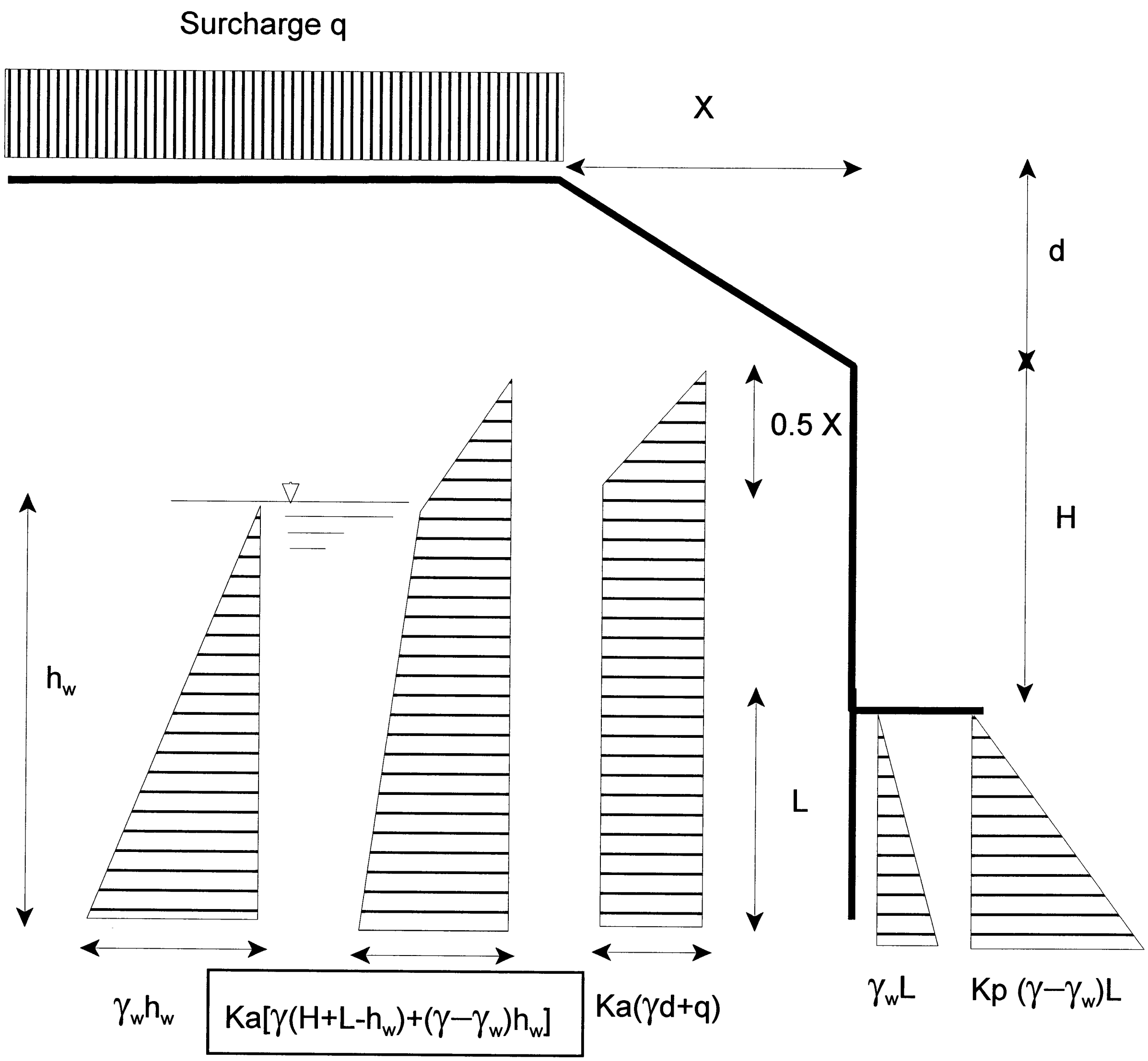
APPENDIX E

Figure



γ	=	unit weight of soil
γ_w	=	unit weight of water
Ka	=	active earth pressure coefficient
Kp	=	passive earth pressure coefficient

THURBER

LATERAL PRESSURE DISTRIBUTION

ANCHORED WALLS

FIGURE E1