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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
RETAINING WALL AT HIGHWAY 28
HIGHWAY 401 WIDENING
NORTHUMBERLAND COUNTY
G.W.P. 274-96-00, AGREEMENT NO. 4005-A-000103
(c)**

Submitted to:

**The Greer Galloway Group Inc.
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Peterborough, Ontario
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January 2001

001-1142A

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PART A

**FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by The Greer Galloway Group Inc. (Greer Galloway) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detailed foundation investigation and design services for the widening of Highway 401 between former Highway 2 and Highway 28 near Port Hope, Ontario. The foundations engineering component of the project includes a proposed retaining wall at Highway 28 to accommodate a new W-N/S Ramp, and a permanent cut section between Cranberry Road and Choate Road along the median of Highway 401. This report addresses the proposed retaining wall at Highway 28.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed structure by drilling boreholes, and carrying out in-situ tests and laboratory tests on selected samples. Existing subsurface data from a report prepared by the Ministry of Transportation, Ontario ("Foundation Report on Underpass Bridge at New Highway 401 Crossing Highway 28, 1 Mile North of Port Hope", GEOCREC No. 30M16-8, dated 1957) was used to supplement the data obtained in the current investigation.

The terms of reference for the scope of work are outlined in our Total Project Management proposal P01-1216, dated July 2000. The work was carried out in accordance with our Quality Control Plan for Foundation Design Services, dated August 2, 2000.

The proposed preliminary alignment for the retaining wall was provided to us in digital format by Greer Galloway on October 25, 2000.

2.0 SITE DESCRIPTION

The proposed retaining wall is located parallel to and on the south side of the existing Highway 401, between the south pier and south abutment of the existing Highway 28 underpass structure. The site is located in the Town of Port Hope, in Northumberland County.

Highway 401 has been constructed in cut at the location of the Highway 28 underpass. The existing Highway 28 underpass is a three-span structure supported on steel H-piles, with the underside of the pile cap at the south abutment located at about Elevation 103.9 m, at the existing Highway 401 grade. These founding conditions were determined from the general layout drawing for the existing structure, provided by Greer Galloway (Drawing No. D4001-1, titled "Hope Township Bridge No. 18, Highway 401 Underpass, General Layout", dated May 1958).

Within the project limits, the vegetation cover consists of grass, bushes and small trees.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on October 10, 2000. At this time one borehole – Borehole 11 – was advanced within the limits of the proposed retaining wall, near the west end of the proposed structure. Also referenced in this report are Boreholes 1 and 4 from the 1957 subsurface investigation carried out by the Ministry of Transportation (GEOCREs No. 30M16-8, referenced in Section 1.0). Boreholes 1 and 4 were drilled for the south abutment of the then-proposed Highway 28 bridge, and were located on the east and west sides of Highway 28, respectively.

The investigation was carried out using a bombardier-mounted CME-55 drill rig supplied and operated by Eastern Soil Investigation Ltd. of Courtice, Ontario. Borehole 11 was extended to a depth of 18.4 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration test (SPT) procedure. Groundwater conditions in the open borehole were observed throughout the drilling operations. A piezometer, consisting of a 3 m long slotted section threaded onto 12 mm diameter rigid PVC tubing, was installed in the borehole to permit monitoring of the groundwater level at the site.

The borehole caved after withdrawal of the augers to within about 3 m of ground surface. Above this level, the borehole was backfilled using bentonite pellets. The borehole location will be inspected periodically to monitor for backfill settlement.

The field work was supervised on a full-time basis by a member of our engineering staff who located the borehole in the field, directed the drilling, sampling, and in-situ testing operations, and logged the borehole. The soil samples were identified in the field, placed in labelled containers and transported to our laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits tests and grain size analyses were carried out on selected samples.

The retaining wall location was surveyed and staked in the field by personnel from Greer Galloway, in order to locate the borehole on the retaining wall alignment. The ground surface elevation at Borehole 11 was surveyed by Greer Galloway and is referenced to the geodetic datum. The locations of the boreholes are shown on Drawing 1.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located on the margin of the physiographic region known as the Iroquois Plain. The surficial soils within this region are comprised of sands, silts and clays deposited by glacial Lake Iroquois. To the north of the Port Hope area, these lacustrine deposits are interrupted by drumlins comprised of till soils, which stood as islands in glacial Lake Iroquois. (Reference: "The Physiography of Southern Ontario", 3rd Edition, Chapman and Putnam, 1984.).

Limestone bedrock was encountered at about Elevation 90 m in the Ganaraska River valley, about 200 m west of Highway 28, in a 1957 subsurface investigation by E.M. Peto Associates Ltd. for the Ministry of Transportation. (Reference: "Report on Soil and Foundation Conditions, Highway 401 – Ganaraska River Crossing, Hope Township Bridge 16, W.P. 757-56, GEOCREs No. 30M16-7).

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in Borehole 11 together with the results of laboratory testing carried out on selected soil samples are given on the attached Record of Borehole sheet and Figures 1 and 2. The detailed subsurface conditions encountered in the 1957 boreholes are given on the Record of Borehole sheets in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils at the proposed retaining wall location consist of a thin layer of topsoil overlying a clayey silt till deposit which contains interlayers / lenses of silty clay and sand and gravel. The upper portion of the clayey silt till has a generally stiff to very stiff consistency, while the lower portion of the deposit is hard. Bedrock was not encountered in Borehole 11 or in the boreholes advanced during the 1957 investigation. It is noted that the boreholes drilled in 1957 encountered a 3.5 m to 4 m thick layer of firm to stiff silty clay overlying the till deposit; this firm to stiff material was not encountered in Borehole 11.

The locations and elevations of the borings, together with the interpreted stratigraphic profile along the proposed retaining wall, are shown on Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes for the current and 1957 investigations is provided in the following sections.

4.2.1 Topsoil

The upper 300 mm of soil in Borehole 11 contained organic matter. Boreholes 1 and 4, drilled in 1957, encountered between 300 mm and 600 mm of topsoil.

4.2.2 Silty Clay

Boreholes 1 and 4 from the 1957 investigation encountered a 3.5 m to 4 m thick layer of grey silty clay below the topsoil, overlying the clayey silt till, between about Elevations 106.5 m and 102.5 m. This layer was not encountered in Borehole 11 during the current investigation, due in part to the cut for construction of the Highway 28 underpass.

Measured Standard Penetration Test (SPT) 'N' values in this layer ranged from 6 to 14 blows per 0.3 m of penetration, and laboratory unconfined compression testing measured shear strengths of about 20 kPa to 60 kPa. These test results indicate that this silty clay layer generally has a firm to stiff consistency.

Measured water contents for these samples ranged from 11 to 44 per cent. Atterberg limits testing indicated a plastic limit of 10 to 22 per cent, and a plasticity index of 7 to greater than 34 per cent, indicating that this silty clay is inorganic and of high plasticity. Measured water contents ranged from 11 to 44 per cent.

4.2.3 Clayey Silt Till

A clayey silt till deposit was encountered below Elevation 104.3 m in Borehole 11, and below about Elevation 102.5 m in Boreholes 1 and 4. It is noted that the records for Boreholes 1 and 4 described this deposit as a sandy clay loam with gravel; the results of in-situ and laboratory testing indicate that this loam would be described as clayey silt till according to MTO's current soil classification system. The clayey silt till contains a high percentage of sand, and trace to some

gravel. In Borehole 11, a 2.7 m thick layer containing non-plastic silty sand till was encountered within the predominantly cohesive till deposit; the grain size distribution test result for a sample of this material is shown on Figure 1.

Above Elevations 91 m to 91.5 m, the measured SPT 'N' values in the three boreholes ranged from 4 to 29 blows per 0.3 m of penetration, but were typically less than 20 blows per 0.3 m of penetration. Laboratory unconfined compression testing measured shear strengths of about 10 kPa to 20 kPa in Borehole 1, as shown on the Record of Borehole sheet. These test results indicate that this portion of the till deposit is firm to very stiff, but predominantly stiff to very stiff. Below Elevation 91 m to 91.5 m, the measured SPT 'N' values were generally greater than 100 blows per 0.3 m of penetration, indicating a hard consistency.

Atterberg limits testing carried out on selected clayey silt till samples indicate that it is of low plasticity with plastic limits of 9 to 13 per cent, liquid limits of 13 to 15 per cent, and a plasticity index between 3 and 5 per cent; the results of the Atterberg testing are shown on Figure 2 and on the Record of Borehole sheets. The measured natural water contents ranged from about 8 to 13 per cent, typically near the plastic limit of the soil.

Between about Elevation 95 m and 93 m in all boreholes, a thin interlayer of silty clay was encountered or inferred from laboratory testing; the measured plastic and liquid limits of this interlayer were about 12 to 15 and 20 to 49 per cent, respectively, with measured water contents ranging from 15 to 26 per cent.

In Borehole 11, a 1.4 m thick interlayer of water-bearing, grey sand and gravel was encountered at about Elevation 91.4 m, immediately overlying the hard clayey silt till. The SPT 'N' value was greater than 100 blows per 0.3 m of penetration indicating a very dense state of packing. Boreholes 1 and 4 show refusal to dynamic cone penetration testing at about Elevation 91.1 m which likely corresponds to this interlayer.

4.3 Groundwater Conditions

Water levels were not recorded for Boreholes 1 and 4, drilled in the 1957 investigation. A piezometer was installed in Borehole 11, and the water level was measured on October 23, 2000 at 3.8 m depth (approximately Elevation 100.8 m).

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to be higher during wet periods of the year.

GOLDER ASSOCIATES LTD.



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SEMP/LCC/ASP/FJH/clg

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PART B

**FOUNDATION DESIGN REPORT
RETAINING WALL AT HIGHWAY 28
HIGHWAY 401 WIDENING
NORTHUMBERLAND COUNTY
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5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides recommendations on the geotechnical aspects of design of the proposed retaining wall on the south side of Highway 401, based on interpretation of the factual information obtained during the current and 1957 investigations. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made regarding 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 method and scheduling.

It is understood that the W-N/S ramp will be shifted southward to pass between the south pier and south abutment of the existing Highway 28 underpass, in order to accommodate the widening of Highway 401. The grade of both the existing Highway 401 and the proposed W-N/S ramp is about Elevation 104 m. This will require that the south abutment foreslope be cut; a retaining wall is proposed to maximize the space available for the new ramp. It is understood that the front face of the proposed retaining wall is to be located 5.9 m to 7.3 m north of the south abutment face. It is further understood that the wall is to be about 60 m in length, and that it will have a maximum height of approximately 4 m.

Based on the general layout drawing provided by Greer Galloway for the existing Highway 28 underpass structure (Drawing No. D4001-1, titled "Hope Township Bridge No. 18, Highway 401 Underpass, General Layout", dated May 1958), the bridge abutments and piers are founded on steel H-piles driven to about Elevations 90.5 m to 89.5 m. The underside of the pile cap at the south abutment was designed to be at about Elevation 103.9 m. It is noted that the front two rows of piles at the south abutment were designed to be battered at 3 vertical to 1 horizontal.

5.2 Retaining Wall Options

The subsoils at the proposed retaining wall location consist of a thin surficial layer of topsoil overlying clayey silt till. The till is generally stiff to very stiff above Elevation 91 m, and hard

below that elevation. In the 1957 boreholes, drilled prior to the highway cut, a 3.5 m thick layer of silty clay was present atop the clayey silt till. It is anticipated that the proposed retaining wall will be founded on either shallow foundations placed on the predominantly stiff to very stiff clayey silt till, or on deep foundations (steel H-piles) driven to practical refusal on the hard clayey silt till below about Elevation 91 m. It is understood that three concepts are under consideration for the proposed retaining wall:

- a soldier pile and concrete panel wall;
- a mechanically-reinforced soil retaining wall system; and
- a cast-in-place concrete cantilever retaining wall.

Foundation engineering recommendations are provided for each of these types of retaining systems, and construction considerations applicable to the types of foundation systems are provided in the following subsections.

5.3 Soldier Pile and Concrete Panel Wall

A soldier pile and concrete panel wall would consist of steel H-piles driven to the hard clayey silt till at about Elevation 91 m, approximately 13 m below the proposed W-N/S Ramp grade. Lateral support to the soldier pile and concrete panel wall system would be in the form of permanent soil anchors. Suitable drainage and insulation should be provided to the back of this type of wall which will otherwise be formed against the in-situ clayey silt till deposit and fill materials.

5.3.1 Axial Geotechnical Resistance

For design, the factored axial resistance at Ultimate Limit States (ULS) for steel HP 310 x 110 H-piles driven to practical refusal on the hard clayey silt till below Elevation 91 m may be taken as 1,600 kN. The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 1,100 kN. To achieve these design resistances, the piles should be driven to at least Elevation 91 m and to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.

The contract drawing for the wall foundations should include the following note with respect to the piles:

“Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,200 kN per pile but must be driven below Elevation 91 m.”

5.3.2 Soil Anchor Design

Soil anchors may be necessary for lateral restraint of the wall. These permanent anchors should be provided with suitable corrosion protection.

The sustained working load should not be greater than 60 per cent of the ultimate tensile strength of the anchor tendons or bars. The anchors may be sized based on the following ultimate bond stresses acting between the grout and soil:

Firm to very stiff clayey silt till above Elevation 91 m	40 kPa
Hard clayey silt till below Elevation 91 m	120 kPa

The above ultimate bond stresses assume a fixed anchor length (bond zone) of not greater than 8 m. The fixed length of the anchor should be maintained behind a line drawn upward at 45 degrees from the base of the piles. In addition, the fixed length should be maintained outside of the area of existing piles supporting the bridge abutment.

Because the ground-to-anchor bond in soil is highly dependent upon the installation technique, the Contractor should be held to an anchor performance specification enforced by proof tests on all anchors and a performance test on at least one anchor. The Contract Documents should incorporate a Non-Standard Special Provision (NSSP) to this end. Anchor installation and testing should be carried out under the full-time inspection of a geotechnical engineer. A performance test should be carried out, to 200 per cent of the design working load, on at least one anchor to confirm the design and the Contractor's installation method. In addition, each anchor should be proof-tested to 125 per cent of the working load. The tensile stress in the anchor bar or strands during test loading should not exceed 80 per cent of the guaranteed ultimate tensile strength of the bar or strands.

5.3.3 Resistance to Lateral Loads

The horizontal resistance for the soils in front of an individual pile can be estimated using the following equation:

$$k_h = n_h z / d$$

where:

k_h = coefficient of horizontal subgrade reaction (MPa/m)

d = pile width (m)

$n_h z$ = constant of horizontal subgrade reaction (MPa) = 15 MPa for the firm to very stiff clayey silt till deposit (Note: for these cohesive soils, $n_h z$ is equivalent to $k_{s1}/5$)

5.4 Mechanically-Reinforced Soil Retaining Wall System (Retained Soil Systems)

A mechanically-reinforced soil retaining wall system (Retained Soil System or RSS wall) consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material. Use of this system is considered appropriate for construction of the proposed W-N/S Ramp retaining wall, given that the maximum wall height will be about 3.5 m. The founding stratum is variable along the length of the wall, with very stiff clayey silt till at the west end and firm to very stiff clayey silt till along the remainder; this will result in some differential settlement along the wall of the order of 25 mm, which would be within the allowable limits for this system.

The reinforced earth mass will be founded on firm to very stiff, but predominantly stiff to very stiff clayey silt till assuming it will be placed at a nominal depth below the proposed W-N/S Ramp grade. For the reinforced earth mass founded on this clayey silt till, a factored geotechnical resistance at ULS of 250 kPa may be assumed for design. The geotechnical resistance at SLS will be governed by the actual dimensions of the RSS wall. It is recommended that the existing ground surface grades behind the wall be maintained. Provided that the top of the wall is no higher than the existing grade, the loading on the subsoils will not induce significant settlements and a geotechnical resistance at SLS of 150 kPa may be assumed. A coefficient of friction equal to 0.55 may be assumed between the granular fill of the RSS wall and the clayey silt till founding soils.

The internal stability of the mechanically-reinforced soil wall should be checked by the RSS supplier / designer. For the proposed height of wall and assuming level ground behind the wall, an adequate factor of safety against failure is obtained for the wall for global stability. The majority of the wall, however, will have sloping ground behind it and further analysis would be required to confirm global stability adequacy. If the RSS option is feasible, therefore, full details of the ground surface profiles behind and in front of the wall must be provided to complete the analysis.

The granular fill material for the reinforced soil system should be placed on native, undisturbed soils. In this regard, all topsoil, existing fill and soft or loose materials should be removed and the subgrade should be inspected by qualified geotechnical personnel prior to placement of the granular materials.

5.5 Cast-in-Place Concrete Cantilever Retaining Wall

For this option, the concrete cantilever wall may be supported on steel H-piles driven to found within the hard clayey silt till at approximately 13 m below the proposed W-N/S Ramp grade. Consideration has also been given to the use of shallow spread footings for support of the wall.

5.5.1 Shallow Foundations

Given the proposed W-N/S Ramp grade of Elevation 104 m, the highest acceptable founding elevation will be 102.8 m in order to provide a minimum of 1.2 m of earth cover for protection against frost penetration. Spread footings at this level will be placed on the clayey silt till which has variable consistency along the length of the wall. The factored geotechnical resistance at ULS which should be used for footing design at this founding elevation is 250 kPa.

The settlement of footings founded on this firm clayey silt till will be dependent on the footing size and configuration, and on the applied loads. For example, assuming spread footings founded at Elevation 102.8 m, it is expected that under an applied load of 70 kPa the settlement of the wall would be less than 12 mm. This is due to the fact that the original ground surface in this area was at about Elevation 107 m or some 2 m higher than the current ground surface. For a 3 m wide footing, it is expected that up to 25 mm of settlement would occur under an applied load of 150 kPa. Due to the sensitivity of the settlement to the footing size and founding elevation, a settlement analysis

should be undertaken if spread footings are a feasible option. The analysis should be based on the footing design and applied loads to determine appropriate design values for SLS conditions.

The geotechnical resistances provided are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC, assuming an unfactored coefficient of friction of 0.4 between the concrete and the clayey silt till founding soils.

5.5.2 Deep Foundations

5.5.2.1 Axial Geotechnical Resistance

Consideration could be given to supporting a cast-in-place concrete wall on steel H-piles driven to refusal on the hard clayey silt till at about Elevation 91 m, about 13 m below the proposed W-N/S Ramp grade. For design, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to practical refusal may be taken as 1,600 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,100 kN.

To achieve the above design resistances, the piles should be driven to at least Elevation 91 m and to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.

The contract drawing for the wall foundations should include the following note with respect to piles:

“Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,200 kN per pile but must be driven below Elevation 91 m.”

5.5.2.2 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. Where required, the horizontal reaction to the pile can be estimated using the following equation:

$$k_h = n_h z / d$$

where:

- k_h = coefficient of horizontal subgrade reaction (MPa/m)
- d = pile width (m)
- $n_h z$ = constant of horizontal subgrade reaction (MPa) = 15 MPa for the firm to very stiff clayey silt till (Note: for these cohesive soils, $n_h z$ is equivalent to $k_{s1}/5$)

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

<i>Pile Spacing in Direction of Loading D = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor R</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.5.2.3 Frost Protection

The pile caps should be provided with 1.2 m of soil cover for frost protection.

5.6 Lateral Earth Pressures for Design

The lateral pressures acting on the proposed retaining wall will depend on the native soils and, where used, the type and method of placement of the backfill materials behind the wall, as well as on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the retaining wall in accordance with OHBDC:

- Where fill is to be placed behind the proposed wall, select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- Where fill soils are placed and compacted behind the retaining wall using hand-operated equipment only, a compaction surcharge equal to 11 kPa should be included in the lateral earth pressures for structural design, in accordance with OHBDC (Commentary Section C6-7.4.3). Compaction equipment should be used in accordance with OPSS 501.06. This value of 11 kPa is given since small equipment only may be used at this site due to the anticipated restricted access between the wall and the bridge abutment. If it is expected that large compaction equipment will be used, a compaction surcharge equal to 16 kPa should be included.
- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the retaining wall (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).
- For a soldier pile and concrete lagging wall or for retaining structures backfilled as per Case I, the following parameters (unfactored) may be assumed:

Soil unit weight: 20 kN/m³

Coefficients of lateral earth pressure:

'active', K_a	0.35
'at rest', K_o	0.50

- For retaining structures backfilled as per Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
'active', K_a	0.27	0.31
'at rest', K_o	0.43	0.47

- If the wall support allows lateral yielding (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (restrained structure), at-rest pressures should be assumed for geotechnical design.

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. If sloping ground will be present above the wall, the coefficient of active earth pressure, K_a , must be adjusted. Appropriate values for K_a can be provided once the slope configuration is established.

5.7 Design and Construction Considerations

5.7.1 Design and Construction Considerations for Shallow Foundations and Pile Caps

At the proposed retaining wall location, excavation for shallow spread footings or for pile caps will extend a minimum of 1.2 m below the existing Highway 401 grade and will cut into the abutment foreslope, extending through existing fill, soft to stiff silty clay (if present) and firm to very stiff clayey silt till. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The firm to very stiff clayey silt till soils at this site would be classified as Type 3 soil. Temporary open-cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, the excavation could also be carried out within a braced excavation or a trench box.

The clayey silt till soils comprising the excavation subgrade will be sensitive to disturbance from ponded water, construction traffic and frost. All foundation excavations should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that soil having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. Provision should be made to sub-excavate below founding level where unsuitable subgrade soils are encountered, and to replace the removed material with compacted granular soil or lean mix concrete.

Groundwater seepage into the excavations through the fill, clayey silt till and any lenses or interlayers within the till could occur, although this is expected to be minor. Pumping from

properly filtered sumps or a filtered drain located at the base of the excavation outside of the actual footing limits will provide adequate groundwater control during foundation excavations. Surface water run-off should be directed away from the excavation.

5.7.2 Design and Construction Considerations for Deep Foundations and Tied-Back Soldier Pile Wall


The following must be considered in order to ensure that deep foundations can be successfully installed at this site:

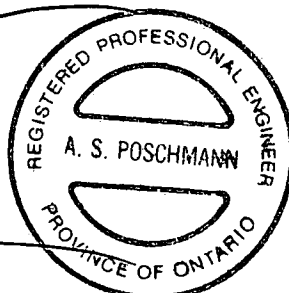
- Piles driven under the bridge will have to be spliced and special pile-driving machinery may be required in order to meet the height restrictions imposed by the vertical clearance under the existing Highway 28 structure.
- Given the location of the proposed wall in relation to the existing south abutment and wing walls – which are supported on piles, some of which are battered – there is a possibility of the piles for the new wall interfering with the existing piles. In addition, where a tied-back soldier pile and concrete lagging wall is utilized, the configuration of the existing piles must be assessed to determine whether installation of soil anchors is feasible. Provision should be made in the Contract Documents for relocating / re-orientating the soil anchors if interference with the existing piles is encountered.
- Given the close proximity of the proposed retaining wall to the existing south abutment, it is recommended that vibration monitoring be carried out during pile installation. A Non-Standard Special Provision (NSSP) should be included in the Contract Documents to require the Contractor to monitor vibrations on the existing bridge and maintain the measured vibration levels below a peak particle velocity of 50 mm / second.
- Although no cobbles or boulders were encountered during the current drilling investigation and no such obstructions are listed on the logs from the 1957 investigation, it should be noted that cobbles and boulders are inherent in glacially-derived materials. Cobbles and boulders should therefore be expected during driving for the soldier pile installation, and during drilling for installation of permanent anchors. It is recommended that provision be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions.


5.7.3 Design and Construction Considerations for Mechanically-Reinforced Soil Systems

For the proposed wall height, construction of a mechanically-reinforced earth wall will involve removal of existing fill material (comprising the south abutment foreslope) for a distance of at least 3 m behind the wall, assuming a strip length approximately equal to the wall height. This involves excavations which will be within about 3 m of the eastern end of the south abutment. In considering this option, the structural designers must ensure that the stability of the existing bridge abutment and its pile cap will not be compromised by the short-term removal of this material.


GOLDER ASSOCIATES LTD.


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SEMP/LCC/ASP/FJH/clg
S:\FINAL\DAT\PROJECTS\2000\001-1142\2001\001142WALLRPTA01.DOC

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

Consistency

	<u>kPa</u>	<u>psf</u>
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

(b) Cohesive Soils

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.

S:\FINAL\DATA\ABBREV\2000\LOFA-D00.DOC

LIST OF SYMBOLS

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

I GENERAL

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

II STRESS AND STRAIN

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

III SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
*	Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity Index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

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

(d) Consolidation (one-dimensional)

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

(e) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1. $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = (Compressive strength)/2

PROJECT 001-1142		RECORD OF BOREHOLE No 11		1 OF 2	METRIC
W.P. 274-96-00		LOCATION N 4870453.41; E 401926.58		ORIGINATED BY SP	
DIST 41 HWY 401		BOREHOLE TYPE 114mm Solid Stem Augers		COMPILED BY SP	
DATUM Geodetic		DATE Oct. 10/00		CHECKED BY LCC	

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						
104.58	GROUND SURFACE						20 40 60 80 100	20 40 60 80 100	10 20 30					
0.00	Clayey Silt with sand, trace gravel Firm to very stiff Brown Moist (Till) Trace roots and grass from ground surface to 0.3m depth.		1	SS	4									
			2	SS	17									
			3	SS	29									
			4	SS	28									
101.68														
2.90	Silty Sand, some gravel, trace clay (non-plastic) to clayey silt with sand, some gravel Compact Grey Moist (Till)		5	SS	15									
			6	SS	17									
98.98														
5.60	Clayey Silt, some to with sand, trace gravel Stiff to very stiff Grey Moist (Till)		7	SS	11									
			8	SS	17									
			9	SS	23									
94.48														
10.10	Silty Clay Very stiff Grey Moist		10	SS	15									
92.88														
11.70	Clayey Silt with sand, trace gravel Very stiff Grey Moist to wet (Till)		11	SS	18									
91.38														
13.20	Sand and Gravel Very dense Grey Wet		12	SS	60/15									
89.98														
14.60														

ON MOT 001-1142.GPJ ON MOT.GDT 9/1/01

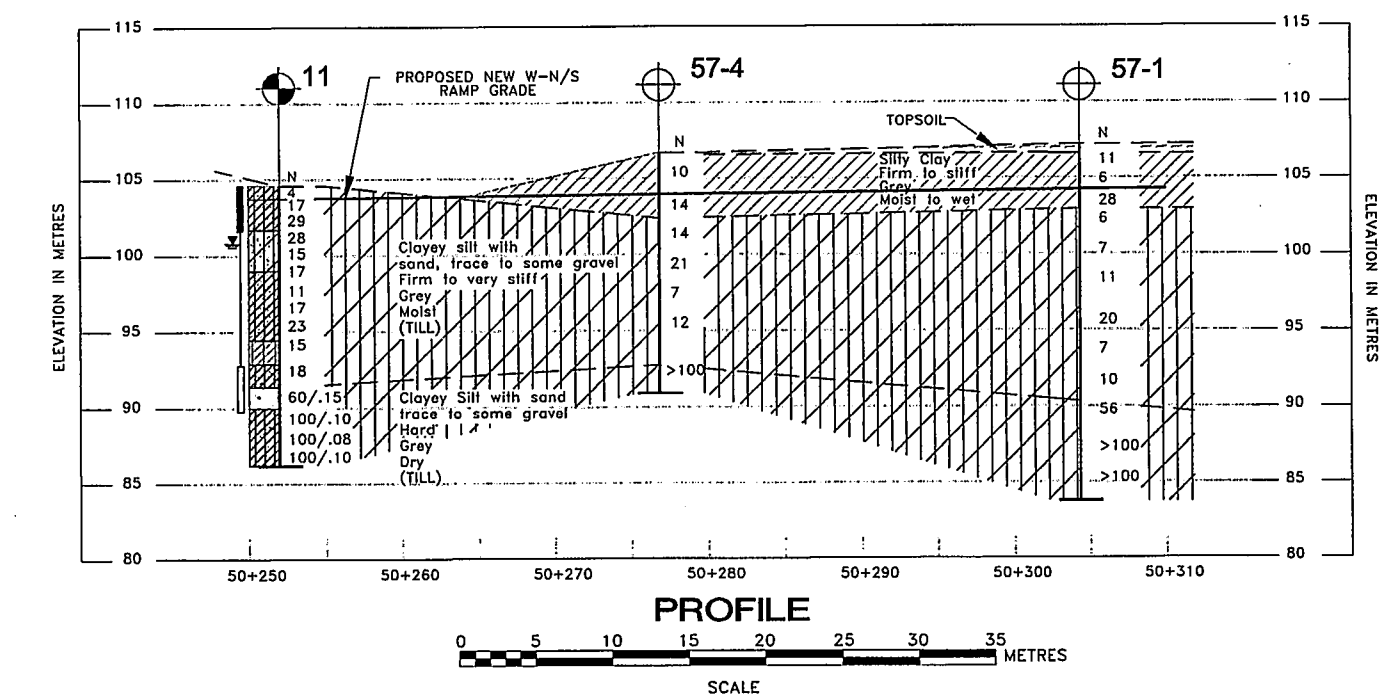
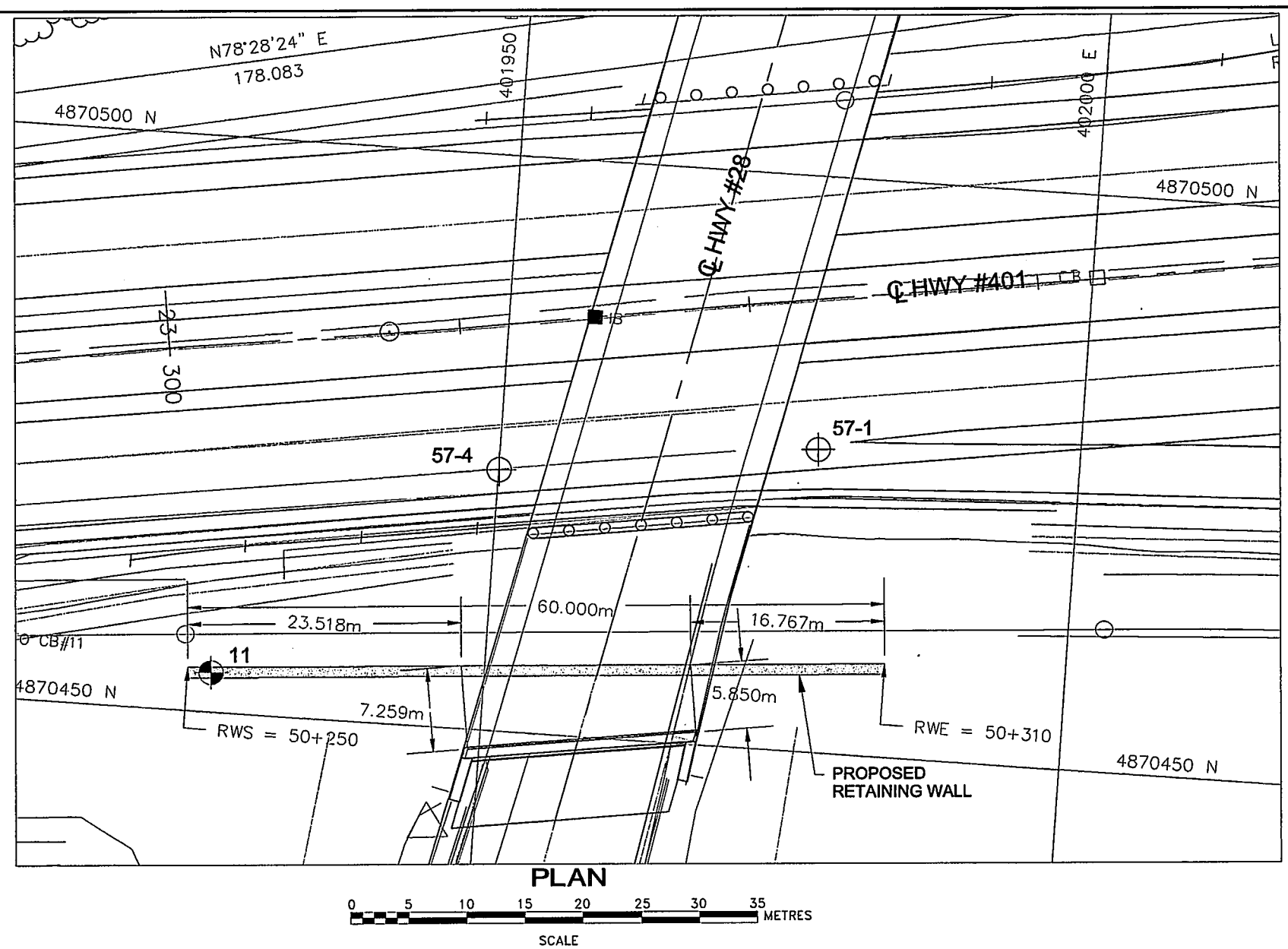
Continued Next Page

+ 3, X 3. Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT <u>001-1142</u>				RECORD OF BOREHOLE No 11				2 OF 2		METRIC							
W.P. <u>274-96-00</u>				LOCATION <u>N 4870453.41; E 401926.58</u>				ORIGINATED BY <u>SP</u>									
DIST <u>41</u> HWY <u>401</u>				BOREHOLE TYPE <u>114mm Solid Stem Augers</u>				COMPILED BY <u>SP</u>									
DATUM <u>Geodetic</u>				DATE <u>Oct 10/00</u>				CHECKED BY <u>LCC</u>									
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 (%)
--- CONTINUED FROM PREVIOUS PAGE ---																	
	Cleyey Silt with sand, trace gravel Hard Grey Dry (Till)		13	SS	1000/10												
86.19																	
18.39	END OF BOREHOLE																
	Note: 1. Water level measured in piezometer at 3.8m depth (Elev. 100.78m) on Oct.23, 2000.																

ON MOT 001-1142.GPJ ON MOT.GDT 9/1/01



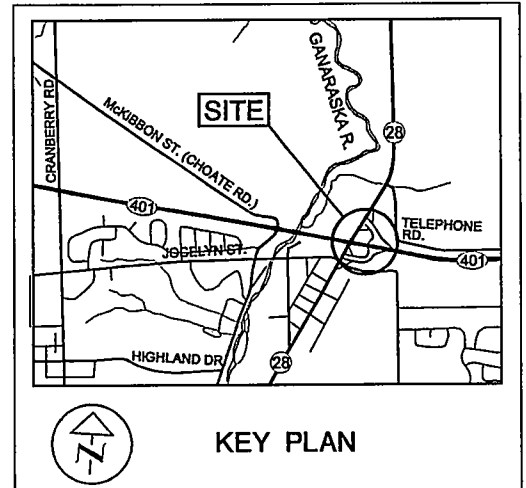
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN







DIST No. 41 HWY 401
CONT No.
GWP No. 274-96-00

HIGHWAY 401 AT HWY 28
RETAINING WALL
BOREHOLE LOCATIONS & SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



LEGEND			
	Borehole - Current Golder Associates Ltd. Investigation		
	Approximate location of boreholes from 1957 MTO investigation		
	Seal		
	Piezometer		
N	Blows/0.3m (Std. Pen. Test, 475 j/blow)		
	WL in piezometer on October 23, 2000		
	WL upon completion of drilling		
LOCATION			
No.	ELEVATION	NORTHINGS	EASTINGS
11	104.58	4870453.41	401926.58
57-1	107.2	4870476.34	401977.38
57-4	106.7	4870472.70	401950.04

NOTE
Locations for Borehole 57-1 and 57-4 estimated from drawing included in GEOCRES No. 30M16-8 Report, dated 1957.

NOTE
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REFERENCE
This drawing was created from digital files "PL-RETWALL.dwg" and "PR-RETWALL.DWG" dated October 25, 2000 provided by The Greer Galloway Group Inc.

NO.	DATE	BY	REVISION

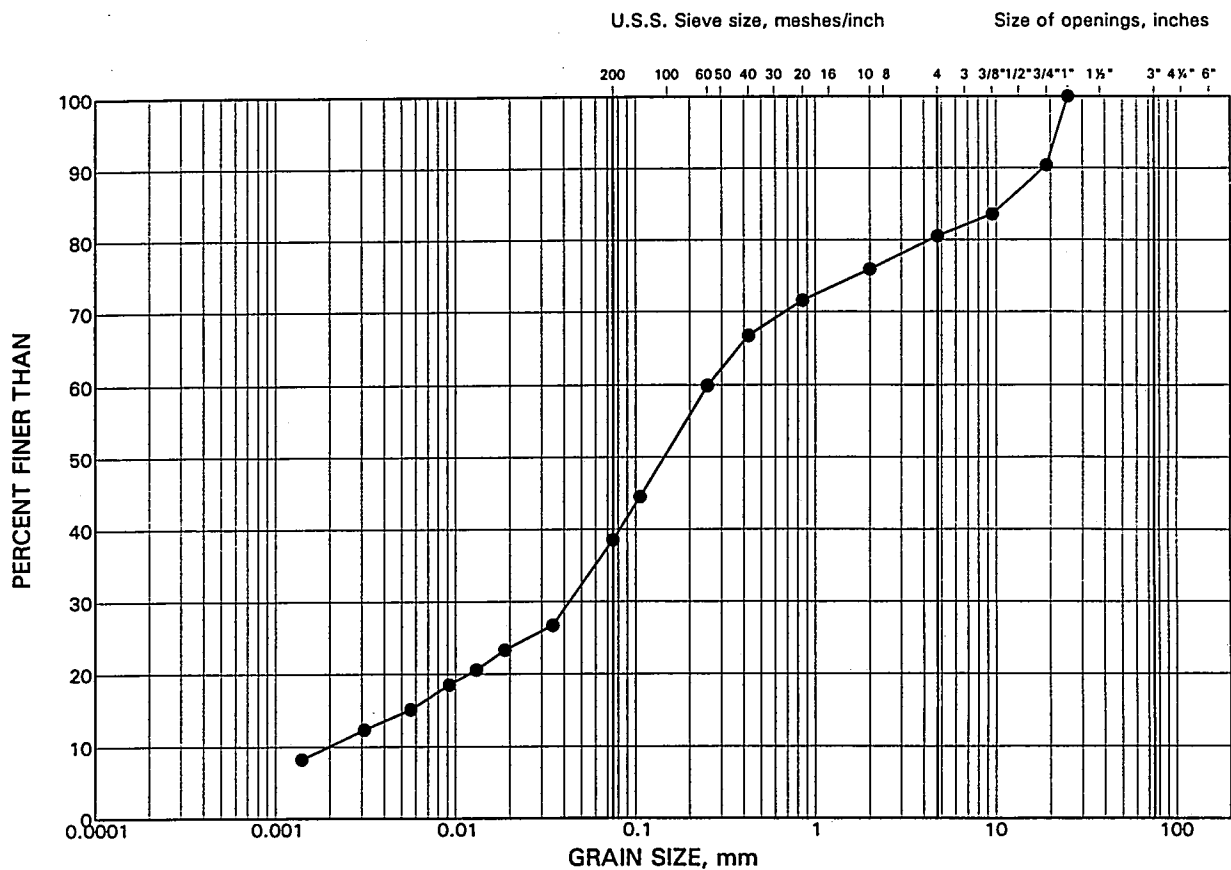
Geocres No.

HWY 401	PROJECT NO.: 001-1142	DIST. 41
SUBM'D. SP	CHKD: LCC	DATE: 2001 01 09
DRAWN: JFC	CHKD: LCC	APPD: ASP
		DWG. 1

01142A01.DWG

GRAIN SIZE DISTRIBUTION SILTY SAND TILL (NON-PLASTIC)

FIGURE 1

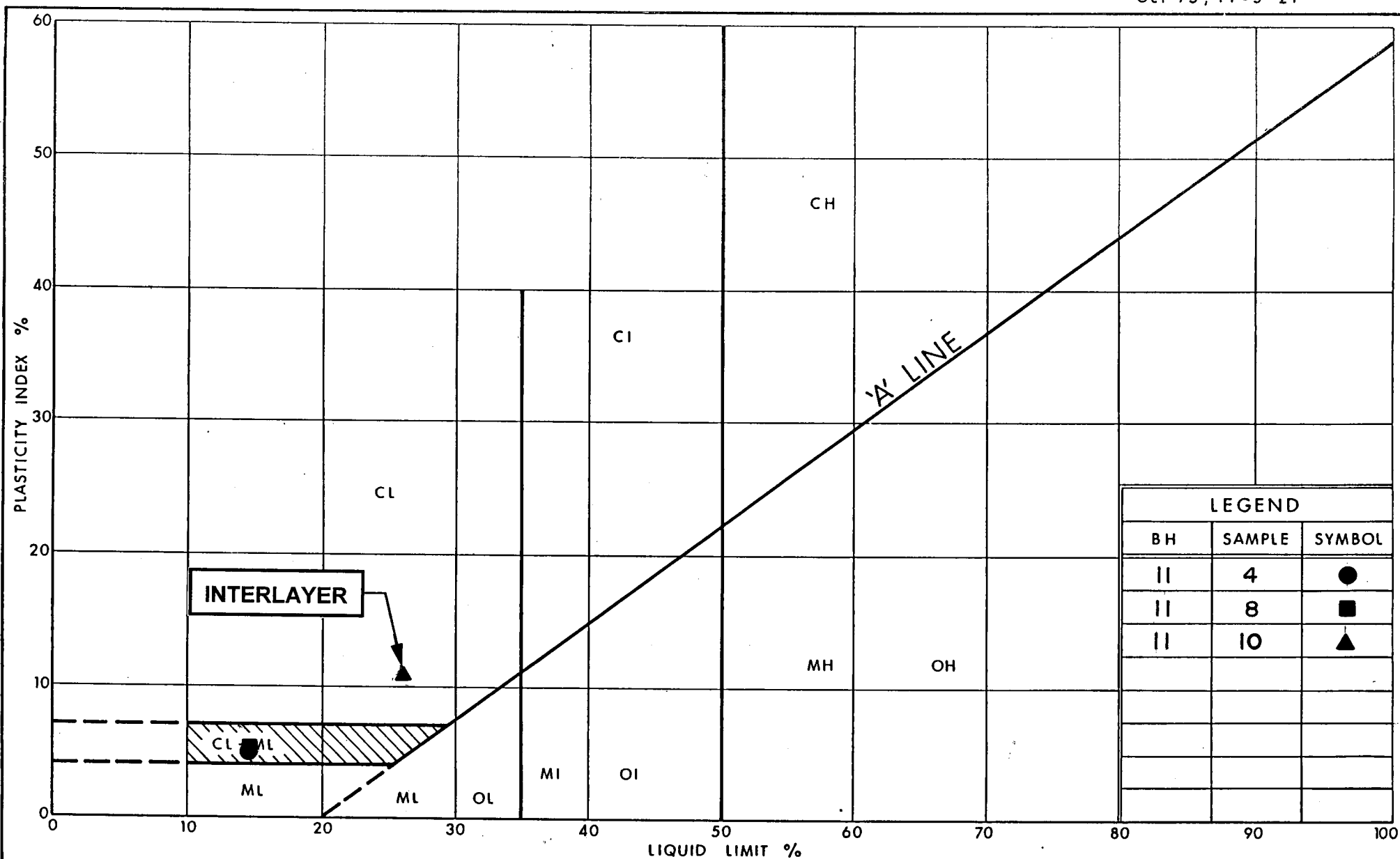


SILT AND CLAY SIZES			FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED			SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	11	6	100.0

Oct 75, FF-S-21



Ministry of
Transportation
Ontario

PLASTICITY CHART CLAYEY SILT (TILL) AND SILTY CLAY INTERLAYER

FIG No 2

W P 274 - 96 - 00

January 2001

001-1142A

APPENDIX A
RECORD OF BOREHOLE SHEETS
1957 MTO INVESTIGATION

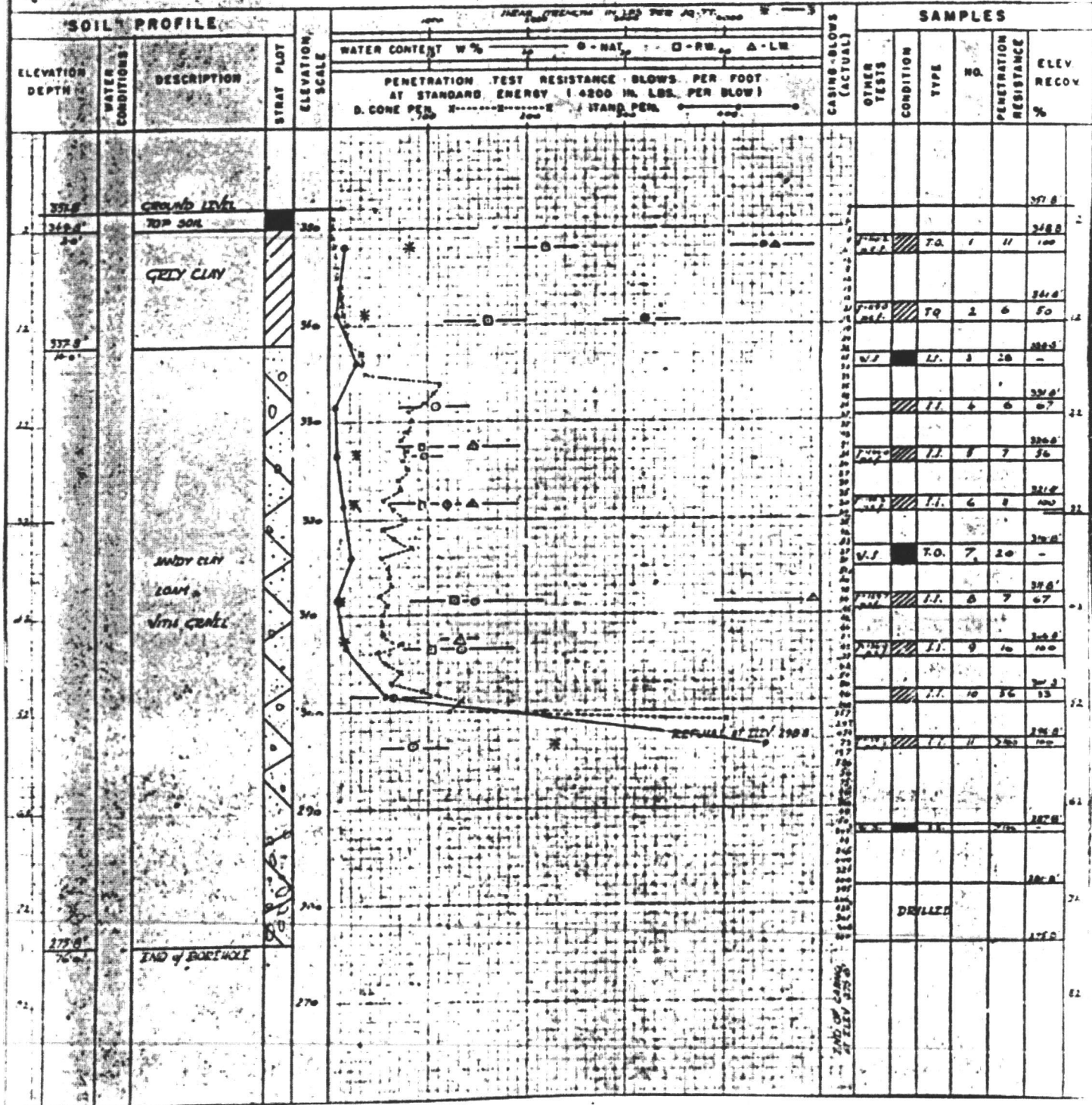
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-1 OPERATION BORT & FINITN JOB 7-57-7 WP 44-57 BORING 1 STA 530.37 (AS TO 27' 1000)
CASINGS 21 (standard samplers to fit unless noted) DATUM CHODER DATE REPORT JULY 1957
SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY H.J. CHECKED BY AL DATE BORING 25 APRIL 1957


ABBREVIATIONS
V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMEABILITY CS - CHURK
M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION DG - DRIVE OPEN
U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING DF - DRIVE FOOT VALVE
C - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL T - UNIT WEIGHT TQ - THIN WALLED OPEN
RC - ROCK CORE

SAMPLE TYPES
SS - SLEEVE SAMPLE
PS - PISTON SAMPLE
WS - WASHED SAMPLE
RC - ROCK CORE

SAMPLE CONDITION
- DISTURBED
- FAIR
- GOOD
- LOST



DRILL RIG 54-1 OPERATION BORIS PENLIN JOB F-57-7 WP 4-57 BORING 4 STA 432
CASING B1 (standard samplers to fit unless noted) DATUM CHOTIC DATE REPORT JULY 1957
SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY HJ CHECKED BY AL DATE BORING 4 MAY 1957

ABBREVIATIONS				SAMPLE TYPES	SAMPLE CONDITION
V - INSITU VANE SHEAR TEST	Q - TRIAXIAL QUICK	R - PERMEABILITY	CS - CRUSH	SS - SLEEVE SAMPLE	 <ul style="list-style-type: none"> - DISTURBED - FAIR - GOOD - BEST
N - MECHANICAL ANALYSIS	S - TRIAXIAL SLOW	C - CONSOLIDATION	DO - DRIVE OPEN	PS - PISTON SAMPLE	
U - UNCONFINED COMPRESSION	WL - WATER LEVEL IN CASING	CS - CASING	DF - DRIVE FOOT VALVE	WS - WASHED SAMPLE	
QC - TRIAXIAL CONSOLIDATED QUICK	WT - WATER TABLE IN SOIL	γ - UNIT WEIGHT	TO - THIN-WALLED OPEN	RC - ROCK CORE	

[illegible]

**FOUNDATION
INVESTIGATION
REPORT**

CONTRACT NO. 2001- 4012

FOR INFORMATION PURPOSES ONLY

Golder Associates Ltd.

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Mississauga, Ontario, Canada L5N 5S3
Telephone (905) 567-4444
Fax (905) 567-6561



REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
RETAINING WALL AT HIGHWAY 28
HIGHWAY 401 WIDENING
NORTHUMBERLAND COUNTY
G.W.P. 274-96-00, AGREEMENT NO. 4005-A-000103**

Submitted to:

**The Greer Galloway Group Inc.
973 Crawford Drive
Peterborough, Ontario
K9J 3X1**

DISTRIBUTION:

- 4 Copies - Ministry of Transportation, Ontario,
Downsview, Ontario**
- 3 Copies - The Greer Galloway Group Inc.,
Peterborough, Ontario**
- 2 Copies - Golder Associates Ltd.,
Mississauga, Ontario**

January 2001

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Figures 1 and 2
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January 2001

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PART A

**FOUNDATION INVESTIGATION REPORT
RETAINING WALL AT HIGHWAY 28
HIGHWAY 401 WIDENING
NORTHUMBERLAND COUNTY
G.W.P. 274-96-00, AGREEMENT NO. 4005-A-000103**

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1.0 INTRODUCTION

Golder Associates Ltd. has been retained by The Greer Galloway Group Inc. (Greer Galloway) on behalf of the Ministry of Transportation, Ontario (MTO) to provide detailed foundation investigation and design services for the widening of Highway 401 between former Highway 2 and Highway 28 near Port Hope, Ontario. The foundations engineering component of the project includes a proposed retaining wall at Highway 28 to accommodate a new W-N/S Ramp, and a permanent cut section between Cranberry Road and Choate Road along the median of Highway 401. This report addresses the proposed retaining wall at Highway 28.

The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed structure by drilling boreholes, and carrying out in-situ tests and laboratory tests on selected samples. Existing subsurface data from a report prepared by the Ministry of Transportation, Ontario ("Foundation Report on Underpass Bridge at New Highway 401 Crossing Highway 28, 1 Mile North of Port Hope", GEOCREC No. 30M16-8, dated 1957) was used to supplement the data obtained in the current investigation.

The terms of reference for the scope of work are outlined in our Total Project Management proposal P01-1216, dated July 2000. The work was carried out in accordance with our Quality Control Plan for Foundation Design Services, dated August 2, 2000.

The proposed preliminary alignment for the retaining wall was provided to us in digital format by Greer Galloway on October 25, 2000.

2.0 SITE DESCRIPTION

The proposed retaining wall is located parallel to and on the south side of the existing Highway 401, between the south pier and south abutment of the existing Highway 28 underpass structure. The site is located in the Town of Port Hope, in Northumberland County.

Highway 401 has been constructed in cut at the location of the Highway 28 underpass. The existing Highway 28 underpass is a three-span structure supported on steel H-piles, with the underside of the pile cap at the south abutment located at about Elevation 103.9 m, at the existing Highway 401 grade. These founding conditions were determined from the general layout drawing for the existing structure, provided by Greer Galloway (Drawing No. D4001-1, titled "Hope Township Bridge No. 18, Highway 401 Underpass, General Layout", dated May 1958).

Within the project limits, the vegetation cover consists of grass, bushes and small trees.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out on October 10, 2000. At this time one borehole – Borehole 11 – was advanced within the limits of the proposed retaining wall, near the west end of the proposed structure. Also referenced in this report are Boreholes 1 and 4 from the 1957 subsurface investigation carried out by the Ministry of Transportation (GEOCREs No. 30M16-8, referenced in Section 1.0). Boreholes 1 and 4 were drilled for the south abutment of the then-proposed Highway 28 bridge, and were located on the east and west sides of Highway 28, respectively.

The investigation was carried out using a bombardier-mounted CME-55 drill rig supplied and operated by Eastern Soil Investigation Ltd. of Courtice, Ontario. Borehole 11 was extended to a depth of 18.4 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration test (SPT) procedure. Groundwater conditions in the open borehole were observed throughout the drilling operations. A piezometer, consisting of a 3 m long slotted section threaded onto 12 mm diameter rigid PVC tubing, was installed in the borehole to permit monitoring of the groundwater level at the site.

The borehole caved after withdrawal of the augers to within about 3 m of ground surface. Above this level, the borehole was backfilled using bentonite pellets. The borehole location will be inspected periodically to monitor for backfill settlement.

The field work was supervised on a full-time basis by a member of our engineering staff who located the borehole in the field, directed the drilling, sampling, and in-situ testing operations, and logged the borehole. The soil samples were identified in the field, placed in labelled containers and transported to our laboratory in Mississauga for further examination. Index and classification tests consisting of water content determinations, Atterberg Limits tests and grain size analyses were carried out on selected samples.

The retaining wall location was surveyed and staked in the field by personnel from Greer Galloway, in order to locate the borehole on the retaining wall alignment. The ground surface elevation at Borehole 11 was surveyed by Greer Galloway and is referenced to the geodetic datum. The locations of the boreholes are shown on Drawing 1.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located on the margin of the physiographic region known as the Iroquois Plain. The surficial soils within this region are comprised of sands, silts and clays deposited by glacial Lake Iroquois. To the north of the Port Hope area, these lacustrine deposits are interrupted by drumlins comprised of till soils, which stood as islands in glacial Lake Iroquois. (Reference: "The Physiography of Southern Ontario", 3rd Edition, Chapman and Putnam, 1984.).

Limestone bedrock was encountered at about Elevation 90 m in the Ganaraska River valley, about 200 m west of Highway 28, in a 1957 subsurface investigation by E.M. Peto Associates Ltd. for the Ministry of Transportation. (Reference: "Report on Soil and Foundation Conditions, Highway 401 - Ganaraska River Crossing, Hope Township Bridge 16, W.P. 757-56, GEOCRE No. 30M16-7).)

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions encountered in Borehole 11 together with the results of laboratory testing carried out on selected soil samples are given on the attached Record of Borehole sheet and Figures 1 and 2. The detailed subsurface conditions encountered in the 1957 boreholes are given on the Record of Borehole sheets in Appendix A. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils at the proposed retaining wall location consist of a thin layer of topsoil overlying a clayey silt till deposit which contains interlayers / lenses of silty clay and sand and gravel. The upper portion of the clayey silt till has a generally stiff to very stiff consistency, while the lower portion of the deposit is hard. Bedrock was not encountered in Borehole 11 or in the boreholes advanced during the 1957 investigation. It is noted that the boreholes drilled in 1957 encountered a 3.5 m to 4 m thick layer of firm to stiff silty clay overlying the till deposit; this firm to stiff material was not encountered in Borehole 11.

The locations and elevations of the borings, together with the interpreted stratigraphic profile along the proposed retaining wall, are shown on Drawing 1. A detailed description of the subsurface conditions encountered in the boreholes for the current and 1957 investigations is provided in the following sections.

4.2.1 Topsoil

The upper 300 mm of soil in Borehole 11 contained organic matter. Boreholes 1 and 4, drilled in 1957, encountered between 300 mm and 600 mm of topsoil.

4.2.2 Silty Clay

Boreholes 1 and 4 from the 1957 investigation encountered a 3.5 m to 4 m thick layer of grey silty clay below the topsoil, overlying the clayey silt till, between about Elevations 106.5 m and 102.5 m. This layer was not encountered in Borehole 11 during the current investigation, due in part to the cut for construction of the Highway 28 underpass.

Measured Standard Penetration Test (SPT) 'N' values in this layer ranged from 6 to 14 blows per 0.3 m of penetration, and laboratory unconfined compression testing measured shear strengths of about 20 kPa to 60 kPa. These test results indicate that this silty clay layer generally has a firm to stiff consistency.

Measured water contents for these samples ranged from 11 to 44 per cent. Atterberg limits testing indicated a plastic limit of 10 to 22 per cent, and a plasticity index of 7 to greater than 34 per cent, indicating that this silty clay is inorganic and of high plasticity. Measured water contents ranged from 11 to 44 per cent.

4.2.3 Clayey Silt Till

A clayey silt till deposit was encountered below Elevation 104.3 m in Borehole 11, and below about Elevation 102.5 m in Boreholes 1 and 4. It is noted that the records for Boreholes 1 and 4 described this deposit as a sandy clay loam with gravel; the results of in-situ and laboratory testing indicate that this loam would be described as clayey silt till according to MTO's current soil classification system. The clayey silt till contains a high percentage of sand, and trace to some

gravel. In Borehole 11, a 2.7 m thick layer containing non-plastic silty sand till was encountered within the predominantly cohesive till deposit; the grain size distribution test result for a sample of this material is shown on Figure 1.

Above Elevations 91 m to 91.5 m, the measured SPT 'N' values in the three boreholes ranged from 4 to 29 blows per 0.3 m of penetration, but were typically less than 20 blows per 0.3 m of penetration. Laboratory unconfined compression testing measured shear strengths of about 10 kPa to 20 kPa in Borehole 1, as shown on the Record of Borehole sheet. These test results indicate that this portion of the till deposit is firm to very stiff, but predominantly stiff to very stiff. Below Elevation 91 m to 91.5 m, the measured SPT 'N' values were generally greater than 100 blows per 0.3 m of penetration, indicating a hard consistency.

Atterberg limits testing carried out on selected clayey silt till samples indicate that it is of low plasticity with plastic limits of 9 to 13 per cent, liquid limits of 13 to 15 per cent, and a plasticity index between 3 and 5 per cent; the results of the Atterberg testing are shown on Figure 2 and on the Record of Borehole sheets. The measured natural water contents ranged from about 8 to 13 per cent, typically near the plastic limit of the soil.

Between about Elevation 95 m and 93 m in all boreholes, a thin interlayer of silty clay was encountered or inferred from laboratory testing; the measured plastic and liquid limits of this interlayer were about 12 to 15 and 20 to 49 per cent, respectively, with measured water contents ranging from 15 to 26 per cent.

In Borehole 11, a 1.4 m thick interlayer of water-bearing, grey sand and gravel was encountered at about Elevation 91.4 m, immediately overlying the hard clayey silt till. The SPT 'N' value was greater than 100 blows per 0.3 m of penetration indicating a very dense state of packing. Boreholes 1 and 4 show refusal to dynamic cone penetration testing at about Elevation 91.1 m which likely corresponds to this interlayer.

4.3 Groundwater Conditions

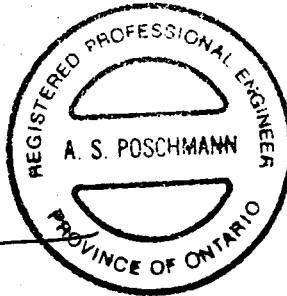
Water levels were not recorded for Boreholes 1 and 4, drilled in the 1957 investigation. A piezometer was installed in Borehole 11, and the water level was measured on October 23, 2000 at 3.8 m depth (approximately Elevation 100.8 m).

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to be higher during wet periods of the year.

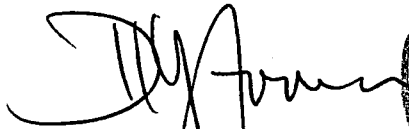
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PART B

**FOUNDATION DESIGN REPORT
RETAINING WALL AT HIGHWAY 28
HIGHWAY 401 WIDENING
NORTHUMBERLAND COUNTY
G.W.P. 274-96-00, AGREEMENT NO. 4005-A-000103**

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides recommendations on the geotechnical aspects of design of the proposed retaining wall on the south side of Highway 401, based on interpretation of the factual information obtained during the current and 1957 investigations. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made regarding 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 method and scheduling.

It is understood that the W-N/S ramp will be shifted southward to pass between the south pier and south abutment of the existing Highway 28 underpass, in order to accommodate the widening of Highway 401. The grade of both the existing Highway 401 and the proposed W-N/S ramp is about Elevation 104 m. This will require that the south abutment foreslope be cut; a retaining wall is proposed to maximize the space available for the new ramp. It is understood that the front face of the proposed retaining wall is to be located 5.9 m to 7.3 m north of the south abutment face. It is further understood that the wall is to be about 60 m in length, and that it will have a maximum height of approximately 4 m.

Based on the general layout drawing provided by Greer Galloway for the existing Highway 28 underpass structure (Drawing No. D4001-1, titled "Hope Township Bridge No. 18, Highway 401 Underpass, General Layout", dated May 1958), the bridge abutments and piers are founded on steel H-piles driven to about Elevations 90.5 m to 89.5 m. The underside of the pile cap at the south abutment was designed to be at about Elevation 103.9 m. It is noted that the front two rows of piles at the south abutment were designed to be battered at 3 vertical to 1 horizontal.

5.2 Retaining Wall Options

The subsoils at the proposed retaining wall location consist of a thin surficial layer of topsoil overlying clayey silt till. The till is generally stiff to very stiff above Elevation 91 m, and hard

below that elevation. In the 1957 boreholes, drilled prior to the highway cut, a 3.5 m thick layer of silty clay was present atop the clayey silt till. It is anticipated that the proposed retaining wall will be founded on either shallow foundations placed on the predominantly stiff to very stiff clayey silt till, or on deep foundations (steel H-piles) driven to practical refusal on the hard clayey silt till below about Elevation 91 m. It is understood that three concepts are under consideration for the proposed retaining wall:

- a soldier pile and concrete panel wall;
- a mechanically-reinforced soil retaining wall system; and
- a cast-in-place concrete cantilever retaining wall.

Foundation engineering recommendations are provided for each of these types of retaining systems, and construction considerations applicable to the types of foundation systems are provided in the following subsections.

5.3 Soldier Pile and Concrete Panel Wall

A soldier pile and concrete panel wall would consist of steel H-piles driven to the hard clayey silt till at about Elevation 91 m, approximately 13 m below the proposed W-N/S Ramp grade. Lateral support to the soldier pile and concrete panel wall system would be in the form of permanent soil anchors. Suitable drainage and insulation should be provided to the back of this type of wall which will otherwise be formed against the in-situ clayey silt till deposit and fill materials.

5.3.1 Axial Geotechnical Resistance

For design, the factored axial resistance at Ultimate Limit States (ULS) for steel HP 310 x 110 H-piles driven to practical refusal on the hard clayey silt till below Elevation 91 m may be taken as 1,600 kN. The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 1,100 kN. To achieve these design resistances, the piles should be driven to at least Elevation 91 m and to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.

The contract drawing for the wall foundations should include the following note with respect to the piles:

"Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,200 kN per pile but must be driven below Elevation 91 m."

5.3.2 Soil Anchor Design

Soil anchors may be necessary for lateral restraint of the wall. These permanent anchors should be provided with suitable corrosion protection.

The sustained working load should not be greater than 60 per cent of the ultimate tensile strength of the anchor tendons or bars. The anchors may be sized based on the following ultimate bond stresses acting between the grout and soil:

Firm to very stiff clayey silt till above Elevation 91 m	40 kPa
Hard clayey silt till below Elevation 91 m	120 kPa

The above ultimate bond stresses assume a fixed anchor length (bond zone) of not greater than 8 m. The fixed length of the anchor should be maintained behind a line drawn upward at 45 degrees from the base of the piles. In addition, the fixed length should be maintained outside of the area of existing piles supporting the bridge abutment.

Because the ground-to-anchor bond in soil is highly dependent upon the installation technique, the Contractor should be held to an anchor performance specification enforced by proof tests on all anchors and a performance test on at least one anchor. The Contract Documents should incorporate a Non-Standard Special Provision (NSSP) to this end. Anchor installation and testing should be carried out under the full-time inspection of a geotechnical engineer. A performance test should be carried out, to 200 per cent of the design working load, on at least one anchor to confirm the design and the Contractor's installation method. In addition, each anchor should be proof-tested to 125 per cent of the working load. The tensile stress in the anchor bar or strands during test loading should not exceed 80 per cent of the guaranteed ultimate tensile strength of the bar or strands.

5.3.3 Resistance to Lateral Loads

The horizontal resistance for the soils in front of an individual pile can be estimated using the following equation:

$$k_h = n_h z / d$$

where:

k_h = coefficient of horizontal subgrade reaction (MPa/m)

d = pile width (m)

$n_h z$ = constant of horizontal subgrade reaction (MPa) = 15 MPa for the firm to very stiff clayey silt till deposit (Note: for these cohesive soils, $n_h z$ is equivalent to $k_{s1}/5$)

5.4 Mechanically-Reinforced Soil Retaining Wall System (Retained Soil Systems)

A mechanically-reinforced soil retaining wall system (Retained Soil System or RSS wall) consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material. Use of this system is considered appropriate for construction of the proposed W-N/S Ramp retaining wall, given that the maximum wall height will be about 3.5 m. The founding stratum is variable along the length of the wall, with very stiff clayey silt till at the west end and firm to very stiff clayey silt till along the remainder; this will result in some differential settlement along the wall of the order of 25 mm, which would be within the allowable limits for this system.

The reinforced earth mass will be founded on firm to very stiff, but predominantly stiff to very stiff clayey silt till assuming it will be placed at a nominal depth below the proposed W-N/S Ramp grade. For the reinforced earth mass founded on this clayey silt till, a factored geotechnical resistance at ULS of 250 kPa may be assumed for design. The geotechnical resistance at SLS will be governed by the actual dimensions of the RSS wall. It is recommended that the existing ground surface grades behind the wall be maintained. Provided that the top of the wall is no higher than the existing grade, the loading on the subsoils will not induce significant settlements and a geotechnical resistance at SLS of 150 kPa may be assumed. A coefficient of friction equal to 0.55 may be assumed between the granular fill of the RSS wall and the clayey silt till founding soils.

The internal stability of the mechanically-reinforced soil wall should be checked by the RSS supplier / designer. For the proposed height of wall and assuming level ground behind the wall, an adequate factor of safety against failure is obtained for the wall for global stability. The majority of the wall, however, will have sloping ground behind it and further analysis would be required to confirm global stability adequacy. If the RSS option is feasible, therefore, full details of the ground surface profiles behind and in front of the wall must be provided to complete the analysis.

The granular fill material for the reinforced soil system should be placed on native, undisturbed soils. In this regard, all topsoil, existing fill and soft or loose materials should be removed and the subgrade should be inspected by qualified geotechnical personnel prior to placement of the granular materials.

5.5 Cast-in-Place Concrete Cantilever Retaining Wall

For this option, the concrete cantilever wall may be supported on steel H-piles driven to found within the hard clayey silt till at approximately 13 m below the proposed W-N/S Ramp grade. Consideration has also been given to the use of shallow spread footings for support of the wall.

5.5.1 Shallow Foundations

Given the proposed W-N/S Ramp grade of Elevation 104 m, the highest acceptable founding elevation will be 102.8 m in order to provide a minimum of 1.2 m of earth cover for protection against frost penetration. Spread footings at this level will be placed on the clayey silt till which has variable consistency along the length of the wall. The factored geotechnical resistance at ULS which should be used for footing design at this founding elevation is 250 kPa.

The settlement of footings founded on this firm clayey silt till will be dependent on the footing size and configuration, and on the applied loads. For example, assuming spread footings founded at Elevation 102.8 m, it is expected that under an applied load of 70 kPa the settlement of the wall would be less than 12 mm. This is due to the fact that the original ground surface in this area was at about Elevation 107 m or some 2 m higher than the current ground surface. For a 3 m wide footing, it is expected that up to 25 mm of settlement would occur under an applied load of 150 kPa. Due to the sensitivity of the settlement to the footing size and founding elevation, a settlement analysis

should be undertaken if spread footings are a feasible option. The analysis should be based on the footing design and applied loads to determine appropriate design values for SLS conditions.

The geotechnical resistances provided are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Ontario Highway Bridge Design Code (OHBDC).

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with Section 6-8.4.3 of the OHBDC, assuming an unfactored coefficient of friction of 0.4 between the concrete and the clayey silt till founding soils.

5.5.2 Deep Foundations

5.5.2.1 Axial Geotechnical Resistance

Consideration could be given to supporting a cast-in-place concrete wall on steel H-piles driven to refusal on the hard clayey silt till at about Elevation 91 m, about 13 m below the proposed W-N/S Ramp grade. For design, the factored axial resistance at ULS for steel HP 310 x 110 H-piles driven to practical refusal may be taken as 1,600 kN. The axial resistance at SLS for 25 mm of settlement may be taken as 1,100 kN.

To achieve the above design resistances, the piles should be driven to at least Elevation 91 m and to a final set of no less than 15 blows per 25 mm of penetration using a hammer with rated energy of about 50 kJ, and not exceeding 60 kJ. Provision should be made to re-tap all piles to confirm the set after adjacent piles have been driven.

The contract drawing for the wall foundations should include the following note with respect to piles:

“Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,200 kN per pile but must be driven below Elevation 91 m.”

5.5.2.2 Resistance to Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. Where required, the horizontal reaction to the pile can be estimated using the following equation:

$$k_h = n_h z / d$$

where:

- k_h = coefficient of horizontal subgrade reaction (MPa/m)
- d = pile width (m)
- $n_h z$ = constant of horizontal subgrade reaction (MPa) = 15 MPa for the firm to very stiff clayey silt till (Note: for these cohesive soils, $n_h z$ is equivalent to $k_{s1}/5$)

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R as follows:

<i>Pile Spacing in Direction of Loading D = Pile Diameter</i>	<i>Subgrade Reaction Reduction Factor R</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

5.5.2.3 Frost Protection

The pile caps should be provided with 1.2 m of soil cover for frost protection.

5.6 Lateral Earth Pressures for Design

The lateral pressures acting on the proposed retaining wall will depend on the native soils and, where used, the type and method of placement of the backfill materials behind the wall, as well as on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the retaining wall in accordance with OHBDC:

- Where fill is to be placed behind the proposed wall, select free-draining granular fill meeting the specifications of OPSS Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. All granular fill should be compacted in lifts of loose thickness not greater than 200 mm to 95 per cent of the material's Standard Proctor maximum dry density.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill.
- Where fill soils are placed and compacted behind the retaining wall using hand-operated equipment only, a compaction surcharge equal to 11 kPa should be included in the lateral earth pressures for structural design, in accordance with OHBDC (Commentary Section C6-7.4.3). Compaction equipment should be used in accordance with OPSS 501.06. This value of 11 kPa is given since small equipment only may be used at this site due to the anticipated restricted access between the wall and the bridge abutment. If it is expected that large compaction equipment will be used, a compaction surcharge equal to 16 kPa should be included.
- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the retaining wall (Case I) or within the wedge-shaped zone defined by a 60 degree line extending up and back from the bottom of the rear face of the footing (Case II).
- For a soldier pile and concrete lagging wall or for retaining structures backfilled as per Case I, the following parameters (unfactored) may be assumed:

Soil unit weight: 20 kN/m^3

Coefficients of lateral earth pressure:

'active', K_a 0.35
'at rest', K_o 0.50

- For retaining structures backfilled as per Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of lateral earth pressure:		
'active', K_a	0.27	0.31
'at rest', K_o	0.43	0.47

- If the wall support allows lateral yielding (unrestrained structure), active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding (restrained structure), at-rest pressures should be assumed for geotechnical design.

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. If sloping ground will be present above the wall, the coefficient of active earth pressure, K_a , must be adjusted. Appropriate values for K_a can be provided once the slope configuration is established.

5.7 Design and Construction Considerations

5.7.1 Design and Construction Considerations for Shallow Foundations and Pile Caps

At the proposed retaining wall location, excavation for shallow spread footings or for pile caps will extend a minimum of 1.2 m below the existing Highway 401 grade and will cut into the abutment foreslope, extending through existing fill, soft to stiff silty clay (if present) and firm to very stiff clayey silt till. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act for Construction Activities. The firm to very stiff clayey silt till soils at this site would be classified as Type 3 soil. Temporary open-cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, the excavation could also be carried out within a braced excavation or a trench box.

The clayey silt till soils comprising the excavation subgrade will be sensitive to disturbance from ponded water, construction traffic and frost. All foundation excavations should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that soil having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. Provision should be made to sub-excavate below founding level where unsuitable subgrade soils are encountered, and to replace the removed material with compacted granular soil or lean mix concrete.

Groundwater seepage into the excavations through the fill, clayey silt till and any lenses or interlayers within the till could occur, although this is expected to be minor. Pumping from

properly filtered sumps or a filtered drain located at the base of the excavation outside of the actual footing limits will provide adequate groundwater control during foundation excavations. Surface water run-off should be directed away from the excavation.

5.7.2 Design and Construction Considerations for Deep Foundations and Tied-Back Soldier Pile Wall

The following must be considered in order to ensure that deep foundations can be successfully installed at this site:

- Piles driven under the bridge will have to be spliced and special pile-driving machinery may be required in order to meet the height restrictions imposed by the vertical clearance under the existing Highway 28 structure.
- Given the location of the proposed wall in relation to the existing south abutment and wing walls – which are supported on piles, some of which are battered – there is a possibility of the piles for the new wall interfering with the existing piles. In addition, where a tied-back soldier pile and concrete lagging wall is utilized, the configuration of the existing piles must be assessed to determine whether installation of soil anchors is feasible. Provision should be made in the Contract Documents for relocating / re-orientating the soil anchors if interference with the existing piles is encountered.
- Given the close proximity of the proposed retaining wall to the existing south abutment, it is recommended that vibration monitoring be carried out during pile installation. A Non-Standard Special Provision (NSSP) should be included in the Contract Documents to require the Contractor to monitor vibrations on the existing bridge and maintain the measured vibration levels below a peak particle velocity of 50 mm / second.
- Although no cobbles or boulders were encountered during the current drilling investigation and no such obstructions are listed on the logs from the 1957 investigation, it should be noted that cobbles and boulders are inherent in glacially-derived materials. Cobbles and boulders should therefore be expected during driving for the soldier pile installation, and during drilling for installation of permanent anchors. It is recommended that provision be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions.


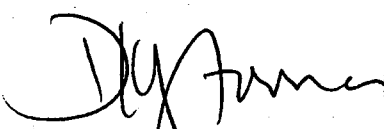
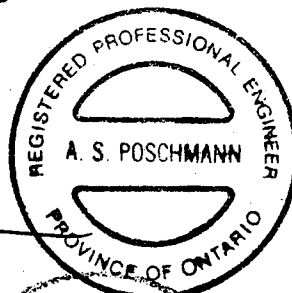
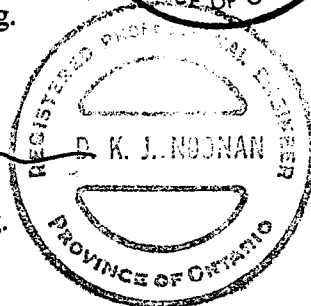
5.7.3 Design and Construction Considerations for Mechanically-Reinforced Soil Systems

For the proposed wall height, construction of a mechanically-reinforced earth wall will involve removal of existing fill material (comprising the south abutment foreslope) for a distance of at least 3 m behind the wall, assuming a strip length approximately equal to the wall height. This involves excavations which will be within about 3 m of the eastern end of the south abutment. In considering this option, the structural designers must ensure that the stability of the existing bridge abutment and its pile cap will not be compromised by the short-term removal of this material.

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LIST OF ABBREVIATIONS

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

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

(b) Cohesive Soils

Consistency	c_u, s_u	psf
	kPa	
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.

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LIST OF SYMBOLS

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

I GENERAL

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

II STRESS AND STRAIN

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

III SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)	

(a) Index Properties (con't.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity Index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_c	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

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

(d) Consolidation (one-dimensional)

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

(e) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1. $\tau = c' + \sigma' \tan \phi'$

2. Shear strength = (Compressive strength)/2

PROJECT 001-1142				RECORD OF BOREHOLE No 11				1 OF 2		METRIC				
W.P. 274-96-00				LOCATION N 4870453.41; E 401926.58				ORIGINATED BY SP						
DIST 41 HWY 401				BOREHOLE TYPE 114mm Solid Stem Augers				COMPILED BY SP						
DATUM Geodetic				DATE Oct 10/00				CHECKED BY LCC						
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						
104.58	GROUND SURFACE													
0.00	Clayey Silt with sand, trace gravel Firm to very stiff Brown Moist (TII)		1	SS	4									
	Trace roots and grass from ground surface to 0.3m depth.		2	SS	17									
			3	SS	29									
			4	SS	28									
101.58														
2.90	Silty Sand, some gravel, trace clay (non-plastic) to clayey silt with sand, some gravel Compact Grey Moist (TII)		5	SS	15									
			6	SS	17									
98.98														
5.60	Clayey Silt, some to with sand, trace gravel Stiff to very stiff Grey Moist (TII)		7	SS	11									
			8	SS	17									
			9	SS	23									
94.48														
10.10	Silty Clay Very stiff Grey Moist		10	SS	15									
92.88														
11.70	Clayey Silt with sand, trace gravel Very stiff Grey Moist to wet (TII)		11	SS	18									
91.38														
13.20	Sand and Gravel Very dense Grey Wet		12	SS	60/15									
89.98														
14.60														

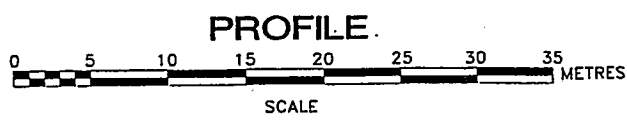
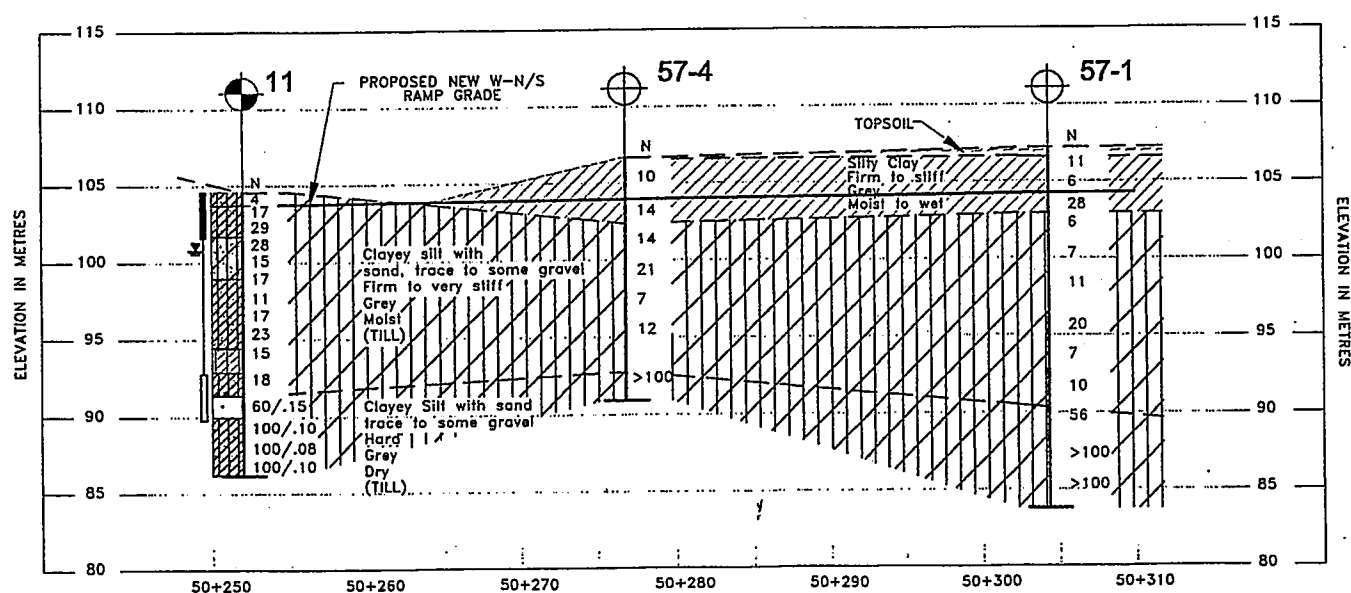
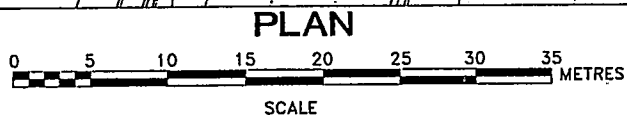
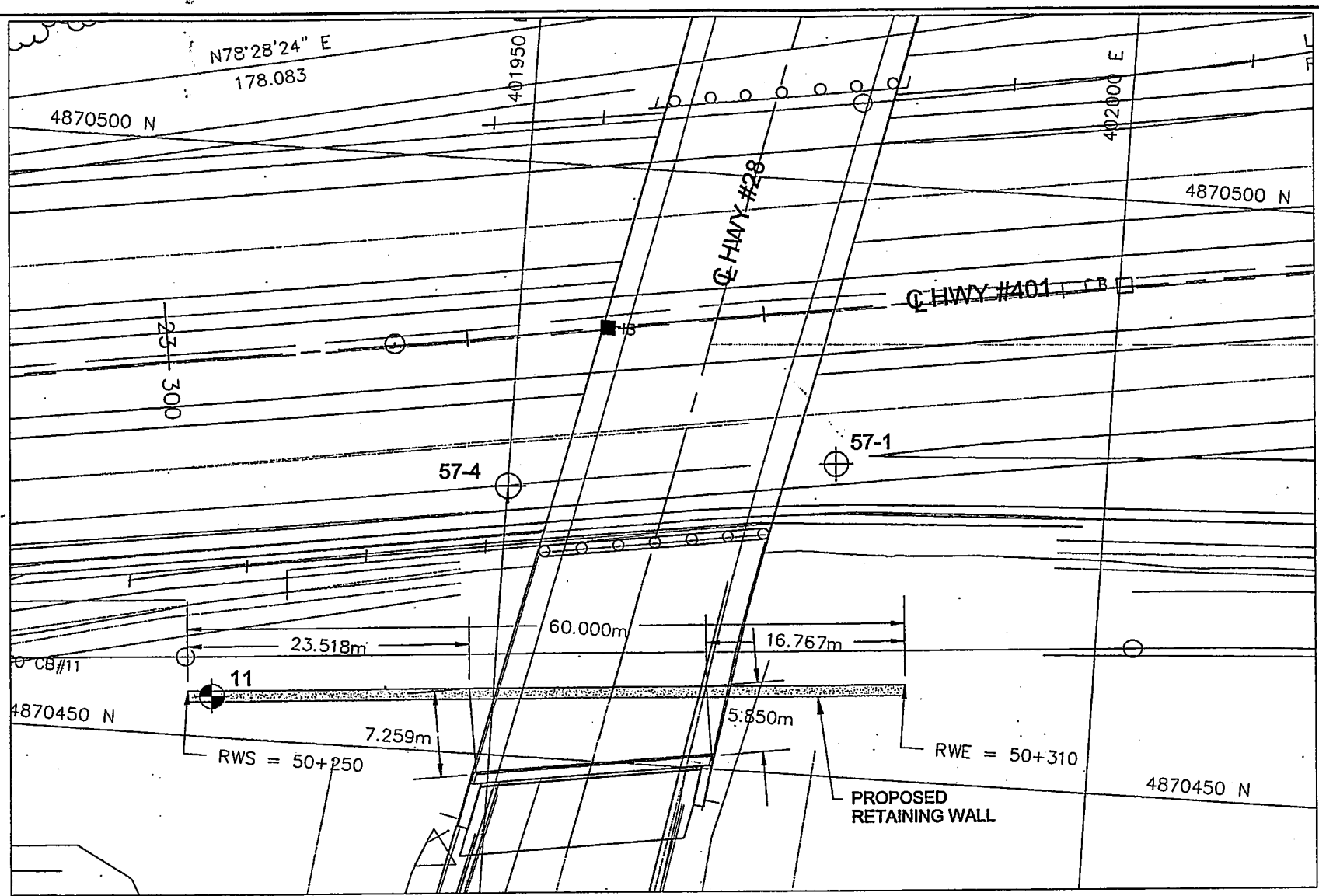
ON MOT 001-1142.GPJ ON MOT.GDT 8/1/01

Continued Next Page

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

RECORD OF BOREHOLE No 11										2 OF 2		METRIC					
PROJECT 001-1142			LOCATION N 4870453.41; E 401926.58			ORIGINATED BY SP											
W.P. 274-98-00			BOREHOLE TYPE 114mm Solid Stem Augers			COMPILED BY SP											
DIST 41 HWY 401			DATE Oct. 10/00			CHECKED BY LCC											
DATUM Geodetic																	
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	T _N VALUES			SHEAR STRENGTH kPa									
— CONTINUED FROM PREVIOUS PAGE —								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED				20 40 60 80 100 WATER CONTENT (%) 10 20 30					
	Clayey Silt with sand, trace gravel Hard Grey Dry (Till)		13	SS	100/0/0		89										
			14	SS	100/0/0		88										
			15	SS	100/0/0		87										
86.19 18.39	END OF BOREHOLE																
	Note: 1. Water level measured in piezometer at 3.8m depth (Elev. 100.78m) on Oct. 23, 2000.																

ON MOT 001-1142.GPJ ON MOT.GDT 9/1/01

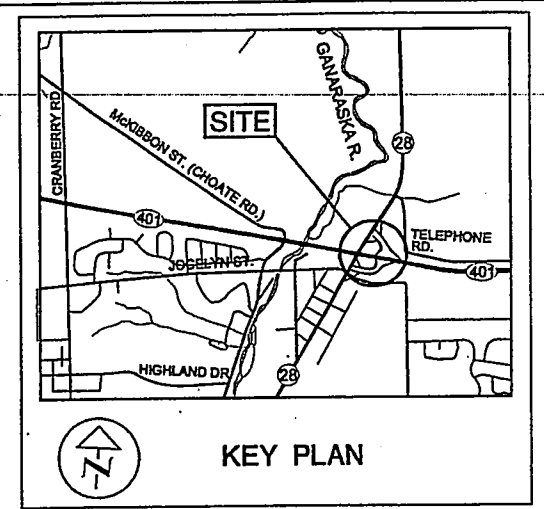


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN







DIST No. 41 HWY 401
CONT No.
GWP No. 274-96-00
HIGHWAY 401 AT HWY 28
RETAINING WALL
BOREHOLE LOCATIONS & SOIL STRATA



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND			
	Borehole - Current Golder Associates Ltd. Investigation		
	Approximate location of boreholes from 1957 MTO investigation		
	Seal		
	Piezometer		
N	Blows/0.3m (Std. Pen. Test, 475 j/blow)		
	WL in piezometer on October 23, 2000		
	WL upon completion of drilling		
LOCATION			
No.	ELEVATION	NORTHINGS	EASTINGS
11	104.58	4870453.41	401926.58
57-1	107.2	4870476.34	401977.38
57-4	106.7	4870472.70	401950.04

NOTE
Locations for Borehole 57-1 and 57-4 estimated from drawing included in GEOCREs No. 30M16-8 Report, dated 1957.

NOTE
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

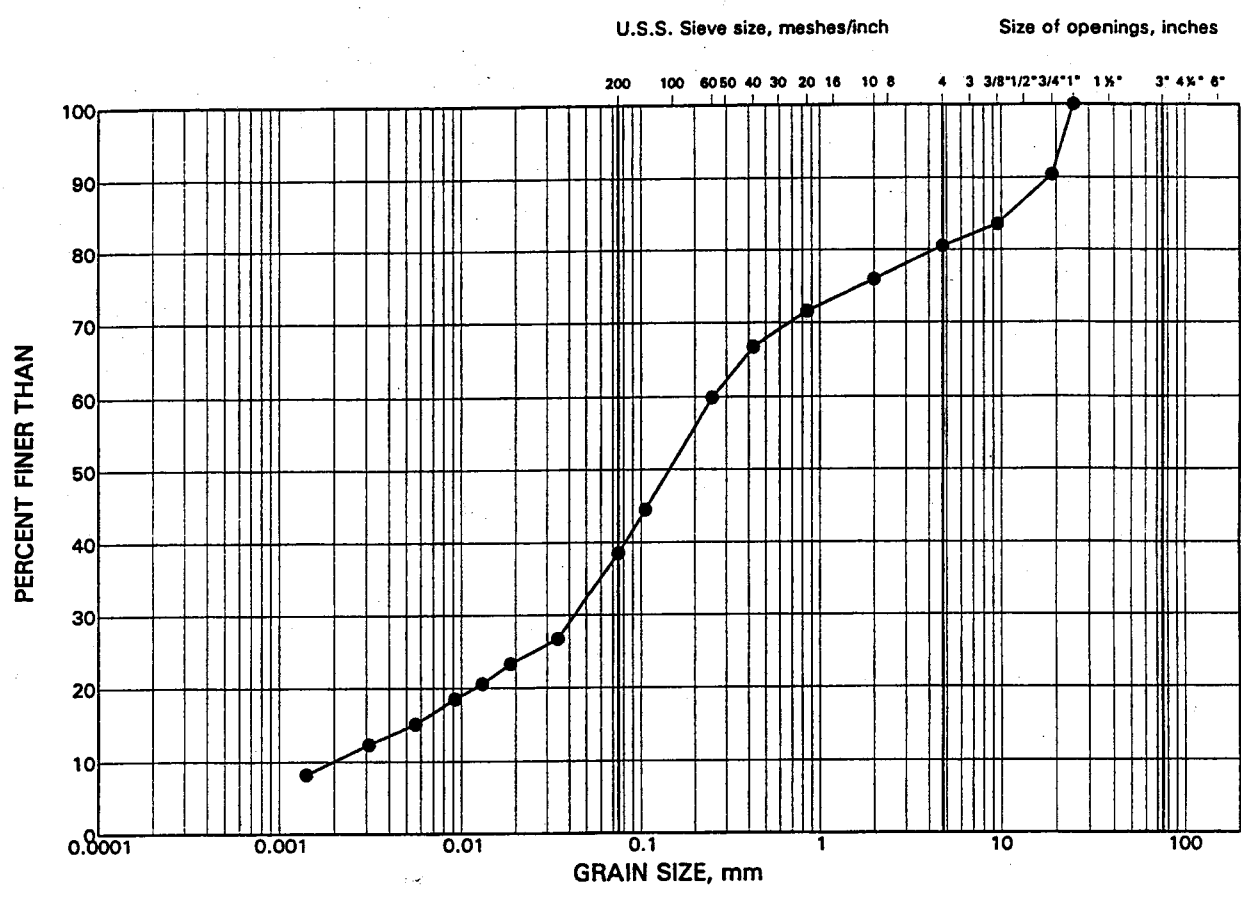
REFERENCE
This drawing was created from digital files "PL-RETWALL.dwg" and "PR-RETWALL.dwg" dated October 25, 2000 provided by The Greer Galloway Group Inc.

NOTE:
Clayey silt (III) as encountered in Borehole 11 was described as "sandy clay loam with gravel" in Boreholes 57-1 and 57-4.

NO.	DATE	BY	REVISION
Geocres No.			
HWY 401	PROJECT NO.: 001-1142	DIST. 41	
SUBM'D. SP	CHKD: LCC	DATE: 2001 01 09	SITE
DRAWN: JFC	CHKD: LCC	APPD: ASP	DWG. 1

GRAIN SIZE DISTRIBUTION SILTY SAND TILL (NON-PLASTIC)

FIGURE 1

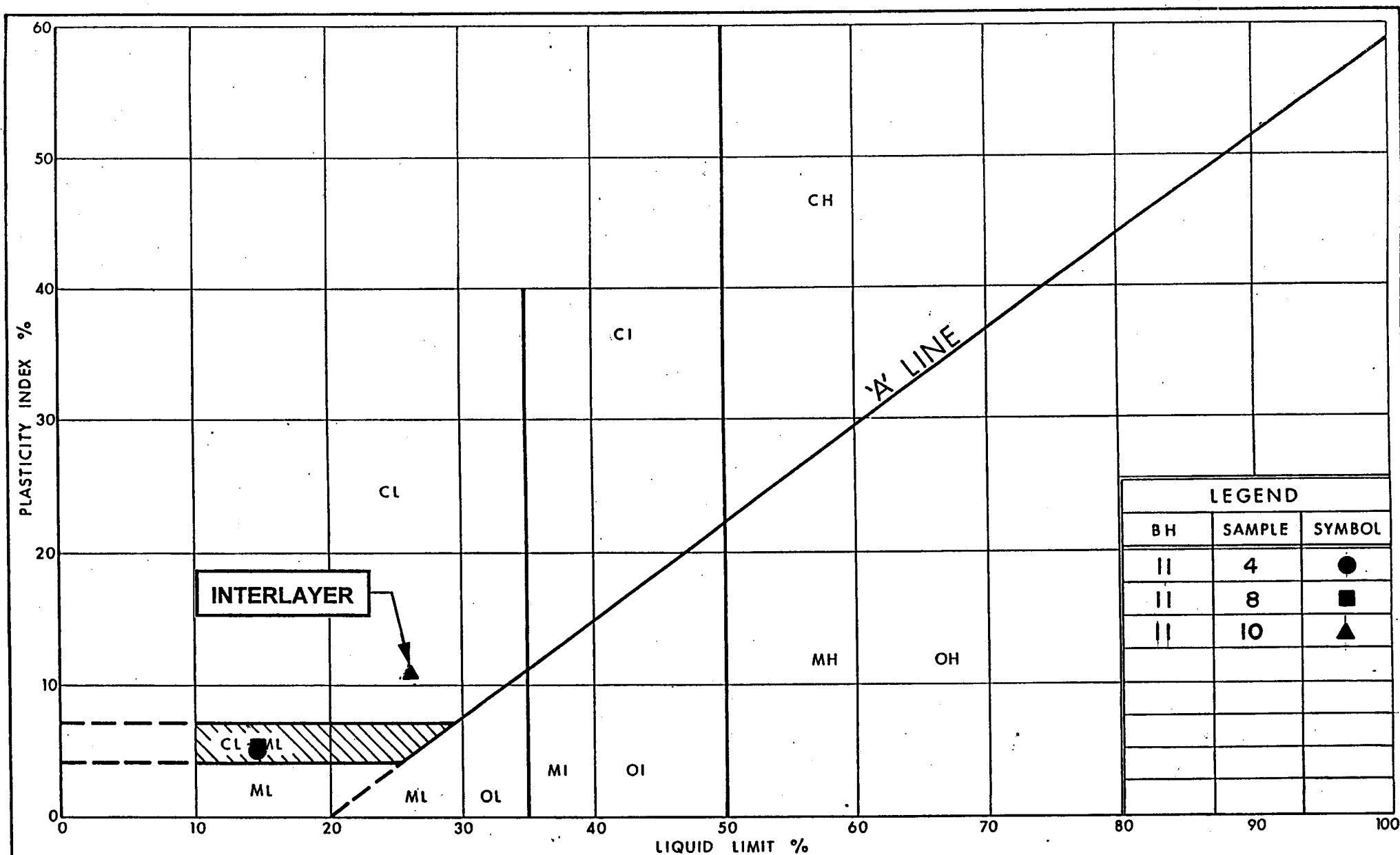


SILT AND CLAY SIZES			FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED			SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	11	6	100.0

Oct 75, FF-S-21



Ontario

Ministry of
Transportation

PLASTICITY CHART **CLAYEY SILT (TILL) AND SILTY CLAY INTERLAYER**

FIG No 2

W P 274 - 96 - 00

January 2001

001-1142A

APPENDIX A

RECORD OF BOREHOLE SHEETS 1957 MTO INVESTIGATION

