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**REPORT ON**

**PRELIMINARY FOUNDATION INVESTIGATION  
AND DESIGN  
PROPOSED WELLINGTON ROAD UNDERPASS  
HIGHWAY 401, SITE 19-369, GWP 476-89-00  
AGREEMENT NUMBER 3005-A-000117**

Submitted to:

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**PART A**

**PRELIMINARY FOUNDATION INVESTIGATION REPORT  
PROPOSED WELLINGTON ROAD UNDERPASS  
HIGHWAY 401, SITE 19-369, GWP 476-89-00  
AGREEMENT NUMBER 3005-A-000117**

## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by Cole, Sherman & Associates (CSA) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out preliminary foundation investigations at the sites of the proposed Highway 401 underpasses for Wonderland Road and Wellington Road in London, Ontario. This report addresses the Wellington Road Underpass (Site 19-369).

It is understood that it is proposed to replace the existing Wellington Road structure over Highway 401. The purpose of the foundation investigation is to determine the subsurface conditions at the site of the proposed new bridge by drilling boreholes and carrying out in-situ tests and laboratory tests on selected samples. The terms of reference for the work are outlined in Golder's Total Project Management (TPM) proposal P01-3064, dated August 2000. The work was carried out in accordance with our Quality Control of TPM Services Plan, Agreement No. 3005-A-000117, dated August 2000.

CSA provided Golder with preliminary drawings for the two Highway 401 underpasses. The centreline and stations of the proposed alignments were surveyed by others prior to commencing the foundation field investigation program. The General Arrangement plan showing the proposed abutment layout of the replacement structure was provided to us in digital format on December 14, 2001. The Ministry of Transportation, Ontario's (MTO's) Foundation Investigation Report, Geocres No. 40I14-122 entitled "Wellington Road Underpass, W.P. 476-89-05, Site 19-369, Highway 401, District 2, London", dated January 18, 1994 has been referenced during the preparation of this report. The relevant borehole records from that report are provided in Appendix A.

## **2.0 SITE DESCRIPTION**

The project area covered by this report extends along Highway 401 at the crossing with Wellington Road in the southern portion of the City of London, Ontario (see Figure 1). Highway 401 runs approximately northeast-southwest and Wellington Road runs northwest-southeast.

The existing and proposed bridge decks are at about elevation 269 metres and the pavement surface on Highway 401 is currently at about elevation 262 metres.

### **3.0 INVESTIGATION PROCEDURES**

The field work for this investigation was carried out on August 28 and 29, 2001. At that time, two boreholes were put down at the site of the proposed new bridge abutments. The boreholes were drilled and sampled to depths of about 19.5 and 20.3 metres. The borehole locations are shown in plan on Drawing 1.

The investigation was carried out using an all terrain vehicle mounted CME-750 drill rig supplied and operated by Lantech Drilling Services Inc. The boreholes were advanced using 208 millimetre outside diameter continuous flight hollow stem augers. In the boreholes, samples of the overburden were obtained at regular intervals of depth using 50 millimetre outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedures. Groundwater conditions in the open boreholes were observed throughout the drilling operations. Piezometers were installed in the boreholes to permit monitoring of the groundwater levels at the site. Both of the boreholes were backfilled using MTO recommended procedures. Water levels in the installations were obtained between August 29 and September 18, 2001 to determine stabilized levels.

The field work was supervised on a full-time basis by members of our engineering staff who located the boreholes in the field, directed the drilling, sampling and in-situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labeled containers and transported to our laboratory in London, Ontario for further examination. Index and classification tests consisting of grain size analyses, Atterberg limits determinations and water content determinations were carried out on selected samples. The results of the field and laboratory testing are given on the Record of Borehole sheets and in Appendix B.

The as drilled borehole locations and elevations were surveyed by Callon Dietz Inc. The elevations at the borehole locations are understood to be referenced to geodetic datum.

## **4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY**

### **4.1 Site Geology**

The area of the site is located in the physiographic region known as the Mount Elgin Ridges in the area between the Ingersoll and Westminster Moraines. Geological information indicates that the general soil conditions at the site consist of the Port Stanley silty clay till overlain locally by patches of lacustrine silts, silty sands and clayey silts.

The bedrock in the area of the site is considered to consist of limestone belonging to the Dundee Formation of the Hamilton Group of Middle Devonian Age. The bedrock surface is estimated to be at about elevation 205 metres, some 56 metres below ground surface.

### **4.2 Site Stratigraphy**

The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets following the text of this report and in Appendix B. The records of relevant boreholes drilled by others are provided in Appendix A. The stratigraphic boundaries shown on the borehole sheets are inferred from non-continuous sampling and, therefore, may 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 site generally consist of variable thicknesses of topsoil and fill materials to about elevation 260 metres which are underlain by a clayey silt deposit about 13 metres thick. The clayey silt deposit is underlain by sand, silty fine sand and silt materials, which extend to about elevation 243 metres. These fine grained granular materials are underlain by clayey silt extending to the depths explored.

Locations and elevations of the borings, together with the interpreted stratigraphical profiles, are shown on the attached Drawings 1 and 2. A detailed description of the subsurface conditions encountered in the boreholes for this investigation is provided on the Record of Borehole sheets and is summarized in the following sections.

#### **4.2.1 Topsoil and Fill**

Topsoil and/or fill layers were encountered in both of the current boreholes. The current boreholes encountered topsoil layers about 0.2 metres thick underlain by 0.5 to 0.9 metres of sandy fill materials. In borehole 1, a 0.3 metre thick layer of topsoil was encountered under the fill. A single standard penetration test in the fill/topsoil materials indicated an N value of 44

blows per 0.3 metres penetration. The topsoil and fill materials had water contents of about 22 and 4 per cent, respectively.

Previous boreholes 4 and 8 encountered 0.5 to 1.9 metres of fill forming the existing north shoulder and embankment, respectively.

#### **4.2.2 Upper Clayey Silt**

Beneath the topsoil and fill materials in current boreholes 1 and 2 and previous boreholes 4 and 8, and at ground surface in other boreholes, a deposit of clayey silt with 6.1 to 14.8 metres total thickness was encountered. The clayey silt deposit contained trace of sand and gravel to about elevation 254 metres and silt layers to about elevation 247 metres in the current boreholes and a number of sand and silt layers were encountered in the previous boreholes. The upper clayey silt deposit extended to elevations between 246.1 and 254.4 metres.

The upper clayey silt deposits had standard penetration test N values typically ranging from 14 to 58 blows per 0.3 metres penetration, with an average N value of 27 blows per 0.3 metres penetration indicating a generally stiff to hard consistency. Figure B1 in Appendix B shows gradation curves for five samples recovered from the upper clayey silt deposit encountered in the current boreholes.

The water contents of the upper clayey silt samples were between about 10 and 26 per cent, with an average of about 17 per cent. The average plastic and liquid limits for the clayey silt, based on four samples tested, are 16 and 24 per cent, respectively, with an average plasticity index of 8 per cent. The results of the Atterberg Limits testing are shown on Figure B2.

#### **4.2.3 Sand**

Beneath the clayey silt in current borehole 1 and a silty fine sand layer in current borehole 2, some 0.8 to 1.1 metres of sand was encountered between about elevations 245.2 and 247.1 metres. The sand had standard penetration test N values of 16 blows per 0.3 metres penetration to 75 blows per 150 millimetres penetration indicating a compact to very dense condition. Water contents were between about 19 and 20 per cent.

#### **4.2.4 Silty Sand and Sandy Silt**

Compact to very dense silty fine sand to sandy silt layers 1.2 to 7.2 metres thick were encountered between the upper and lower clayey silt deposits at about elevation 241.1 to 247.7 metres. The silty sand and sandy silt layers had standard penetration test N values of 10 blows per 0.3 metres penetration to 50 blows per 75 millimetres penetration, with typical N values in the



order of 100 blows per 0.3 metres penetration. Water contents ranged between about 14 and 23 per cent.

#### **4.2.5 Silt**

Current borehole 1 encountered a 0.5 metre thick silt deposit between the layers of lower clayey silt material at about elevation 242 metres. The silt deposit had a standard penetration test N value of 43 blows per 0.3 metres penetration, based on a single standard penetration test. The water content of the silt sample collected was about 19 per cent.

#### **4.2.6 Lower Clayey Silt**

Current boreholes 1 and 2 were terminated in a clayey silt deposit at elevations 241.2 and 241.5 metres after exploring this deposit for some 0.6 to 1.6 metres. The previous boreholes proved this deposit to a maximum depth of 43 metres below ground surface, or elevation 219 metres. The lower clayey silt had N values of 17 to 66 blows per 0.3 metres, with an average of about 35 blows per 0.3 metres penetration, indicating a very stiff to hard consistency. The water content of the lower clayey silt samples ranged between 14 and 19 per cent.

### **4.3 Groundwater Conditions**

Water levels were noted in the boreholes during and upon completion of the drilling operations. These levels are shown on the attached Record of Borehole sheets. Piezometers were sealed in both boreholes to permit the monitoring of the groundwater levels at the site. Details of the piezometer installations and water level measurements are shown on the attached Record of Borehole sheets.

Water was noted during drilling in the boreholes at elevation 247.3 metres. The encountered water levels and those measured in the piezometers between August 29, 2001 and September 18, 2001 are summarized in the following table. On this latter date, the water levels were between elevations 257.4 and 259.3 metres. It should be noted that the groundwater level is subject to seasonal fluctuations.

<b>Borehole Number and Piezometer</b>	<b>Ground Surface Elevation (m)</b>	<b>Encountered Water Level Elevation (m)</b>	<b>Water Level Elevations in Piezometers (2001)</b>				
			<b>August 29 (m)</b>	<b>August 30 (m)</b>	<b>August 31 (m)</b>	<b>September 6 (m)</b>	<b>September 29 (m)</b>
1 Shallow	261.50	247.33	Dry	-	Dry	Dry	257.36
1 Deep			256.75	-	258.07	258.12	258.16
2 Shallow	261.00	247.28	-	257.27	258.33	259.70	259.32
2 Deep			-	255.08	257.37	257.79	258.16

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**PART B**

**PRELIMINARY FOUNDATION DESIGN REPORT  
PROPOSED WELLINGTON ROAD UNDERPASS  
HIGHWAY 401, SITE 19-369, GWP 476-89-00  
AGREEMENT NUMBER 3005-A-000117**

## **5.0 ENGINEERING RECOMMENDATIONS**

### **5.1 General**

This section of the report provides our recommendations on the foundation aspects of the preliminary design of the proposed Wellington Road Underpass based on our interpretation of the factual information obtained during the investigation. It should be noted that the interpretation and recommendations are intended for use only by the design engineer. Where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

It is understood that the existing bridge will be replaced with a new wider and longer structure and that the new approaches will tie in with the existing pavements. Highway 401 will remain at the current grade and the new bridge structure centreline will be just east of the existing bridge.

### **5.2 Replacement Bridge Foundations**

The subsoils encountered in the boreholes put down during the present and previous investigations typically consist of surficial fills and topsoil over stiff to hard clayey silt to about elevation 247 metres, very dense sands and silts to about elevation 243 metres and very stiff to hard clayey silt deposits extending to the depths explored. The water level was measured between about elevations 257.4 and 259.3 metres.

Based on the understanding that the proposed structure is to be built using integral abutments with piers on spread footings and the subsurface information noted above, consideration may be given to supporting the new abutments on steel piles driven into the very dense sands and silty sands at elevation 245 metres. It is understood that the piles will be driven through a Granular A pad constructed to elevation 263.6 metres and the top of the piles will be at elevation 264.2 metres. Alternatively, the entire structure may be founded on spread footings bearing on the clayey silt materials at about elevation 259.5 metres.

#### **5.2.1 Shallow Foundations**

Spread footings may be used to support the new bridge abutments and piers. For the configuration of the proposed bridge and the profile of Highway 401, the highest possible founding level would be within the upper very stiff clayey silt deposit.

### Geotechnical Resistance

For the configuration of the bridge as shown on the General Arrangement drawing, and based on the results of this investigation, the founding level for spread footings on the native very stiff clayey silt materials would be at about elevation 259.5 metres. The factored geotechnical resistance design values for spread footings are given in the table below.

<b><i>Founding Option</i></b>	<b><i>Factored Geotechnical Resistance</i></b>	<b><i>Geotechnical Resistance</i></b>
	<b><i>ULS</i></b>	<b><i>SLS</i></b>
Spread Footings on Stiff to Very Stiff Clayey Silt	450 kPa	300 kPa

The founding level for the footings on the clayey silt is at about 1.5 to 2.0 metres below ground surface. Alternatively, perched abutment foundations on the compacted Granular A pad may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 900 kilopascals (kPa) and a geotechnical resistance at Serviceability Limit States (SLS) of 350 kPa.

The above geotechnical resistances assume, however, that appropriate construction procedures are adopted during footing construction to ensure that the excavations are properly dewatered and the founding soils are not softened/disturbed prior to concrete placement.

If perched abutment foundations are to be constructed, or for the construction of integral abutments, the base of the Granular A pad should extend into the upper very stiff clayey silt deposit.

### Resistance to Lateral Forces

Resistance to lateral forces/sliding between the concrete spread footings and the subsoil should be calculated in accordance with Section 6-8.4.3 of the Ontario Highway Bridge Design Code (OHBD). Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, the following angle of friction between the concrete and the founding soils and corresponding coefficient of friction,  $\tan \delta$ , may be used:

Footings on clayey silt	angle of friction	29°
	$\tan \delta$	0.55

### Frost Protection

All footings should be provided with a minimum of 1.2 metres of earth cover for frost protection purposes.

### Construction Considerations

The founding soils are sensitive to disturbance and softening due to water seepage and/or ponding. Placement of a mud coat will be required at the base of excavation for the footing area. Exposure without protection of the mud coat will result in softening of the founding soils. The cleaned excavation base should be inspected by qualified geotechnical personnel prior to placing the mud coat. It is recommended that the footing excavation be carried out such that the final 0.5 metres of excavation is completed with the geotechnical personnel on site and the mud coat be placed immediately after footing inspection.

## **5.2.2 Deep Foundations**

It is understood that consideration is being given to designing the replacement structure with integral abutments. Driven steel H-piles are considered suitable for the abutment support. In the case of integral abutments, augering and placement of a corrugated steel pipe (CSP) liner around the upper 3 metres of the pile is required.

### Geotechnical Axial Resistance – Driven Steel H-Piles

For design, the factored axial geotechnical resistance at Ultimate Limit States (ULS) for HP 310 x 110 piles driven into the very dense sandy deposits to about elevation 245 metres may be taken as 1,300 kilonewtons (kN). The geotechnical resistance at SLS will depend on pile configuration; however a value of 1,000 kN may be assumed for preliminary assessment.

Care will be required during pile driving to ensure that the piles are not overdriven into the lower clayey silt deposit. However, in accordance with Special Provision 903S01, provision should be made to re-tap the piles to confirm the set after adjacent piles have been driven.

The pile driving note to be added to the drawing is:

"Piles to be driven in accordance with Standard SS 103-11 using an ultimate capacity of 3,250 kN per pile but must be driven below elevation 247 metres and not below elevation 244 metres without approval of the Engineer."

### Geotechnical Axial Resistance – Driven Steel Tube Piles

Alternatively, 300 millimetre diameter tube piles driven closed-ended into the very dense sands and silty sands may be utilized. The tube piles may be designed with a factored axial geotechnical resistance at ULS of 1,300 kN and the geotechnical resistance at SLS may be taken as 1,000 kN.

### Downdrag Load (Negative Skin Friction)

As will be discussed in a later section, the approach embankments will cause consolidation settlement of the underlying thick clayey silt deposits as a result of increased vertical grades. The consolidation settlement is time-dependent and will not completely occur during the construction period. That is, post-construction settlement of the clayey silt deposits will take place and settlement of the clayey silt relative to the piles will result in the development of negative skin friction acting on the piles. Therefore, negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the abutments. If the approach embankments are constructed well in advance of the piling, the downdrag amount may be reduced or even eliminated.

The magnitude of the downdrag load acting on a pile is a function of the adhesion (skin friction) that develops between the pile and the clay, and the surface area of the pile within the clay deposit and the embankment loading. The load calculated in this manner is a nominal (unfactored) load. The structural engineer needs to multiply this load by a load factor of 1.25, as defined in the OHBDC, and include it as part of the load effects acting on the pile as described in the OHBDC. For preliminary design, the negative skin friction load on a single pile may be taken as 250 kN. If the embankments are not constructed well in advance of the piling, the downdrag load will have to be reassessed during the detailed design stage by the foundation engineer.

### Resistance of Lateral Loads

The lateral loading could be resisted fully or partially by the use of battered piles. In the case of integral abutments, the vertical piles must provide the resistance to the lateral loading. In this case, the horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient where:

$$k_s = \text{coefficient of horizontal subgrade reaction (MPa/m)} = \begin{cases} n_h (z/d) & \text{for cohesionless soils} \\ \frac{k_{si}}{5d} & \text{for cohesive soils} \end{cases}$$

$d$  = pile width or diameter (m)

$n_h, k_{si}$  = coefficient of horizontal subgrade reaction (MPa/m)

$z$  = depth below adjacent road grade (m)

Above elevation 247 metres      stiff to hard clayey silt       $k_{si} = 15 \text{ to } 35 \text{ MPa/m}$

Below elevation 247 metres      Very dense sands and silty sands       $n_h = 10 \text{ to } 15 \text{ MPa/m}$

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 horizontal subgrade reaction in the direction of loading by a reduction factor  $R$  as follows:

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

### Frost Protection

The pile caps should be provided with a minimum of 1.2 metres of soil cover for frost protection.



### **5.3 Embankments**

It is understood that embankment fills up to 8 metres in height will be constructed for the proposed new bridge. Embankment side slopes formed no steeper than 2 horizontal to 1 vertical are considered suitable for this site. A Factor of Safety against deep seated failure of greater than 1.3 is obtained for embankments constructed with the native clayey silts. Preliminary settlement calculations indicate that consolidation settlement in the order of 160 millimetres should be expected in the clayey silt deposit due to the embankment fill. It should be noted that such settlements will cause a considerable grade difference if the bridge is constructed on piled foundations and consideration should be given to constructing the embankments well in advance of the foundations. Alternatively, shallow foundations constructed no less than one year following embankment construction may be used to reduce the effects of differential settlement.

The topsoil and organic materials should be removed from within the area of the embankment and the exposed subgrade soils should be proofrolled prior to fill placement.

Construction of the embankment above the prepared subgrade may be carried out using clean earth fill (in accordance with Ontario Provincial Standard Specifications (OPSS) 212) or select subgrade material (in accordance with OPSS 1010) depending on material availability. All embankment fill should be placed in regular lifts and properly benched into the existing embankment, where required.

### **5.4 Excavations and Temporary Cut Slopes**

Excavations for footing construction will extend through fill materials and clayey silt deposits. At the proposed new bridge abutment locations, the excavations for the construction of spread footings and/or granular pad for pile construction will be up to about 2 metres in depth. The base of the spread footing excavations will be above the groundwater level as measured in the piezometers. The groundwater level is expected to fluctuate and the depth of excavation below the groundwater will depend on the time of year of construction. Temporary open cut slopes within the fill and clayey silt deposits above the groundwater level should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V).

Water seepage into the excavations through the fill should be expected and will be heavier during periods of sustained precipitation. Pumping from well filtered sumps located at the base of the excavations may be required to provide groundwater control during foundation excavations.

The consideration with respect to protection of the founding soils, however, as given in Section 5.2.1 under the heading Construction Considerations must be recognized. Sumps should be maintained outside of the actual footing limits. Surface water runoff should be directed away

from the excavations at all times. The appropriate Non Standard Special Provision (NSSP) should be included in the contract documents.

Where space is restricted and will not permit open cuts, a temporary support system should be installed to support the sides of the excavation and permit the use of vertical cuts. The temporary support system could consist of soldier piles and lagging where the H-piles would be driven into the clayey silt and horizontal lagging installed as the excavation proceeds. Support to the soldier pile and lagging wall system could be in the form of struts and walers in the case of footing excavations or rakers and anchors in the case of roadway protection excavations.

The design of braced soldier pile and lagging walls should be based on a rectangular earth pressure distribution using the design parameters given below. Where the support to the wall is provided by anchors or rakers, the wall design should be based on a triangular earth pressure distribution using the design parameters given below. The raker/anchor support must be designed to accommodate the loads applied from pressures and surcharge pressures from area, line or point loads as well as the impact of sloping ground behind the system.

Unfactored triangular earth pressure distribution ( $p$  in  $\text{kN/m}^2$ ; increasing with depth), can be calculated as follows:

$$p = K_a \gamma H$$

where

$$H = \text{the height of the excavation at any point in metres}$$

$$K_a = 0.3 \text{ for level ground behind excavation}$$

$$\gamma = \text{soil unit weight} = 20 \text{ kN/m}^3$$

Unfactored rectangular earth pressure distribution ( $p$  in  $\text{kN/m}^2$ ; constant with depth), can be calculated as follows:

$$p = K_a \gamma H$$

where

$$H = \text{the height of the excavation}$$

$$K_a = 0.3 \text{ for level ground behind excavation}$$

$$\gamma = \text{soil unit weight} = 20 \text{ kN/m}^3$$

Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter. The coefficient of passive lateral earth pressure,  $K_p$ , for the socket within the very stiff to hard clayey silt may be taken as 8.7. The soil unit weight should be taken as  $21 \text{ kN/m}^3$  and the unit weight of water should be taken as  $9.8 \text{ kN/m}^3$ . A groundwater level at elevation 259 metres can be assumed at the bridge footing locations.

All excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Ontario Occupational Health and Safety Act and Regulations For Construction Projects. The fill at this site would be classified as a Type 3 soil and the clayey silt deposit as a Type 2 soil.

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## LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole", on the figures and in the text of the report, are as follows:

### I. SAMPLE TYPES

<i>AS</i>	auger sample
<i>CS</i>	chunk sample
<i>DO</i>	drive open
<i>DS</i>	Denison type sample
<i>FS</i>	foil sample
<i>RC</i>	rock core
<i>ST</i>	slotted tube
<i>TO</i>	thin-walled, open
<i>TP</i>	thin-walled, piston
<i>WS</i>	wash sample
<i>SS</i>	split spoon

### II. PENETRATION RESISTANCES

#### Dynamic Penetration Resistance:

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 0.3 m (12 in.).

#### Standard Penetration Resistance, 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 0.3 m (12 in.).

<i>WH</i>	sampler advanced by static weight-weight, hammer
<i>PH</i>	sampler advanced by hydraulic force
<i>PM</i>	sampler advanced by manual force

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Relative Density	"N" Blows/0.3 m or Blow/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils

Consistency	"Cu" = "Su" kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

### IV. SOIL TESTS

<i>C</i>	consolidation test
<i>H</i>	hydrometer analysis
<i>M</i>	sieve analysis
<i>MH</i>	combined analysis, sieve and hydrometer <sup>1</sup>
<i>Q</i>	undrained triaxial <sup>2</sup>
<i>R</i>	consolidated undrained triaxial <sup>2</sup>
<i>S</i>	drained triaxial
<i>U</i>	unconfined compression
<i>V</i>	field vane test
<i>Chem</i>	chemical analysis

#### NOTES:

1. Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.
2. Undrained triaxial tests in which pore pressures are measured are shown as Q or R.

## LIST OF SYMBOLS

### I. GENERAL

$\pi$	= 3.1416
e	= base of natural logarithms 2.7183
$\log_e$	a or $\ln$ a, natural logarithm of a
$\log_{10}$	a or $\log$ a, logarithm of a to base 10
$t$	time
$g$	acceleration due to gravity
$V$	volume
$W$	weight
$m$	mass
$M$	moment
$F$	factor of safety

### II. STRESS AND STRAIN

$u$	pore pressure
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\sigma$ is also used)
$\tau$	shear stress
$\varepsilon$	linear strain
$\varepsilon_{sy}$	shear strain
$\nu$	Poisson's ration ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

#### (a) Unit weight

$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	unit weight of submerged soil
$G_s$	specific gravity of solid particles $G_s = \gamma_s/\gamma_w$
$e$	void ratio
$n$	porosity
$w$	water content
$S_r$	degree of saturation

#### (b) Consistency

$w_L$	liquid limit
$w_P$	plastic limit
$I_P$	plasticity index
$w_S$	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
$D_r$	relative density = $(e_{max} - e)/(e_{max} - e_{min})$

#### (c) Permeability

$h$	hydraulic head or potential
$q$	rate of discharge
$v$	velocity of flow
$i$	hydraulic gradient
$\kappa$	coefficient of permeability
$j$	seepage force per unit volume

#### (d) Consolidation (one-dimensional)

$m_v$	coefficient of volume change = $-\Delta e/(1+e)\Delta\sigma'$
$C_c$	compression index = $-\Delta e/\Delta\log_{10}\sigma'$
$c_v$	coefficient of consolidation
$T_F$	time factor = $c_v t/d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

#### (e) Shear strength

$\tau_f$	shear strength	$\left. \begin{array}{l} \text{in terms} \\ \text{of effective} \\ \text{stress} \end{array} \right\} \tau_f = c' + \sigma' \tan \phi$
$c'$	effective cohesion intercept	
$\phi'$	effective angle of shearing resistance, or friction	
$S_u$	apparent cohesion*	
$\phi_u$	apparent angle of shearing resistance, or friction	$\left. \begin{array}{l} \text{in terms of} \\ \text{total stress} \end{array} \right\} \tau_f = cu + \sigma \tan \phi_u$
$\mu$	coefficient of friction	
$S_t$	sensitivity	

\*For the case of a saturated cohesive soil,  $\phi_u = 0$  and the undrained shear strength  $\tau_f = S_u$  is taken as half the undrained compressive strength.

# RECORD OF BOREHOLE No 1

1 OF 2

**METRIC**

PROJECT 001-3225-1  
W.P. 476-89-00 LOCATION 4754411.2 N, 409936.9 E ( WELLINGTON ROAD SITE No. 19-369 )  
DIST HWY 401 BOREHOLE TYPE POWER AUGER (HOLLOW STEM)  
DATUM GEODETIC DATE 28.8.01

ORIGINATED BY DJM  
COMPILED BY DJM  
CHECKED BY AMH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
261.50	GROUND SURFACE							20 40 60 80 100					kN/m <sup>3</sup>	GR SA SI CL
0.00	TOPSOIL, sandy													
0.20	Brown													
	FILL, silty fine sand, with gravel													
	Dense													
	Brown													
260.43			1	SS	44									
1.07	TOPSOIL, silty													
260.13	Compact													
1.37	Black													
	CLAYEY SILT, trace sand, trace		2	SS	22									
	gravel													
	Very stiff to hard													
	Brown to Grey at 5.2m depth													
			3	SS	27									2 10 54 34
			4	SS	49									
			5	SS	29									
			6	SS	19									
			7	SS	22									4 62 34
			8	SS	22									
			9	SS	23									1 2 66 31
254.03	CLAYEY SILT, with silt layers, trace													
7.47	sand		10	SS	39									
	Stiff to hard													
	Grey													
			11	SS	31									
			12	SS	27									
			13	SS	14									
			14	SS	19									2 65 33
247.17	SAND, fine, trace silt													
14.33	Compact		15	SS	16									
	Grey													

ON\_MOT 0013225B.GPJ ON\_MOT.GDT 18-1-02 DATA INPUT:

Continued Next Page

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

# RECORD OF BOREHOLE No 1

2 OF 2

**METRIC**

PROJECT 001-3225-1  
W.P. 476-89-00 LOCATION 4754411.2 N, 409936.9 E ( WELLINGTON ROAD SITE No. 19-369 ) ORIGINATED BY DJM  
DIST HWY 401 BOREHOLE TYPE POWER AUGER (HOLLOW STEM) COMPILED BY DJM  
DATUM GEODETIC DATE 28.8.01 CHECKED BY AMH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20   40   60   80   100		W <sub>p</sub> W   W <sub>L</sub>				
								○ UNCONFINED   + FIELD VANE						
								● QUICK TRIAXIAL   × LAB VANE						
								20   40   60   80   100		10   20   30				
246.11														
15.39	SILTY FINE SAND Very dense Grey		16	SS	90		246							
			17	SS 100/125mm			245							
			18	SS 85/150mm			244							
			19	SS 72/150mm			243							
			20	SS 100/225mm			242							
242.75														
18.75	CLAYEY SILT, trace sand, trace gravel													
242.30	Hard Grey		21	SS	43									
241.84	SILT, trace sand													
19.66	Dense Grey													
241.23	CLAYEY SILT, trace sand		22	SS	26									
20.27	Very stiff Grey													
	End of Borehole													
NOTE: Water level encountered in Borehole at elev. 247.33 during drilling Aug. 28, 2001  Water level in Piezometer #1 (deep) at elev. 258.16 Sept. 29, 2001 Water level in Piezometer #2 (shallow) at elev. 257.36 Sept. 29, 2001														

# RECORD OF BOREHOLE No 2

1 OF 2

METRIC

PROJECT 001-3225-1

W.P. 476-89-00

LOCATION 4754471.6 N, 409891.9 E ( WELLINGTON ROAD SITE No. 19-369 )

ORIGINATED BY DJM

DIST HWY 401

BOREHOLE TYPE POWER AUGER (HOLLOW STEM)

COMPILED BY DJM

DATUM GEODETIC

DATE 29.8.01

CHECKED BY AMH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
261.00	GROUND SURFACE														
0.00	TOPSOIL, silty														
0.15	Brown														
260.39	FILL, sand trace silt, with gravel														
0.61	CLAYEY SILT, trace sand, trace gravel														
	Very stiff to hard		1	SS	58										
	Brown to Grey at 3.7m depth		2	SS	39										
			3	SS	23										
			4	SS	22										
			5	SS	19										
			6	SS	24										
			7	SS	20										
			8	SS	34										
			9	SS	27										
253.53	CLAYEY SILT, with silt layers, trace sand		10	SS	17										
7.47	Very stiff grey														
	Grey		11	TW	PH										
			12	SS	16										
			13	SS	23										
247.13	SILTY FINE SAND		14	SS	49										
13.87	Dense to very dense		15	SS 93/250mm											
	Grey														

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE



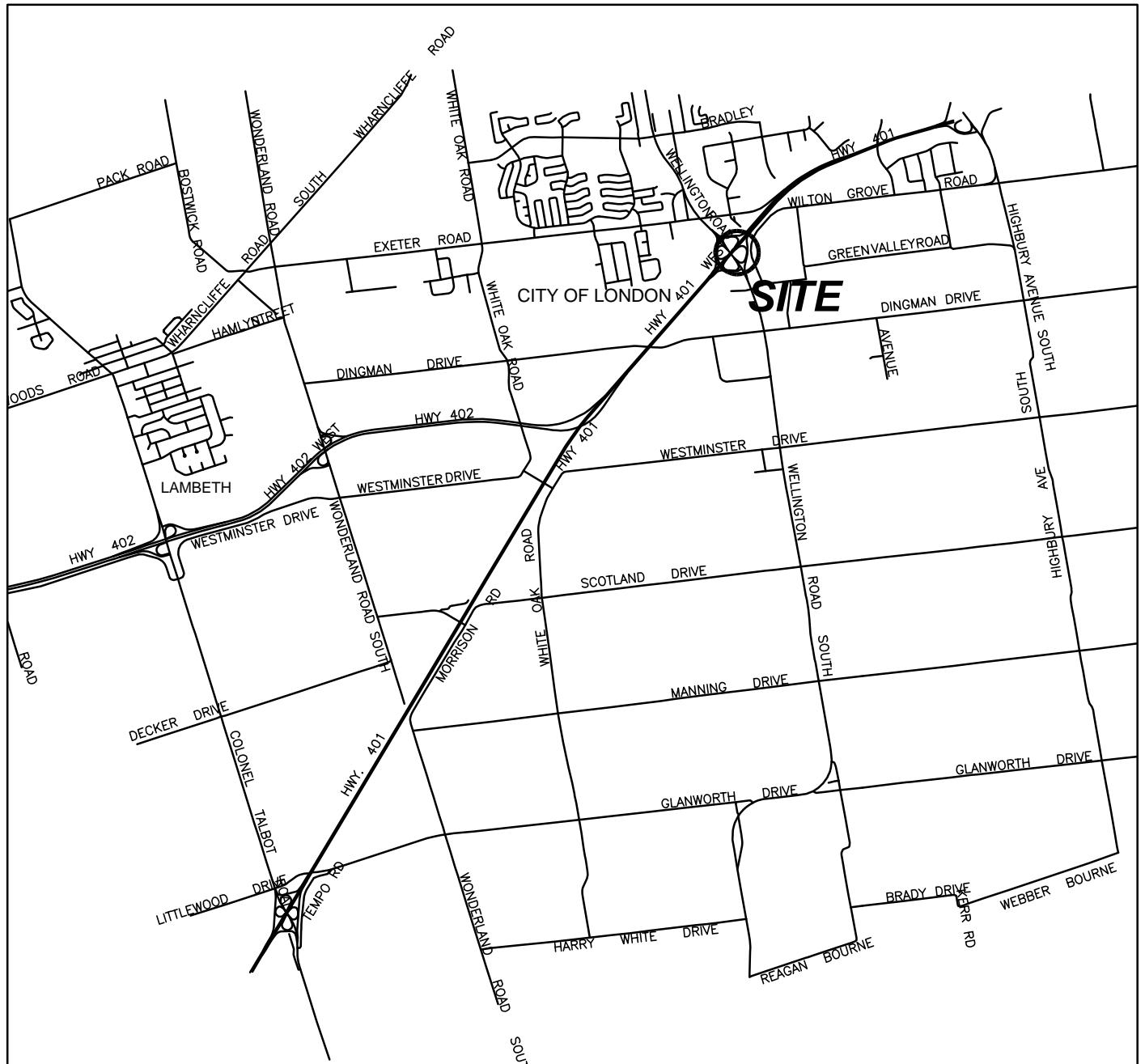
# RECORD OF BOREHOLE No 2


2 OF 2

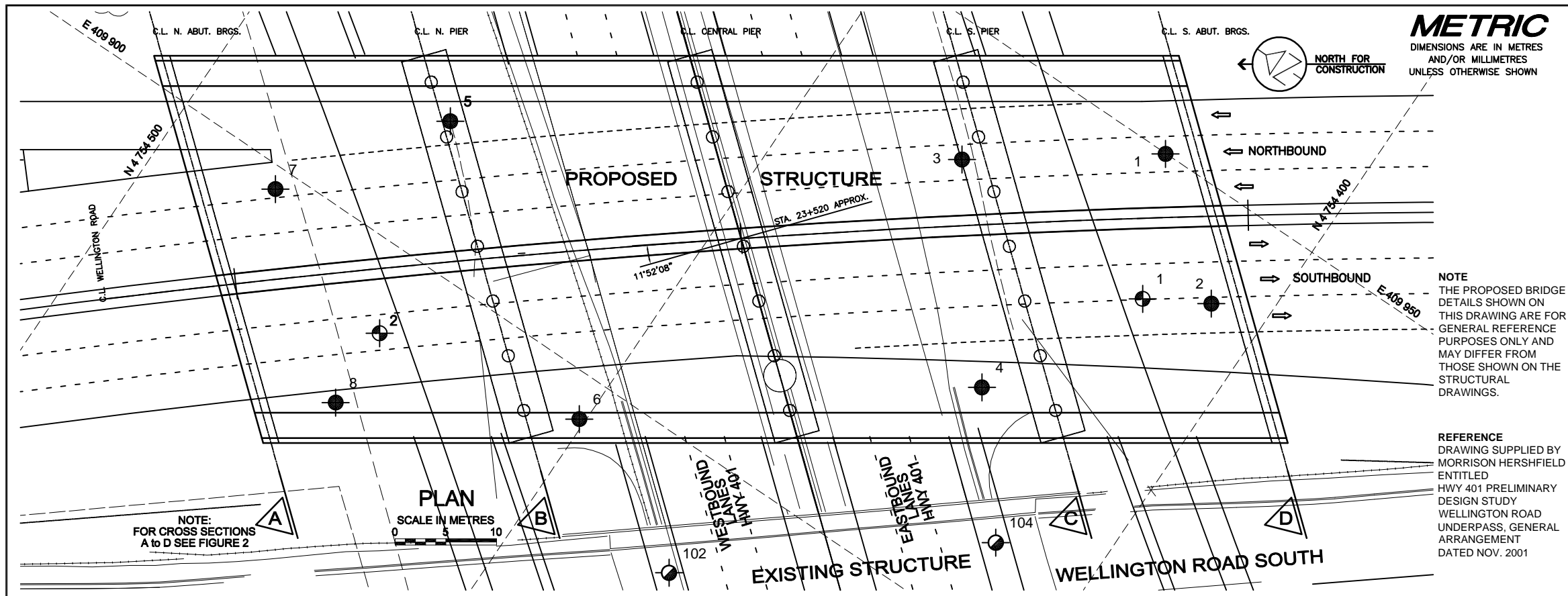
**METRIC**

PROJECT 001-3225-1  
W.P. 476-89-00 LOCATION 4754471.6 N, 409891.9 E ( WELLINGTON ROAD SITE No. 19-369 ) ORIGINATED BY DJM  
DIST HWY 401 BOREHOLE TYPE POWER AUGER (HOLLOW STEM) COMPILED BY DJM  
DATUM GEODETIC DATE 29.8.01 CHECKED BY AMH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE										
								● QUICK TRIAXIAL × LAB VANE										
245.85	SAND, fine, trace silt Very dense Grey		16	SS	75/150mm													
245.15			17	SS	53/150mm													
15.85			SILTY FINE SAND Very dense Grey	18	SS	50/75mm												
	19	SS		50/75mm														
243.11	CLAYEY SILT, with silt layers Very stiff to hard Grey	20		SS	66													
17.89																		
		21	SS	27														
241.49	End of Borehole																	
19.51	NOTE: Water level encountered in Borehole at elev. 247.28 during drilling Aug. 29, 2001  Water level in Piezometer #1 (deep) at elev. 258.16 Sept. 29, 2001 Water level in Piezometer #2 (shallow) at elev. 259.32 Sept. 29, 2001																	



PROJECT		WELLINGTON ROAD STRUCTURE WP. 476-89-00 HWY. 401	
TITLE		SITE LOCATION MAP	
 <b>Golder Associates</b> LONDON, ONTARIO		PROJECT No.	001-3225-1
		DESIGN	
		CADD	WDF 12/18/01
		CHECK	AMH 12/18/01
		REVIEW	
		FILE No.	00132250102
		SCALE	N.T.S. REV. 0
		<b>FIGURE 1</b>	



DIST

CONT. No.

WP No. 476-89-00

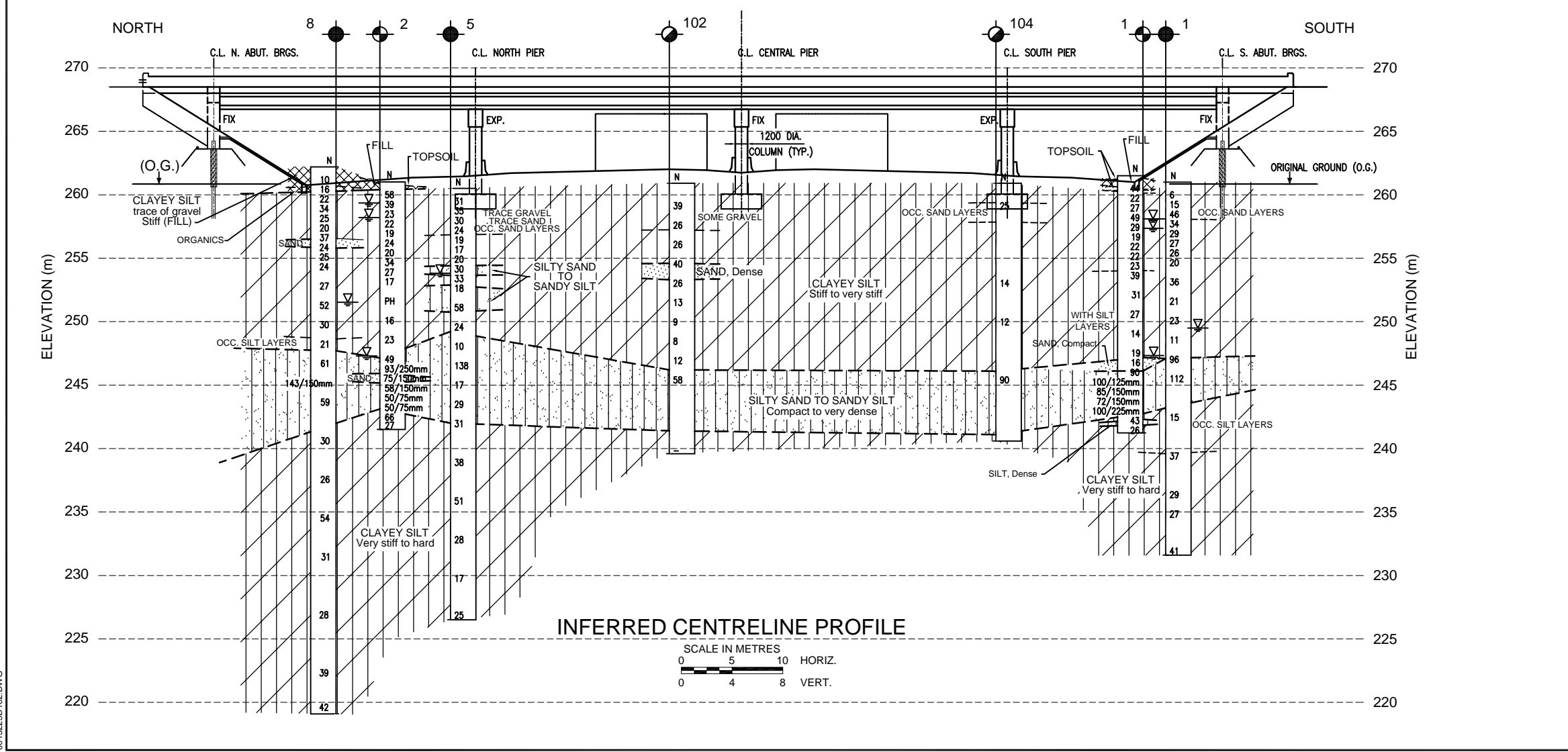
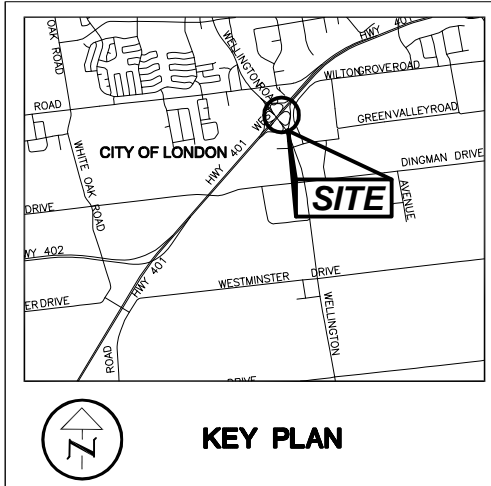
HWY. 401






SHEET

WELLINGTON ROAD STRUCTURE

BOREHOLE LOCATIONS & SOIL STRATA

Golder Associates Ltd.  
LONDON, ONTARIO, CANADA



LEGEND	
	Borehole ( Current Investigation by Golder Associates Ltd. )
	Borehole ( Previous Investigation WP 476-89-05, Jan 1994. )
	Borehole ( Previous Investigation WP 476-89-05, Jan 1994. )
	Note: Boreholes from Previous Report 54-f-224C by Racey MacCallum and Associates. Dated 1954.
N	Blows/0.3m (Std. Pen. Test, 475 j/blow)
	WL in piezometer
	WL during drilling

No.	ELEVATION (metres)	CO-ORDINATES	
		NORTH	EAST
1	261.50	4 754 411.2	409 936.9
2	261.00	4 754 471.6	409 891.9
1	261.00	4 754 417.4	409 950.0
2 (Cone)	261.70	4 754 405.4	409 940.3
3 (Cone)	260.70	4 754 433.7	409 938.3
4 (Cone)	261.80	4 754 419.5	409 920.8
5	260.50	4 754 477.6	409 913.1
6 (Cone)	262.00	4 754 450.6	409 896.0
7 (Cone)	260.80	4 754 488.1	409 898.0
8	262.20	4 754 471.4	409 883.9
102	260.90	4 754 434.8	409 888.4
104	260.90	4 754 409.8	409 908.9

CO-ORDINATES HAVE BEEN REVISED TO  
CURRENT SUPPLIED PLAN, SEE REFERENCE

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

NO.	DATE	BY	REVISION
Geocres No.			
HWY. No.	401	PROJECT NO.:	001-3225-1
SUBM'D.	-	CHKD:	-
DRAWN:	WDF	CHKD.	-
DATE:	JAN. 2002	APPD.	
DWG.	1		

0013225D102.DWG



**APPENDIX A**

**RECORDS OF PREVIOUS BOREHOLES**

# RECORD OF BOREHOLE No 1

1 OF 1

METRIC

W.P. 476 - 89 - 05 LOCATION Co-ords: N 4 754 197.7; E 409 942.7 ORIGINATED BY M V  
DIST 2 HWY 401 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V  
DATUM GEODETIC DATE 93 01 25 TO 93 02 02 CHECKED BY P P

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20						40	60	80	100	20	40
261.0	Ground Surface																		
0.0	Trace of Sand and Trace of Gravel ----- Occasional Sand Layers -----  CLAYEY SILT, Hard to Very Stiff		1	SS	6														
			2	SS	15														
			3	SS	46														
			4	SS	34														
			5	SS	29														
			6	SS	27														
			7	SS	26														
			8	SS	20														
			9	SS	36														
			10	SS	21														
			11	SS	23														
			12	SS	11														
	Occasional Silt Layers -----		13	SS	96														
247.1			14	SS	112														
13.9	SILTY SAND to SANDY SILT, Very Dense		15	SS	15														
243.2			16	SS	37														
17.8	Occasional Silt Layers -----  CLAYEY SILT, Very Stiff to Hard		17	SS	29														
			18	SS	27														
			19	SS	41														
231.6																			
29.4	End of Borehole																		

# RECORD OF BOREHOLE No 2

1 OF 1

METRIC

W.P. 476 - 89 - 05 LOCATION Co-ords: N 4 754 185.7; E 409 933.0 ORIGINATED BY M V  
 DIST 2 HWY 401 BOREHOLE TYPE CONE TEST COMPILED BY M V  
 DATUM GEODETIC DATE 93 02 11 CHECKED BY P P

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20					
261.7	Toe of Embankment												
0.0	Probable CLAYEY SILT					260							
259.0													
2.7	End of Cone Test												

# RECORD OF BOREHOLE No 3

1 OF 1 METRIC

W.P. 476 - 89 - 05 LOCATION Co-ords: N 4 754 214.0; E 409 931.0 ORIGINATED BY M V  
 DIST 2 HWY 401 BOREHOLE TYPE CONE TEST COMPILED BY M V  
 DATUM GEODETIC DATE 93 02 11 CHECKED BY P P

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	W <sub>P</sub> W W <sub>L</sub>	10 20 30			
260.7	Ground Surface						260							
0.0	Probable CLAYEY SILT													
258.3														
2.4	End of Cone Test							120	28cm					



# RECORD OF BOREHOLE No 4

1 OF 1

METRIC

W.P. 476 - 89 - 05 LOCATION Co-ords: N 4 754 199.8; E 409 913.5 ORIGINATED BY M V  
DIST 2 HWY 401 BOREHOLE TYPE CONE TEST COMPILED BY M V  
DATUM GEODETIC DATE 93 02 11 CHECKED BY P P

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80	100	W <sub>p</sub>	W		
261.8	Hwy. 401 EBL Shoulder															
0.0	Asphalt Gravelly Sand (Fill)	X 2														
0.5	Probable CLAYEY SILT															
259.4																
2.4	End of Cone Test															

# RECORD OF BOREHOLE No 5

1 OF 2

METRIC

W.P. 476 - 89 - 05 LOCATION Co-ords: N 4 754 252.4; E 409 903.0  
 DIST 2 HWY 401 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST  
 DATUM GEODETIC DATE 93 02 08 & 09

ORIGINATED BY M V  
 COMPILED BY M V  
 CHECKED BY P P

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W <sub>P</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
260.5	Ground Surface									
0.0	Trace of Sand, Trace of Gravel, Occasional Sand Layers		1	SS	31					
			2	SS	35					
			3	SS	30					
			4	SS	24					
	CLAYEY SILT, Hard to Very Stiff		5	SS	19					
			6	SS	17					
			7	SS	20					
254.4			8	SS	30					
6.1	Clayey Silt		9	SS	33					
			10	SS	18					
			11	SS	58					0 88 (12)
	Clayey Silt		12	SS	24					
			13	SS	10					0 1 (99)
	SILTY SAND to SANDY SILT, Occasional Clayey Silt Layers, Compact to Very Dense		14	SS	138					0 32 (68)
			15	SS	17					
			16	SS	29					0 11 89
242.0			17	SS	31					
18.5			18	SS	38					
	CLAYEY SILT, Hard to Very Stiff		19	SS	51					
			20	SS	28					
230.0										

30.5 Continued

+3, x5: Numbers refer to  
Sensitivity  
20  
15-5 (%) STRAIN AT FAILURE  
10

Continued

# RECORD OF BOREHOLE No 5

2 OF 2

METRIC

W.P. 476 - 89 - 05 LOCATION Co-ords: N 4 754 252.4; E 409 903.0 ORIGINATED BY M V  
 DIST 2 HWY 401 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V  
 DATUM GEODETIC DATE 93 02 08 & 09 CHECKED BY P P

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80	100	W <sub>P</sub>	W		
230.0	Continued															
30.5	CLAYEY SILT, Very Stiff		21	SS	17											
226.5						228										
			22	SS	25											
34.0	End of Borehole															



# RECORD OF BOREHOLE No 7

1 OF 1

METRIC

W.P. 476 - 89 - 05 LOCATION Co-ords: N 4 754 268.4; E 409 890.7 ORIGINATED BY M V  
DIST 2 HWY 401 BOREHOLE TYPE CONE TEST COMPILED BY M V  
DATUM GEODETIC DATE 93 02 03 CHECKED BY P P

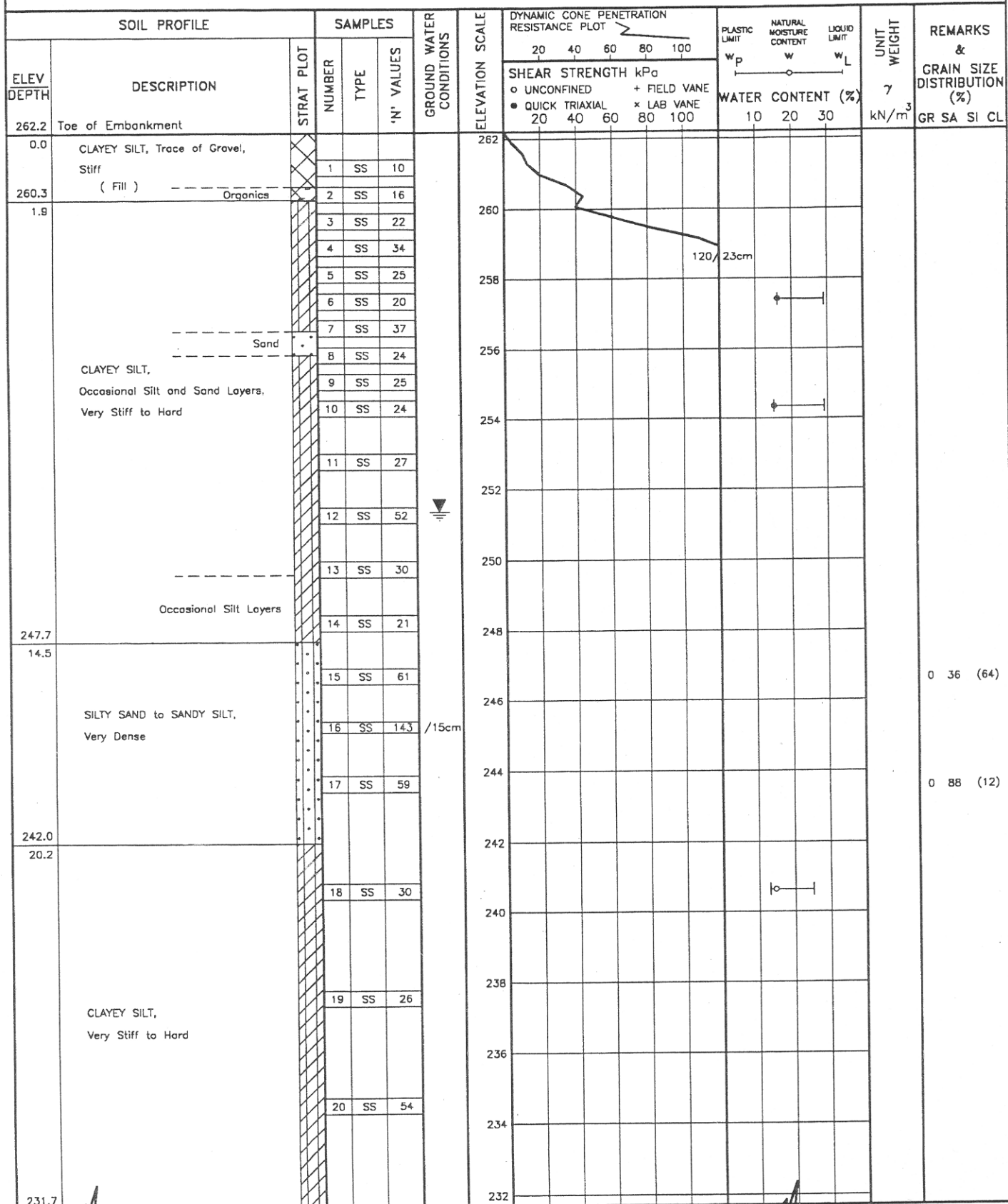
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
260.8	Ground Surface												
0.0	Probable CLAYEY SILT												
256.2													
4.6	End of Cone Test												

# RECORD OF BOREHOLE No 8

1 OF 2

METRIC

W.P. 476 - 89 - 05 LOCATION Co-ords: N 4 754 251.7; E 409 876.6 ORIGINATED BY M V  
DIST 2 HWY 401 BOREHOLE TYPE CONTINUOUS FLIGHT HOLLOW STEM AUGER & CONE TEST COMPILED BY M V  
DATUM GEODETTIC DATE 93 02 03 TO 93 02 05 CHECKED BY P P



30.5 Continued

+3, x5: Numbers refer to  
Sensitivity

20  
15-5 (%) STRAIN AT FAILURE  
10

Continued



# RECORD OF BOREHOLE No 102

1 OF 1

METRIC

W.P. 476 - 89 - 05 LOCATION Co-ords: N 4 754 215.1; E 409 881.1 ORIGINATED BY R MacC&A  
DIST 2 HWY 401 BOREHOLE TYPE WASHBORING COMPILED BY M V  
DATUM GEODETIC DATE 54 10 28 TO 54 10 30 CHECKED BY P P

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	WATER CONTENT (%) W	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100						
260.9	Ground Surface																	
0.0						*												
	Some Gravel		1	SS	39		260											
			2	SS	26		258											
	CLAYEY SILT, Hard to Very Stiff		3	SS	26		256											
254.6			4	SS	40		254											
6.3	SAND, Dense		5	SS	26		252											
253.3			6	SS	13		250											
7.6			7	SS	9		248											
	CLAYEY SILT, Very Stiff to Stiff		8	SS	8		246											
			9	SS	12		244											
246.2			10	SS	58		242											
14.7	SANDY SILT, Very Dense						240											
241.4																		
19.5	CLAYEY SILT, Very Stiff to Hard																	
239.6			11	SS	-													
21.3	End of Borehole Note: Formerly BH# 2 of 54 - F - 224C R MacC&A: Racey MacCollum and Associates * Water Level Not Observed																	



# RECORD OF BOREHOLE No 104

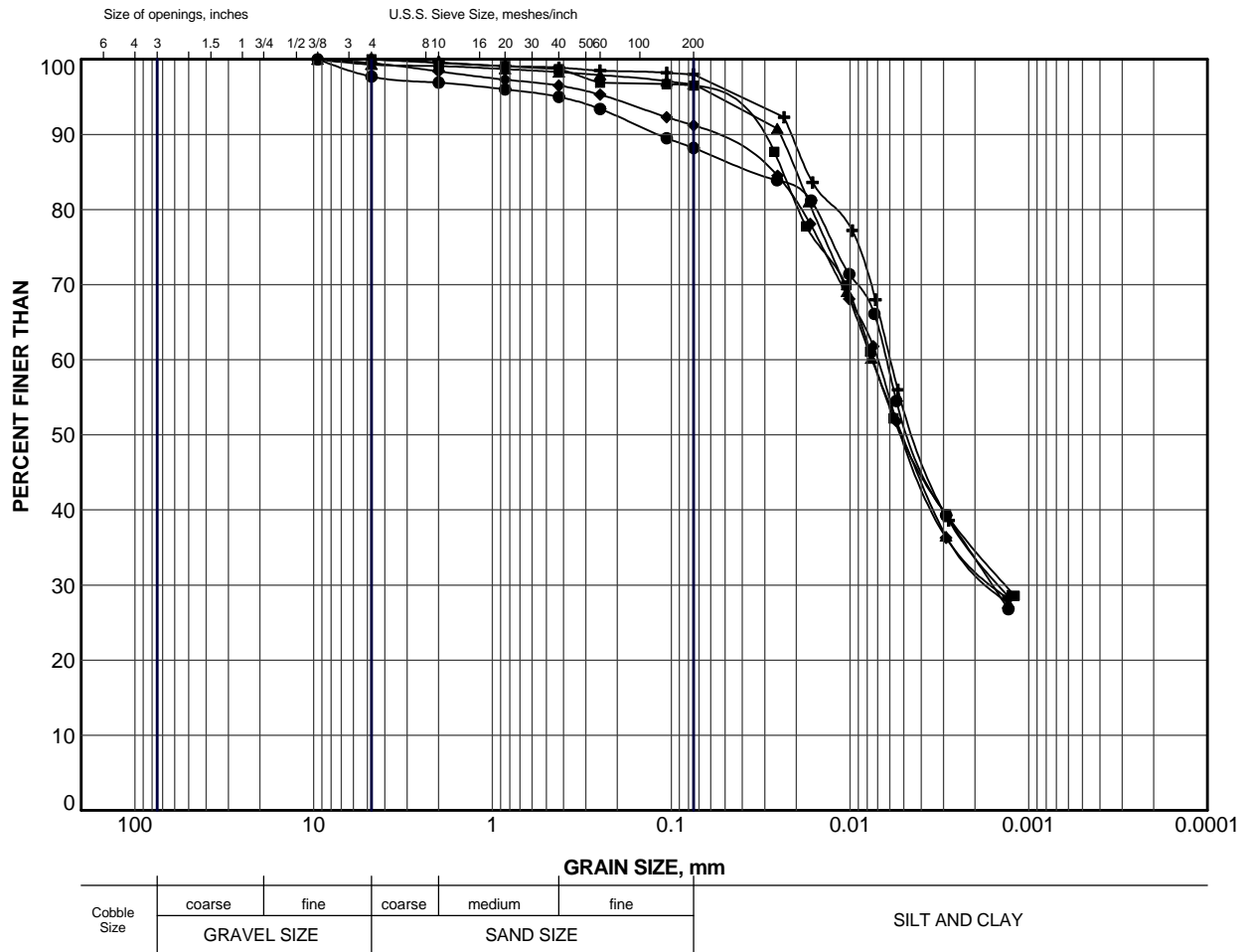
1 OF 1

METRIC


W.P. 476 - 89 - 05 LOCATION Co-ords: N 4 754 190.1; E 409 901.6 ORIGINATED BY R McC&A  
DIST 2 HWY 401 BOREHOLE TYPE WASHBORING COMPILED BY M V  
DATUM GEODETIC DATE 54 11 01 TO 54 11 03 CHECKED BY P P

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	W <sub>P</sub>	W	W <sub>L</sub>		
260.9	Ground Surface																
0.0						*											
	Occasional Sand Layers		1	SS	25												
	Occasional Silt Layers																
			2	SS	14												
	CLAYEY SILT, Very Stiff to Stiff																
			3	SS	12												
246.1																	
14.8			4	SS	90												
	SANDY SILT, Very Dense																
241.1																	
240.6	CLAYEY SILT, Very Stiff																
20.3	End of Borehole Note: Formerly BH# 4 of 54 - F - 224C R McC&A: Racey MacCallum and Associates  * Water Level Not Observed																

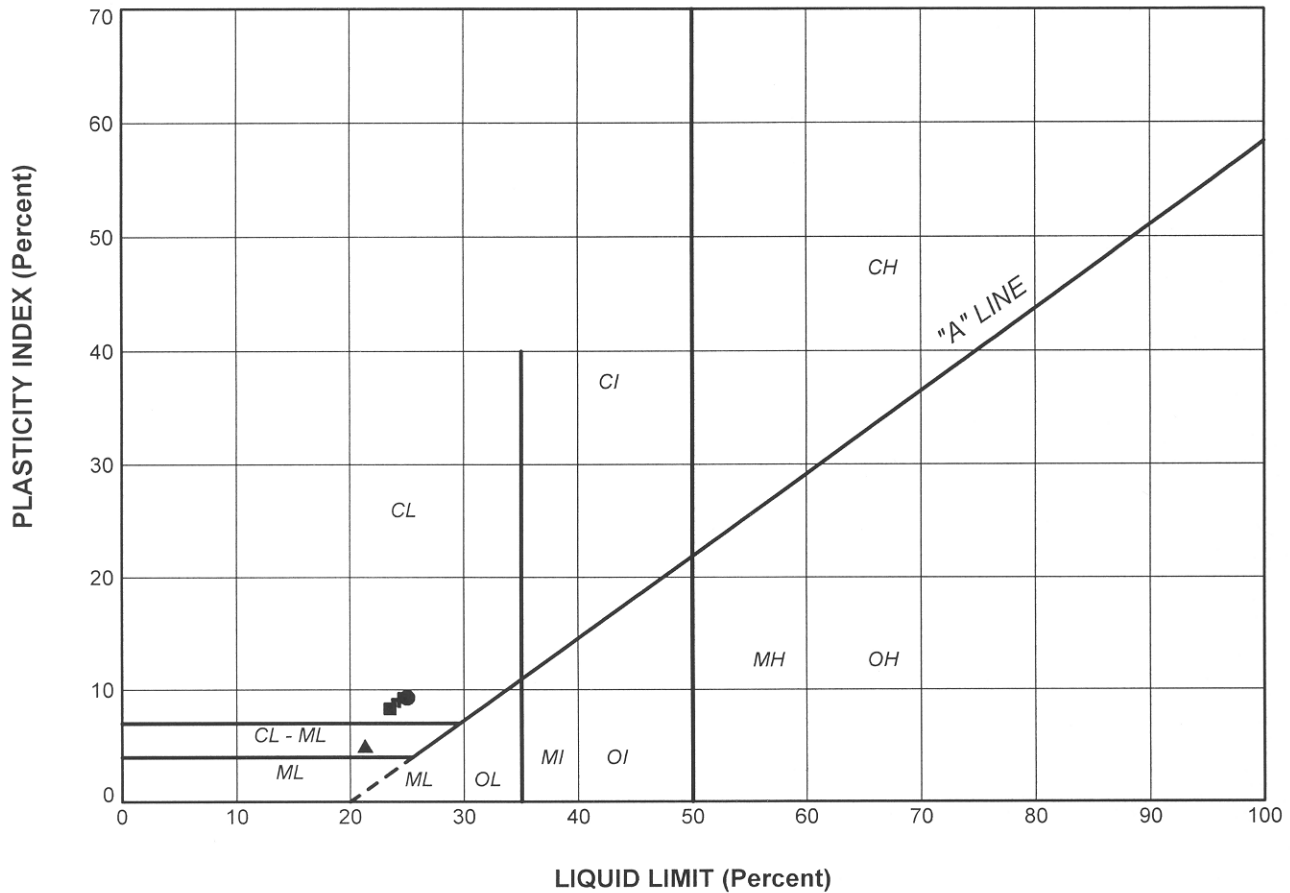
**APPENDIX B**  
**LABORATORY TEST DATA**



LEGEND		
Symbol	Sample	ELEV (m)
●	3	259.0
■	7	255.9
▲	9	254.4
+	14	247.6
◆	5	257.0

PROJECT PROPOSED WELLINGTON RD. BRIDGE HIGHWAY 401, SITE 19-369, GWP 476-89-00				
TITLE <b>GRAIN SIZE DISTRIBUTION</b> <b>CLAYEY SILT</b>				
 <b>Golder Associates</b> LONDON, ONTARIO		PROJECT No. WDF	FILE No. 0013225B.GPJ	SCALE N/A
		DRAWN 6-1-02	REV.	
		CHECK 6-1-02	<b>FIGURE B1</b>	

LDN\_GSD\_NEW\_GLDR\_LDN.GDT



PROJECT				
PROPOSED WELLINGTON RD. BRIDGE HIGHWAY 401, SITE 19-369, GWP 476-89-00				
TITLE				
PLASTICITY CHART				
PROJECT No.		001-3225-1		FILE No.
				0013225B.GPJ
DRAWN		WDF	6-1-02	SCALE
CHECK		Amk	6-1-02	N/A
				REV.



**FIGURE B2**