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FOUNDATION INVESTIGATION AND DESIGN REPORT

Middlesex Road 74 (Westchester Bourne) Underpass
Site Number 19-375
Highway 401 Interchange Improvements/
Structural Replacements
GWP 3053-11-00, Assignment No. 2 (3011-E-0047)
Ministry of Transportation, Ontario - West Region

Submitted to:

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REPORT



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LIST OF SYMBOLS

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PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT

**MIDDLESEX ROAD 74 (WESTCHESTER BOURNE) UNDERPASS
SITE NUMBER 19-375
HIGHWAY 401 INTERCHANGE IMPROVEMENTS/STRUCTURAL REPLACEMENTS
GWP 3053-11-00, ASSIGNMENT No. 2 (3011-E-0047)
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder Associates) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations as part of the preliminary design and 30% detailed design work for GWP 3030-11-00, 3054-11-00, 3053-11-00, 3070-11-00, 3059-11-00, and 3055-11-00. The project involves the preliminary design and 30% detailed design for ten (10) bridges and two (2) culverts, including improvements at five (5) interchanges, of Highway 401.

This report addresses the replacement of the Middlesex Road 74 (Westchester Bourne) Underpass (Site 19-375) and interchange improvements for GWP 3053-11-00.

The purpose of the foundation investigation is to explore the subsurface conditions at the location of the proposed structure replacement by drilling boreholes and carrying out in situ testing and laboratory testing on selected samples. The terms of reference for the scope of work are outlined in the MTO's Request for Proposal and in Golder Associates' proposal P2-1132-0076-P01 dated September 10, 2012. The work was carried out in accordance with our Quality Control Plan for Foundations Engineering dated November 2012.



2.0 SITE DESCRIPTION

2.1 General

The Highway 401 Middlesex Road 74 (Westchester Bourne) Underpass is located just east of the City of London in the Municipality of Thames Centre, Ontario. The structure is located about 1.8 and 3.7 kilometres east and west of Veterans Memorial Parkway and Dorchester Road, respectively. The location of the site is shown on the Key Plan, Figure 1.

For the purposes of this report, Highway 401 and Middlesex Road 74 are assumed to be oriented in an east-west direction and a north-south direction, respectively. This section of Highway 401 is currently a six lane divided highway oriented generally east northeast-west southwest. The highway surface is at approximately elevation 281 metres at the underpass location. Middlesex Road 74 has a pavement surface at about elevation 288 metres. The existing underpass was constructed in 1956 and consists of a single span, concrete rigid frame structure with “tee” type girders. The area adjacent to the site consists of relatively flat-lying agricultural and commercial lands. A municipal works yard is located to the southeast and a truck stop is located to the southwest. It is anticipated that the existing underpass structure will be demolished and replaced with a new structure erected approximately 40 metres to the west of the exiting alignment.

2.2 Site Geology

This project lies within the physiographic region known as the Mount Elgin Ridges. The physiographic mapping indicates that the Middlesex Road 74 Underpass site is situated at the transition between a till moraine and a till plain.¹ Geological mapping indicates that the surficial material consists of the Erie Lobe of Port Stanley silty clay till and clayey silt till, covered by thin patches of lacustrine silt in places.²

The rock formation in the area of the site is described as medium brown, microcrystalline limestone of the Dundee Formation which belongs to the Hamilton Group of Middle Devonian Age.³ The bedrock surface is estimated to be at about elevation 218 metres, some 70 metres below the approximate Highway 401 pavement level at the site.

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

² Dreimanis, A., 1963: Pleistocene Geology of the St. Thomas Area (East Half), Southern Ontario. Ontario Department of Mines, Preliminary Geological Map P.606, scale 1:50,000.

³ Sanford, B.V., 1969: Geology, Toronto-Windsor Area, Ontario. Geological Survey of Canada, Map 1263A, scale 1:250,000.



3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out between March 5 and 11, 2015 during which time four boreholes, numbered 409 to 412, were drilled. Augerholes 1 and 6 drilled in 1954 for the foundation investigation for the original structure (Geocres Report No. 40I14-81) have been used to supplement the current data. The subsurface stratigraphy encountered in the augerholes was summarized in the text of Geocres Report No. 40I14-81, included in Appendix B for reference.

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

Borehole/ Augerhole	Location (m)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing	Easting		
409	4 757 436	418 662	283.8	8.5
410	4 757 426	418 681	284.2	29.4
411	4 757 331	418 673	282.3	29.4
412	4 757 330	418 692	282.2	8.1
1 (40I14-81)	4 757 417	418 697	280.4	7.3
6 (40I14-81)	4 757 395	418 723	280.1	7.9

The current investigation was carried out using truck mounted drilling equipment supplied and operated by a specialist drilling contractor. In the boreholes, samples of the overburden were obtained at generally 0.75 or 1.5 metre intervals of depth using 50 millimetre outside diameter split spoon sampling equipment in accordance with the standard penetration test (SPT) procedures of ASTM D1586. According to ASTM D1586, the SPT resistance, or N value, is defined as the number of blows required by a 63.5 kilogram hammer dropped from a height of 760 millimetres to drive a split-spoon sampler a distance of 300 millimetres, after an initial 150 millimetres of penetration. The samples used in the investigation limit the maximum particle size that can be sampled and tested to about 40 millimetres. Therefore particles that may exist within the soils that are larger than this dimension will not be sampled or represented in the grain size distributions. Larger particle sizes including cobbles and boulders are known to be present in the native till materials as discussed in the text of this report. The results of the SPT testing as presented on the Record of Borehole sheets and in Section 4 of the report are not factored to account for the use of an automatic hammer. In-situ vane shear strength testing was carried out in accordance with ASTM D2573 in the softer cohesive soils as appropriate.

The current boreholes were terminated between about 8 and 29 metres below the existing ground surface. Groundwater conditions in the boreholes were observed throughout the drilling operations and a piezometer was installed in borehole 410 as indicated on the corresponding Record of Borehole sheets. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).



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The field work was monitored on a full-time basis by an experienced Golder Associates staff member who also located the boreholes in the field, monitored the drilling, sampling and in situ testing operations, and logged the boreholes. The samples were identified in the field, placed in labelled containers, and transported to our London laboratory for further examination and testing. Index and classification tests, consisting of water content determinations, Atterberg limits determinations, and grain size distribution analyses were carried out on selected samples. The results of the testing are shown on the Record of Borehole sheets and in Appendix A.

The locations of the boreholes are shown on the Record of Borehole sheets and on Drawing 1, attached.



4.0 SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

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

The boreholes drilled at the site generally encountered surficial topsoil over a layer of silt to clayey silt, over a deposit of clayey silt glacial till which was underlain by layers of silt, clayey silt and sand and gravel which was, in turn, underlain by a further deposit of clayey silt till.

The locations and elevations of the boreholes, together with the interpreted stratigraphic profiles, are shown on Drawing 1. Detailed descriptions of the subsurface conditions encountered in the boreholes are provided on the Record of Borehole sheets and are summarized in the following sections.

The classification of the materials described in the augerholes advanced for Geocres Report No. 40I14-81 have been revised to be consistent with current soils classification procedures based on comparison of the stratigraphy in adjacent boreholes. Materials described as sandy clay in augerholes 1 (40I14-81) and 6 (40I14-81) have been classified as sandy silt, some clay and clayey silt, some sand, respectively. Materials described as light clay, slightly sandy and stoney have been classified as clayey silt glacial till. These classifications have been reflected in the following report sections and the inferred profile, Drawing 1.

4.1.1 Topsoil

Between 150 and 520 millimetres of topsoil was encountered at the ground surface of each of the boreholes. Materials designated as topsoil in this report were classified solely based on visual and textural evidence. Testing of organic content or for other nutrients was not carried out. Therefore, the use of materials classified as topsoil cannot be relied upon for support and growth of landscaping vegetation.

4.1.2 Silt to Sandy Silt

Layers of compact silt to sandy silt, 1.1 to 3.0 metres thick, were encountered in boreholes 409 and 410 and augerhole 1 (40I14-81) beneath the topsoil between elevations 280.1 and 284.0 metres. Measured N values from standard penetration testing in the near surface silt to sandy silt were 14 blows per 0.3 metres. A sample of the silt from borehole 409 was noted to contain topsoil pockets and had a water content of about 47 per cent.

Layers of dense to very dense silt were also encountered beneath layers of clayey silt till in boreholes 410 and 411. The silt layers were 3.6 and 4.6 metres thick, respectively, and were encountered at elevations 269.9 and 269.5 metres. Measured N values in the lower silt layers were 40 to greater than 100 blows per 0.3 metres. Samples of the lower silt had water contents of about 19 and 20 per cent.



Grain size distribution curves for samples of the silt are provided on Figure A-1.

4.1.3 Clayey Silt

Layers of stiff upper clayey silt, 0.9 to 4.0 metres thick, were encountered in boreholes 411 and 412 and augerhole 6 (40I14-81) beneath the topsoil. Measured N values from standard penetration testing in the near surface clayey silt were 9 and 14 blows per 0.3 metres. A sample of the clayey silt from borehole 412 had a water content of about 15 per cent, liquid and plastic limits of about 22 and 15 per cent, respectively, and a plasticity index of about 7 per cent, indicating clay of low plasticity.

Layers of very stiff to hard clayey silt were also encountered beneath the lower layers of silt in boreholes 410 and 411. The lower clayey silt layers were 2.9 and 2.6 metres thick, respectively, and were encountered at elevations 266.3 and 265.0 metres. Measured N values in the lower clayey silt layers were 15 to 40 blows per 0.3 metres. Samples of the clayey silt had water contents of about 20 and 25 per cent. Two Atterberg limits determinations yielded liquid limits of about 29 and 34 per cent, plastic limits of about 18 and 20 per cent, and plasticity indices of about 11 and 14 per cent, indicating clay of low plasticity.

Grain size distribution curves for samples of the clayey silt are provided on Figure A-2. The results of Atterberg limits testing are provided on Figure A-4.

4.1.4 Sand and Gravel

A 0.6 metre thick layer of dense sand and gravel was encountered in borehole 411 beneath the lower clayey silt layer at elevation 262.4 metres. A measured N value in the sand and gravel was 40 blows per 0.3 metres.

4.1.5 Clayey Silt Glacial Till

Firm to hard clayey silt glacial till was encountered beneath the near surface silt, sandy silt and clayey silt in each of the boreholes between elevations 276.1 and 282.9 metres. Boreholes 409 and 412 and augerholes 1 (40I14-81) and 6 (40I14-81) were terminated in the upper clayey silt till after penetrating the layer for 4.0 to 7.2 metres. In boreholes 410 and 411, the upper clayey silt till was 13.0 and 11.3 metres thick, respectively. Measured N values in the clayey silt till ranged from 6 to 52 blows per 0.3 metres. An in-situ vane shear strength test carried out in the upper clayey silt till in borehole 409 yielded a shear strength of greater than 144 kilopascals. Samples of the upper clayey silt till had water contents of about 13 to 21 per cent. Seven Atterberg limits determinations yielded liquid limits of about 21 to 26 per cent, plastic limits of about 14 to 18 per cent, and plasticity indices of about 6 to 9 per cent, indicating clay of low plasticity.

A lower deposit of hard clayey silt glacial till was encountered beneath the lower clayey silt in borehole 410 and beneath the sand and gravel in borehole 411 at elevations 263.4 and 261.8 metres, respectively. The lower clayey silt till layer was penetrated for 8.6 and 8.8 metres in boreholes 410 and 411 before the boreholes were terminated. Measured N values in the clayey silt till ranged from 45 to greater than 100 blows per 0.3 metres. Samples of the lower clayey silt till had water contents of about 11 and 12 per cent. Three Atterberg limits determinations yielded



liquid limits of about 19 to 20 per cent, plastic limits of about 12 to 13 per cent, and plasticity indices of about 6 to 8 per cent, indicating clay of low plasticity.

Borehole 409 was terminated due to auger refusal on a probable boulder at approximately elevation 275.3 metres. The presence of cobbles was inferred in the lower clayey silt till layer. Cobbles and boulders should be anticipated within both the upper and lower clayey silt till layers based on the depositional history of glacial till materials.

Grain size distribution curves for samples of the clayey silt till are provided on Figure A-3. The results of Atterberg limits testing are provided on Figure A-4.

4.2 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling. Also, a piezometer was installed in borehole 410, as shown on the Record of Borehole sheets. Encountered and measured groundwater levels are summarized in the following table.

Borehole	Ground Surface Elevation (m)	Encountered Groundwater Level		Measured Groundwater Elevation (m)	
		Depth (m)	Elevation (m)	March 11, 2015	April 1, 2015
409	283.82	*	*	-	-
410	284.21	*	*	272.48	272.53
411	282.32	2.13	280.19	-	-
412	282.17	*	*	-	-

* Groundwater level not established during drilling.

The above-noted encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions. Based on the measured and encountered groundwater levels, the inferred groundwater level at the site is at about elevation 272.5 metres within the lower silt layer and varies between elevations 280.0 and 281.0 metres within the upper clayey silt till. Groundwater levels are expected to fluctuate seasonally and are expected to be higher during periods of sustained precipitation or during spring melt conditions.

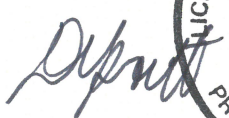



5.0 MISCELLANEOUS


This investigation was carried out using equipment supplied and operated by Lantech Drilling Services Inc., an Ontario Ministry of Environment and Climate Change licensed well contractor. The field operations were supervised by Mr. Lubo Kosc, P.Eng. under the direction of Mr. David J. Mitchell.


The laboratory testing was carried out at Golder Associates' London laboratory under the direction of Mr. Chris M. Sewell. The laboratory is an accredited participant in the MTO Soil and Aggregate Proficiency Program and is certified by the Canadian Council of Independent Laboratories for testing Types C and D aggregates. This report was prepared by Ms. Nicole A. Gould, P.Eng., under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. This report was reviewed by Mr. Michael E. Beadle, P.Eng. an Associate and Senior Geotechnical Engineer with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.


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PART B
PRELIMINARY FOUNDATION INVESTIGATION REPORT

**MIDDLESEX ROAD 74 (WESTCHESTER BOURNE) UNDERPASS
SITE NUMBER 19-375
HIGHWAY 401 INTERCHANGE IMPROVEMENTS/STRUCTURAL REPLACEMENTS
GWP 3053-11-00, ASSIGNMENT No. 2 (3011-E-0047)
MINISTRY OF TRANSPORTATION, ONTARIO - WEST REGION**



6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides our recommendations on the foundation aspects of the design of the replacement of the existing Middlesex Road 74 (Westchester Bourne) Underpass (Site 19-375). This Foundation Investigation and Design Report, with the interpretation and recommendations, are intended for the use of the Ministry of Transportation, Ontario and shall not be used or relied upon for any other purposes or by any other parties including the construction or Design-Build Contractor. The Design-Build Contractor must make their own interpretation based on the factual data in Part A of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of factual information provided as it may affect equipment section, proposed construction methods and scheduling.

Westchester Bourne will be realigned to the west of the existing Highway 401 crossing. The Preliminary General Arrangement drawing dated June 2016 indicates that the replacement structure will be 98 metres long. The proposed two-span structure will have integral abutments supported on deep foundations and a median pier with shallow foundations. Compared to the existing bridge, the new abutments will be located further from the centreline of Highway 401 to permit future widening to an ultimate ten lane configuration. Realignment of Westchester Bourne will require a grade raise of approximately 4.9 metres at the north abutment and 4.7 metres at the south abutment. The grade of Highway 401 will remain unchanged at approximately elevation 281 metres.

6.2 Existing Structure

It is understood that the existing underpass structure, constructed in 1956, is a single span bridge over Highway 401, located east of London, Ontario. The underpass is a rigid frame structure with a span of 35 metres and a curb-to-curb width of 14.33 metres. The deck is composed of eight reinforced concrete "Tee" girders at 2.27 metre spacings on centre.

Based the original contract drawings, the abutments and wing walls are founded on shallow footings. The wing walls are connected to the abutment walls on one side and supported vertically with discrete columns founded on spread footings located approximately 9.8 metres from the abutment face. The vertical supports are connected horizontally with a buried reinforced concrete tie beam, supported at mid-span by a vertical column founded on a spread footing. Based on Department of Highways, Ontario (DHO) Drawing Nos. D-3497-1 and D-3497-2, dated January 20, 1955, the underside of footing elevation for the abutments and wing wall footings are summarized as follows:

Bridge Element	Founding Elevation (m)
North Abutment	277.94
Northwest Wing Wall	277.02
Northeast Wing Wall	277.94
South Abutment	277.36
Southwest Wing Wall	277.36
Southeast Wing Wall	276.44



It should be noted that the abutment elevation drawings indicate that mass concrete with a compressive strength of 10 megapascals was to be placed in sub-excavations to a depth of 0.9 metres at the east quarter of the south abutment footing and the west quarter of the north abutment footing. DHO Drawing Nos. D-3497-1 and 2 have been included in Appendix C for reference.

It appears that the original designers gave consideration to founding the structure on spread footings with an allowable bearing capacity (working stress design) of 200 kilopascals (4,000 pounds per square foot). Six percussion tests carried out by the Franki Compressed Pile Company of Canada Limited for Geocres Report No. 40I14-81 experienced refusal between elevations 272.8 and 274.6 metres in the very stiff to hard clayey silt till.

The existing bridge structure is to be replaced due to deficient conditions and in order to accommodate the future widening of Highway 401.

6.3 Bridge Foundations

The subsurface soil conditions at the site typically consist of surficial topsoil over a layer of silt, sandy silt or clayey silt over deposits of clayey silt glacial till. Native soils were encountered in the boreholes between about elevations 279.8 and 284.0 metres. The elevation of Highway 401 at the site at approximately 281 metres. The inferred groundwater level at the site is at about elevation 272.5 metres within the lower silt layer and between elevations 280.0 and 281.0 metres in the upper clayey silt till layer.

Integral abutments are typically founded on steel H-piles with lateral loads in the direction of the weak axis. Alternatively, integral abutments may be founded on concrete filled steel tube piles provided the additional resistance to lateral loads is adequately considered during structural design. Driven steel H-piles or closed-end concrete filled steel tube piles, drilled caissons or conventional shallow foundations are considered appropriate for support of semi-integral or conventional abutments at the site. If a multi-span structure is designed for the new bridge, piers may be supported on shallow spread footings, driven piles or drilled caissons. Recommendations for each of these foundation systems are provided in subsequent report sections below.

A comparison of foundation alternatives is presented in Table I following the text of this report. The relative costs are compared using the most economical foundation option (shallow foundations) as the base cost. The estimated relative costs are meant to provide an order of magnitude comparison amongst the alternatives and are not indicative of actual construction costs. The preferred technical alternative from a foundation engineering perspective is to found the structure on conventional shallow footings. It may be cost-effective to support the piers on shallow foundations, though further investigation in the area of any proposed piers will be required.



6.3.1 Shallow Foundations

Geotechnical Axial Resistance

The abutments, piers, and retaining walls of the replacement bridge may be founded on spread/strip footings on the clayey silt till founded at or below the following elevations:

- North Abutment – 282.5 metres;
- Pier – 276 metres; and
- South Abutment – 281 metres.

A factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kilopascals and a geotechnical reaction at Serviceability Limit States (SLS) of 400 kilopascals may be used for footings founded on the very stiff to hard clayey silt till. The geotechnical resistances provided assume that the embankments are constructed in advance of the abutments. As discussed further in Section 6.7, some 60 millimetres of settlement is expected for the new embankment fills, the bulk of which are expected to occur during construction. A practical range of footing width between 3 and 6 metres can be used for preliminary design.

The SLS value corresponds to an estimated total settlement of 25 millimetres. The above geotechnical resistance and reaction are given for vertical loading and do not consider the effects of inclined or eccentric loads. Depending on the selected design, an in-depth investigation of the effects of eccentric or inclined loadings may be necessary. In the absence of a detailed analysis of the applied loads, which are typically not known at the Preliminary Design stage, the geotechnical resistance and reaction can be adjusted for the effects of inclined loads using the reduction factors presented in Clause 6.7.4 of the 2006 edition of the Canadian Highway Bridge Design Code (CHBDC 2006). The eccentricity of the resultant load shall be limited as discussed in Clause 6.7.3.4 of the CHBDC 2006. Refined structural and geotechnical analyses which consider eccentricity and inclination should be carried out at the Detailed Design stage, as required.

Resistance to Lateral Forces

Resistance to lateral forces/sliding between the concrete spread/strip footings and the native, undisturbed subsoil should be calculated in accordance with Section 6.7.5 of the CHBDC 2006. Assuming that the founding soils are not loosened/disturbed during excavation and footing construction, an angle of friction between the mass cast-in-place concrete and the founding soils of 28° and corresponding unfactored coefficient of friction, $\tan \delta$, of 0.53 may be used.

Frost Protection

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



6.3.2 Deep Foundations

The abutments and piers for the replacement structure could be founded on 324 millimetre outside diameter (OD) by 9.5 millimetre wall thickness concrete filled steel tube piles, steel HP 310 x 110 piles, or drilled caissons. Integral abutments could be founded on concrete filled steel tube piles or steel HP 310 x 110 piles. Piles supporting integral abutments require pre-augering and placement of a corrugated steel pipe (CSP) liner filled with loose uniform sand around the upper 3 metres of the pile to reduce resistance to lateral movement. Integral abutments are typically founded on steel H-piles with lateral loads in the direction of the weak axis. Alternatively, integral abutments may be founded on concrete filled steel tube piles provided the additional resistance to lateral loads is adequately considered during structural design.

Geotechnical Axial Resistance – Driven Piles

For design, the factored axial geotechnical resistances at ULS and geotechnical reactions at SLS for HP 310 x 110 piles and concrete filled steel tube piles with 324 millimetre OD and 9.5 millimetre wall thickness driven closed ended to practical refusal at or below the elevations shown are provided in the following table. The SLS values correspond to an estimated total settlement of 25 millimetres. The pile driving hammer should be selected such that the energy delivered to the piles is sufficient to achieve the termination criteria (ultimate geotechnical resistances and tip elevations) stated in the Contract Documents.

Pile Type and Location	Assumed Cut-off Elevation (m)	Founding Strata	Maximum Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
HP 310 x 110	286	Hard Clayey Silt Till	258	1,000	800
324 mm OD x 9.5 mm concrete filled steel tube		Hard Clayey Silt Till	261 (N) 258 (S)	800	650

The above cut-off elevation has been assumed based on the existing elevations at the site.

The actual pile penetration and pile set characteristics will be dependent, to some extent, upon the driving equipment selected by the contractor. It is recommended that, following the selection of the driving equipment, the piling contractor submit for review to the geotechnical engineer the proposed pile driving criteria based on the characteristics of the hammer and equipment intended for use.



Geotechnical Axial Resistance – Drilled Caissons

For preliminary design, the vertical load carrying capacity of the caissons derived from skin friction may be calculated using the following equation:

$$Q_s = \pi B \Delta z f_{SN}$$

where Q_s is the nominal skin friction in kilonewtons, B is the shaft diameter in metres, Δz is the thickness of the soil layer over which resistance is calculated in metres and f_{SN} is the nominal unit skin friction in kilopascals. The upper 1.2 metres below the ground surface should be neglected to account for frost action. Any portion of the caisson within fill materials should also be neglected.

Assuming that caissons greater than 1 metre in diameter will be used, the component of the vertical load carrying capacity that may be derived from end bearing in the native soils may be calculated using the following equation:

$$Q_b = q_{BN} A_t$$

where Q_b is the toe resistance in kilonewtons, q_{BN} is the nominal unit base resistance in kilopascals and A_t is the cross-sectional area of the caisson in square metres. Caissons founded in the native soils may be designed using the nominal unit side and base resistances provided in the following table. The stratigraphy presented in the table below has been simplified for the purposes of this report.

Soil Type	Elevation (m)	f_{SN} (kPa)	q_{BN} (kPa)	Unit Weight (kN/m ³)
Fill	Where applicable	-	-	19.0
Sandy Silt (North)	281.8 to 282.9	2	150	19.5
Clayey Silt (South)	281.8 to 282.9	2	40	19.5
Clayey Silt Till	269.7 to 281.8	20	1,500	21.0
Silt	265.6 to 269.7	50	2,800	20.0
Clayey Silt	263.4 (N) to 265.6 262.4 (S)	65	1,350	20.0
Sand and Gravel (South)	261.8 to 262.4	160	6,000	22.0
Clayey Silt Till	252.0 to 263.4 (N) 261.8 (S)	90	4,500	21.0

The ultimate resistance Q_u is the sum of Q_b and Q_s . A resistance factor of 0.5 should be applied to Q_u to obtain the factored axial resistance at ULS.



It is expected that caissons may extend to between elevations 269 to 275 metres. A top of caisson elevation of 286 metres has been assumed. For the purposes of preliminary design, the following factored geotechnical resistances as ULS and geotechnical reactions at SLS may be used for caissons with a nominal diameter of 1.2 metres founded at the elevations recommended in the table below.

Founding Strata	Design Tip Elevation (m)	Embedded Caisson Length (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
Very Stiff to Hard Clayey Silt Till	275	11	1,100	740
Very Dense Silt	269	17	1,700	1,400

Frost Protection

The pile caps should be provided with a minimum of 1.2 metres of soil cover, or thermal equivalent, above the underside of pile cap elevation for frost protection.

Downdrag Load (Negative Skin Friction)

The new approach embankments will cause consolidation settlement of the underlying clayey silt till deposit as a result of the embankment surcharge. A grade raise of approximately 5 metres is proposed at each abutment. Considering the pre-consolidated nature and the low compressibility of the clayey silt till, above elevation 270 metres, negligible skin friction is expected to develop along the new piles. Any potential downdrag loads can be reduced or eliminated by installing the piles a minimum of one month after fill placement.

Resistance to Lateral Loads

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. The stratigraphy presented in the table below has been simplified for the purposes of this report.

The horizontal reaction to the pile can be estimated using the following equation and ranges in subgrade reaction coefficient:

$$\begin{aligned} k_h &= \text{coefficient of horizontal subgrade reaction (MPa/m)} &= n_h (z/d) &\text{for cohesionless soils} \\ & &= \frac{67 S_u}{d} &\text{for cohesive soils} \end{aligned}$$

where:

- d = pile width or diameter (m)
- n_h = constant of horizontal subgrade reaction (MPa/m)
- S_u = undrained shear strength of the soil (MPa)
- z = depth below ground surface grade (m)



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT MIDDLESEX ROAD 74, SITE NUMBER 19-375

The range in values provided below reflects the variability in subsurface conditions as well as the two extremes of design: the requirement for flexibility if integral abutments are selected, and the requirement for lateral support in the cases of non-integral abutments or pier foundations.

Soil Type	Elevation (m)	n_h (MPa/m)	S_u (kPa)
Granular backfill in CSPs for integral abutments	Where applicable	5 - 10	-
Sandy Silt (North)	281.8 to 282.9	4 - 8	-
Clayey Silt (South)	281.8 to 282.9	-	100
Clayey Silt Till	269.7 to 281.8	-	150
Silt	265.6 to 269.7	14 - 21	-
Clayey Silt	263.4 (N) to 265.6 262.4 (S)	-	150
Sand and Gravel (South)	261.8 to 262.4	12 - 16	-
Clayey Silt Till	252.0 to 263.4 (N) 261.8 (S)	-	200

The lateral resistances for the various foundation options are summarized in the following table.

Pile Type	Lateral Resistance	
	Factored ULS (kN)	SLS (kN)
Integral abutments		
- HP 310 x 110, weak axis bending	25	*
- 324 mm OD x 9.5 mm tube	55	*
Semi-Integral or Conventional abutments		
- HP 310 x 110, strong axis bending	220	70
- 324 mm OD x 9.5 mm tube (concrete filled)	160	80
Concrete Caissons, 1.2 metre diameter	1,000	360

* Load to mobilize 10mm horizontal displacement is greater than ULS value, therefore ULS value governs.

The lateral resistances are based on Broms Method from "Design and Construction of Driven Pile Foundations Workshop Manual – Volume 1, FHWA, Pub. No. FHWA H1 97-013, Revised November 1998". Free-headed piles were assumed, with the horizontal load for semi-integral or conventional abutments and pier piles applied at the underside of abutment footing and for integral abutments at a 3 metre stick-up. An ultimate compressive strength of 32 megapascals was assumed for the concrete filled steel tube piles and caissons. The SLS values are based on 10 millimetres of deflection at the ground surface.



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:

<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

6.4 Liquefaction Potential and Seismic Analysis

6.4.1 Seismic Parameters

The site is located near the city of London, Ontario. According to Table A.3.1.1 of the CHBDC 2006, the zonal acceleration ratio, A , applicable to this site is 0.00. The corresponding acceleration-related seismic zone, Z_a is 0.

The replacement bridge is in Seismic Performance Zone (SPZ) 1, based on a CHBDC 2006 classification as an "Emergency Route Bridge". Based on the site stratigraphy, the soil profile type is categorized as Type II with a seismic site response coefficient, S , of 1.2 based on the CHBDC 2006 criteria. Analysis of bridges in SPZ 1 is not a requirement of the CHBDC 2006. However, design forces for restraining elements and bridge support lengths must meet the minimum requirements as outlined in CHBDC Clause 4.4.5.1.

6.4.2 Seismic Hazard Assessment

A preliminary screening of the soil stratigraphy was conducted using the procedure outlined in the FHWA recommended procedures.⁴ Although saturated granular materials are present at depth, they were found to have a normalized N value of greater than 22 blows per 0.3 metres. The liquefaction potential is considered to be relatively low based on the soil profile type, age of the deposits, relative density, and the historically low seismicity. Therefore, a detailed evaluation of the liquefaction potential of the foundation soils, impact of liquefaction on the bridge foundations, and the effect of seismic forces on embankment stability is not considered warranted unless the structure is deemed to be a lifeline bridge.

⁴ FHWA, 1997: "Design Guidance: Geotechnical Earthquake Engineering For Highways. Volume I – Design Principles." *Geotechnical Engineering Circular No. 3: FHWA-SA-97-076*, Washington, D.C.



6.5 Lateral Earth Pressures for Design

The lateral pressures acting on the bridge abutments and associated wing/retaining walls, will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. The following recommendations are made concerning the design of the abutments, in accordance with the CHBDC 2006:

- Select, free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type III but with less than 5 per cent passing the No. 200 sieve should be used as backfill behind the abutments and walls. This fill should be compacted in loose lifts not greater than 200 millimetres in thickness in accordance with OPSS.PROV 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the abutment granular backfill requirements with respect to subdrains and frost taper should be in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150 and 3190.100.
- A compaction surcharge equal to 12 kilopascals should be included in the lateral earth pressures for the structural design of the abutment walls in accordance with CHBDC 2006 Figure 6.6. Compaction equipment should be used in accordance with OPSS.PROV 501.
- The granular fill may be placed either in a zone with a width equal to at least 1.2 metres behind the back of the stem (Case a from Commentary on CHBDC 2006 Figure C6.20) or within the wedge-shaped zone defined by a line drawn at a maximum 1 horizontal to 1 vertical extending up and back from the rear face of the footing (Case b from Commentary on CHBDC 2006 Figure C6.20).
- For Case a, the restrained case, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM):

Soil unit weight: 20 kN/m³

Coefficients of lateral earth pressure:
At rest, K_0 0.50

- For Case b, the unrestrained case, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	<u>GRANULAR A</u>	<u>GRANULAR B</u> <u>Type III</u>
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
Active, K_a	0.27	0.31
Passive, K_p	3.7	3.3

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.



- For integral abutments, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. A resistance factor equal to 0.5 should be applied to the calculated total passive resistance in accordance with Table 6.1 of the CHDBC.

It should be noted that the above design parameters assume level backfill and ground surface behind the wall. The lateral earth pressure coefficients should be adjusted if there is sloping ground at the back of the wall.

6.6 Construction Considerations

6.6.1 Shallow Foundations

The founding soils are sensitive to disturbance and loosening due to water seepage and/or ponding and construction equipment or foot traffic when damp to wet. Placement of a concrete working slab (100 millimetres thick of 20 megapascals concrete) will be required at the base of the excavations for the footing areas. Exposure without protection using the working slab may result in loosening or softening of the founding soils. The cleaned excavation base should be inspected by a Quality Verification Engineer (QVE) qualified in geotechnical engineering prior to placing the working slab. It is recommended that the footing excavations be carried out such that the final 0.5 metres of excavation is completed with the geotechnical personnel on site and the working slab be placed immediately after footing inspection.

6.6.2 Deep Foundations

Cobbles and boulders should be expected in the soils at the site and which may impact pile driving operations. A Non-Standard Special Provision (NSSP) should be added to the Contract Documents to alert the contractor to the need for special procedures to deal with cobbles, boulders and other obstructions during pile installation.

Deep foundations should be installed and monitored in accordance with OPSS 903, as well as OPSD 3000.150, 3001.150, and SS103-11 (Pile Driving Control) for the driven piles. The H-piles and steel tube piles should to be equipped with Type I driving shoes as shown in OPSD 3000.100 and 3001.100, respectively.

6.7 Embankments

It is anticipated that new approach embankments will be required for the new structure. All surficial topsoil, organic, loose, soft and/or otherwise deleterious materials should be stripped from areas of proposed embankments. Prior to placement of embankment fill material, the exposed subgrade should be proofrolled under the direction of the geotechnical QVE. The embankment fills should consist of an approved granular borrow such as SSM, except for the top approximately 0.5 to 1.0 metres where pavement structure will be placed. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts and properly benched into the existing embankments, as appropriate, and adequately compacted. Embankments shall be constructed in accordance with OPSS 206 and OPSD 208.010.



The MTO's settlement criteria⁵ for new non-freeway embankments on compressible soils is summarized in the following table:

Distance from Abutment (m)	0	0 – 20	20 – 50	50 – 75	75+
Maximum Allowable Settlement (mm)	5	25	50	100	200

The maximum acceptable rate of differential or transverse settlement is 200:1.

It is anticipated that construction of the approach embankments will require grade raises in the order of 5 metres resulting in pavement surface elevation along Westchester Bourne that is approximately 8.5 and 7.5 metres above the level of Highway 401 at the north and south abutments, respectively. The width of the embankment crests will be about 22 metres.

A preliminary settlement analysis was carried out using the results of the boreholes and estimates of soil properties from similar soils in the surrounding area. Total settlement at the embankment crests was estimated to be 60 millimetres. If shallow foundations are constructed, the total settlement is expected to range between 60 and 80 millimetres depending on the proposed foundation width. Due to the preconsolidated nature and relatively low compressibility of the underlying deposits, it is expected that much of the settlement will be completed during construction. The maximum estimated post construction settlement is 25 millimetres for the embankment and ranges from 15 to 25 millimetres for the shallow foundations depending on footing width. Although the post construction settlement will be within the MTO's settlement criteria, it is possible that the differential settlement between the embankment and the abutment may be between 5 and 10 millimetres. In order to minimize the effect of settlement of the approach embankments relative to the abutments, consideration may be given to constructing the embankments about one month in advance of bridge construction. No surcharging is considered to be necessary. These settlement estimates are very preliminary and should be refined during detail design using site specific soil properties.

Embankments constructed with SSM and founded on the compact silt to sandy silt or stiff clayey silt are expected to be stable and may be designed with a Factor of Safety against slope failure of 1.3 for embankments with side slopes no steeper than 2 vertical to 1 horizontal. Embankments greater than 8 metres in height should be constructed with a 2 metre wide bench at mid-height.

6.8 Excavations and Temporary Cut Slopes

6.8.1 General

Excavations for the pile caps or shallow foundations will penetrate the existing topsoil and silt into the clayey silt till. The groundwater level at the site is expected to be at about elevation 272.5 metres within the lower silt layer and varying between elevations 280.0 and 281.0 metres within the upper clayey silt till. The excavations are not expected to extend below the groundwater level; however, minor seepage should be anticipated. Seepage volumes are expected to be low due to the presence of clayey soil. If necessary, groundwater control may be

⁵ MTO, 2010: Embankment Settlement Criteria for Design. July 2, 2010.



achieved by pumping from properly filtered sumps. Sumps should be maintained outside of the actual pile cap and/or abutment limits. Surface water runoff should be directed away from the excavations at all times.

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. Any native cohesionless soils below the groundwater level would be classified as Type 3 soils. The native clayey materials and cohesionless soils above the groundwater level would be classified as Type 2.

6.8.2 Temporary Roadway Protection

Temporary road protection systems will be required where space is restricted and will not permit open cuts, to support the sides of the excavation and permit the use of vertical cuts. The design and limits of the systems are to be determined by the contractor. Design and construction of temporary roadway protection should to be carried out to Performance Level 2 in accordance with OPSS.PROV 539.

Temporary support systems could consist of soldier piles and lagging where the H-piles would be driven to a suitable depth and horizontal lagging installed as the excavation proceeds, or of driven steel sheet piling. Support to the systems could be in the form of struts and walers or rakers and anchors. 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.



7.0 RECOMMENDATIONS FOR DETAIL DESIGN

Conventional shallow footings are considered the preferred foundation alternative for the pier and abutments of the replacement structure. The preferred replacement alternative from a structural engineering perspective features integral abutments, therefore, use of driven steel H-piles is preferred for support of abutments in this case. A Foundation Investigation and Design Report will need to be prepared during a future assignment to provide appropriate information for future Detail Design. A standard MTO foundation investigation for a bridge structure is considered appropriate for the site. Specifically, a minimum of two boreholes should be advanced, one at each abutment opposite to existing boreholes 410 and 411, as well as at the location of any proposed central piers. The boreholes should be advanced to 3 metres below refusal, as required for a deep foundation alternative which is defined as material for which the SPT N value is greater than 100 blows per 0.3 metres. Sampling and SPT testing should be carried out at intervals of 0.75 metres. Routine soil testing consisting of grain size analyses, water content determinations, and Atterberg limits determinations, where applicable, is considered appropriate. In addition, if embankment settlement is of concern, odometer testing would be required to determine site specific soil properties for refinement of the settlement analysis.


The recommendations given in this Preliminary Foundation Design Report should be expanded upon and updated in the Foundation Design Report for Detail Design in accordance with MTO's standard requirements for foundation engineering assignments. Detailed recommendations should be provided for foundations for the abutments, wingwalls, and any proposed pier(s). Embankment stability and settlement should also be evaluated. The discussion on temporary roadway protection should include lateral earth pressures and effect of ground conditions on shoring construction design.





8.0 MISCELLANEOUS

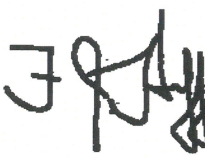
This report was prepared by Ms. Nicole A. Gould, P.Eng. under the direction of the Project Engineer, Ms. Dirka U. Prout, P.Eng. This report was reviewed by Mr. Michael E. Beadle, P.Eng, an Associate and Senior Geotechnical Engineer with Golder Associates. Mr. Fintan J. Heffernan, P.Eng., the Designated MTO Contact and Quality Control Auditor for this assignment, conducted an independent quality review of the report.


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TABLE I

COMPARISON OF FOUNDATION ALTERNATIVES – UNDERPASS REPLACEMENT

Middlesex Road 74, Site 19-375
 Highway 401 Interchange Improvements
GWP 3053-11-00

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS¹	RISKS/ CONSEQUENCES
Spread footings supported on very stiff to hard clayey silt till.	<ul style="list-style-type: none"> • Preferred technical alternative for abutments. • Feasible for central pier(s). 	<ul style="list-style-type: none"> • Least expensive option. • Ease of construction. 	<ul style="list-style-type: none"> • Not compatible with integral abutments. • More settlement expected than with deep foundations. • Larger work area required compared to caissons and driven piles. 	<ul style="list-style-type: none"> • Low 	<ul style="list-style-type: none"> • Relatively low risk. • Deeper excavations required if soil at founding elevation is unsuitable.
End bearing steel H-pile or steel tube pile foundations driven to refusal.	<ul style="list-style-type: none"> • Feasible for abutments. • May be considered for central pier(s). 	<ul style="list-style-type: none"> • High bearing resistance. • Negligible settlement. • Compatible with integral abutments. 	<ul style="list-style-type: none"> • More expensive than shallow foundations. • Can be damaged and deflected by cobbles and boulders within glacial till deposits. • More construction noise and vibration compared to shallow foundations or caissons. • H-Piles cannot be visually inspected at depth. 	<ul style="list-style-type: none"> • Moderate 	<ul style="list-style-type: none"> • Possible pile tip damage if piles are not adequately protected while driving through till deposits. • Variation in pile tip elevations.

COMPARISON OF FOUNDATION ALTERNATIVES

FOUNDATION OPTION	FEASIBILITY	ADVANTAGES	DISADVANTAGES	RELATIVE COSTS¹	RISKS/ CONSEQUENCES
			<ul style="list-style-type: none"> • Integrity inspection requires specialty dynamic testing. 		
<ul style="list-style-type: none"> • Concrete caissons drilled into hard clayey silt till. 	<ul style="list-style-type: none"> • Feasible for abutments but not preferred. • May be preferred for piers. 	<ul style="list-style-type: none"> • Negligible settlement. • Less construction noise and vibration compared to driven piles. • Faster construction and less work space required compared to shallow foundations. • Less potential for caissons to be impeded by cobbles in native till deposits, compared to driven piles. 	<ul style="list-style-type: none"> • Potential for greater settlement compared to driven piles. • Not compatible with integral abutments. • Cannot be visually inspected at depth due to health and safety regulations. 	<ul style="list-style-type: none"> • High 	<ul style="list-style-type: none"> • Cleaning of base could be problematic or overlooked during construction.

- NOTES:
1. The estimated relative costs are intended to provide a comparison between alternatives rather than actual construction costs, and do not take into consideration additional costs such as traffic control, staging and shoring that may influence the total cost.
 2. Table to be read in conjunction with accompanying report.

Prepared By: NG
 Checked By: DUP



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
FoS	factor of safety

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

* Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



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

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.)

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.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	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

(b) Cohesive Soils Consistency

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

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.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

RECORD OF BOREHOLE No 409

1 OF 1

METRIC

PROJECT 12-1132-0076
W.P. 3053-11-00 LOCATION N 4757435.5 , E 418662.2 ORIGINATED BY LK
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY LMK
DATUM GEODETIC DATE March 5, 2015 CHECKED BY NAG

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										
								20	40	60						80	100	WATER CONTENT (%)
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
283.82	GROUND SURFACE																	
0.00	TOPSOIL, silty Black																	
0.15	SILT, some sand, trace gravel, with clayey silt and topsoil pockets Compact Brown		1	SS	14									47	4 11 71 14			
282.51																		
1.31	CLAYEY SILT TILL, trace to some sand, trace to some gravel, with silt partings Firm to hard Brown to grey below about elev. 280.2m		2	SS	12													
			3	SS	9													
			4	SS	21													
			5	SS	36													
			6	SS	14													
			7	SS	52													
			8	SS	6													
275.29	END OF BOREHOLE																	
8.53	Auger refusal at about elev. 275.3m on probable boulder Borehole dry during drilling on March 5, 2015.																	

RECORD OF BOREHOLE No 410

1 OF 3

METRIC

PROJECT 12-1132-0076
W.P. 3053-11-00 LOCATION N 4757425.9 , E 418681.3 ORIGINATED BY LK
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM / MUD ROTARY COMPILED BY LMK
DATUM GEODETIC DATE March 5 - 9, 2015 CHECKED BY NAG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
284.21	GROUND SURFACE													
0.00	TOPSOIL, silty Black													
0.24	SANDY SILT, some clay Compact Brown		1	SS	14		284							
282.87							283							
1.34	CLAYEY SILT TILL, trace to some sand, trace to some gravel Stiff to hard Brown to grey below about elev. 281.2m		2	SS	15		282							
			3	SS	25		281							0 1 74 25
			4	SS	31		280							
			5	SS	37		279							
			6	SS	28		278							
			7	SS	28		277							
			8	SS	17		276							
			9	SS	15		275							
			10	SS	17		274							3 14 61 22
			11	SS	18		273							
			12	SS	35		272							
			13	SS	42		271							
269.88							270							3 7 68 22
14.33	SILT, some sand, trace clay Very dense Grey													

Continued Next Page

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

LDN_MTO_06 12-1132-0076-2001.GPJ LDN_MTO.GDT 12/05/15

PROJECT <u>12-1132-0076</u>		RECORD OF BOREHOLE No 410		2 OF 3		METRIC	
W.P. <u>3053-11-00</u>		LOCATION <u>N 4757425.9 , E 418681.3</u>		ORIGINATED BY <u>LK</u>			
DIST <u> </u> HWY <u>401</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM / MUD ROTARY</u>		COMPILED BY <u>LMK</u>			
DATUM <u>GEODETIC</u>		DATE <u>March 5 - 9, 2015</u>		CHECKED BY <u>NAG</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE												
								20 40 60 80 100		W _P	W	W _L								
	SILT, some sand, trace clay Very dense Grey		14	SS	96		269 <i>Filter sand</i>									0	13	85	2	
							<i>Piezometer</i> 268													
			15	SS	95		267													
266.29																				
17.92	CLAYEY SILT, trace sand Stiff to hard Grey		16	SS	34		266									0	1	56	43	
							265													
			17	SS	15		264													
							263													
263.42																				
20.79	CLAYEY SILT TILL, trace to some sand, trace to some gravel, inferred cobbles Hard Grey		18	SS	47		262													
			19	SS	70		261 <i>Granular bentonite</i>													
							260													
			20	SS	76		259													
			21	SS	98		258													
							257													
			22	SS	76											2	17	60	21	
							256													
254.80			23	SS	67		<i>Cuttings</i> 255													
29.41	END OF BOREHOLE																			
	Water level in piezometer at																			

LDN_MTO_06 12-1132-0076-2001.GPJ LDN_MTO.GDT 12/05/15

Continued Next Page

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

PROJECT 12-1132-0076		RECORD OF BOREHOLE No 411		1 OF 3	METRIC
W.P. 3053-11-00	LOCATION N 4757330.6 , E 418673.0	ORIGINATED BY LK			
DIST _____ HWY 401	BOREHOLE TYPE POWER AUGER, HOLLOW STEM / MUD ROTARY	COMPILED BY LMK			
DATUM GEODETIC	DATE March 10 - 11, 2015	CHECKED BY NAG			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL		
								20	40	60	80	100	W _p	W	W _L							
282.32	GROUND SURFACE					▽																
0.00	TOPSOIL, clayey Black																					
281.80	CLAYEY SILT, some sand, trace gravel, with silt partings Stiff Mottled brown and grey																					
0.52		1	SS	14																		
280.80	CLAYEY SILT TILL, some sand, trace to some gravel Very stiff to hard Brown to grey below about elev. 277.1m		2	SS	18													1	18	54	27	
1.52																						
			3	SS	20																	
			4	SS	31																	
			5	SS	23														6	16	55	23
			6	SS	19																	
		7	SS	25																		
		8	SS	19																		

LDN_MTO_06 12-1132-0076-2001.GPJ LDN_MTO.GDT 12/05/15

Continued Next Page

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

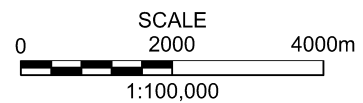
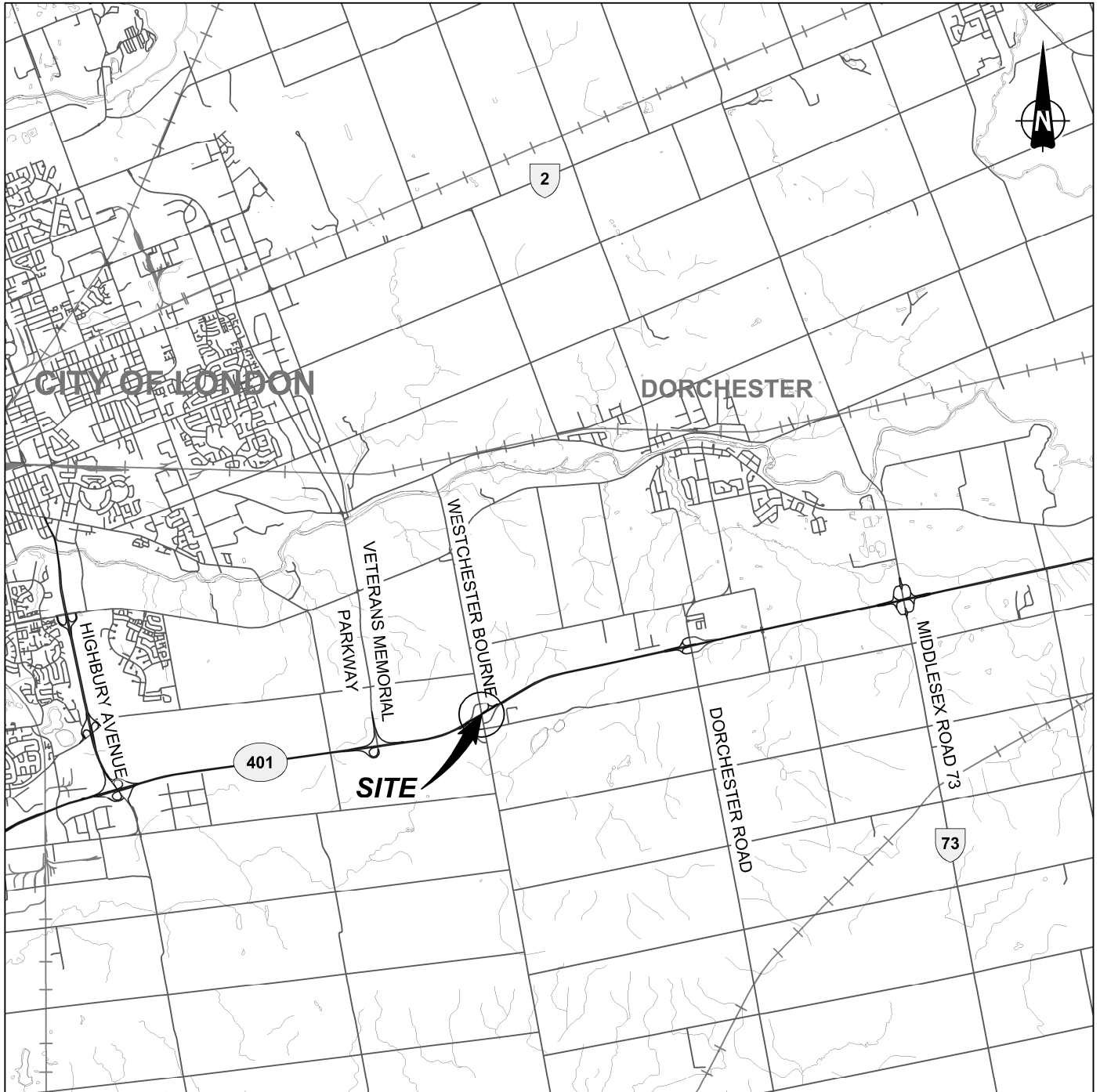
[illegible]

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

PROJECT <u>12-1132-0076</u>		RECORD OF BOREHOLE No 412		1 OF 1		METRIC	
W.P. <u>3053-11-00</u>		LOCATION <u>N 4757330.0 , E 418692.1</u>		ORIGINATED BY <u>LK</u>			
DIST <u> </u> HWY <u>401</u>		BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM</u>		COMPILED BY <u>LMK</u>			
DATUM <u>GEODETIC</u>		DATE <u>March 10, 2015</u>		CHECKED BY <u>NAG</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	W _p	W	W _L						
282.17	GROUND SURFACE																				
0.00	TOPSOIL, silty Black																				
0.24	CLAYEY SILT, sandy to some sand, trace gravel Stiff Mottled brown and grey		1	SS	9												3	26	56	15	
			2	SS	9																
280.04																					
2.13	CLAYEY SILT TILL, some sand, trace to some gravel Stiff to very stiff Brown to grey below about elev. 278.5m		3	SS	10																
			4	SS	15																
			5	SS	13																
			6	SS	12																
			7	SS	25																
			8	SS	15																
274.09																					
8.08	END OF BOREHOLE																				
	Borehole dry during drilling on March 10, 2015.																				

LDN_MTO_06 12-1132-0076-2001.GPJ LDN_MTO.GDT 12/05/15



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT

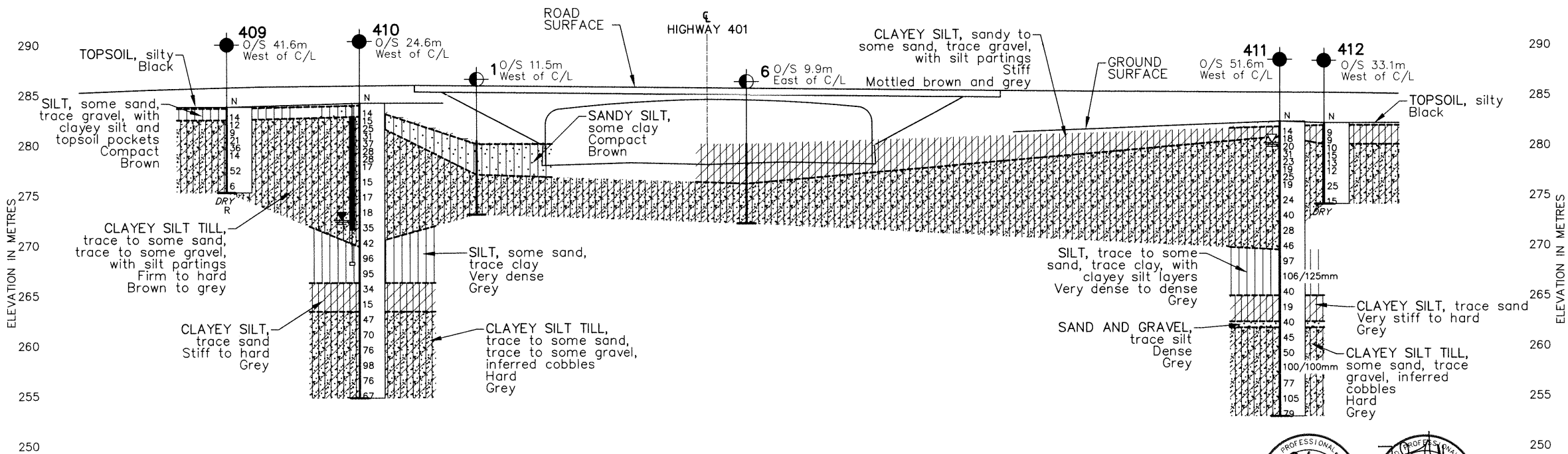
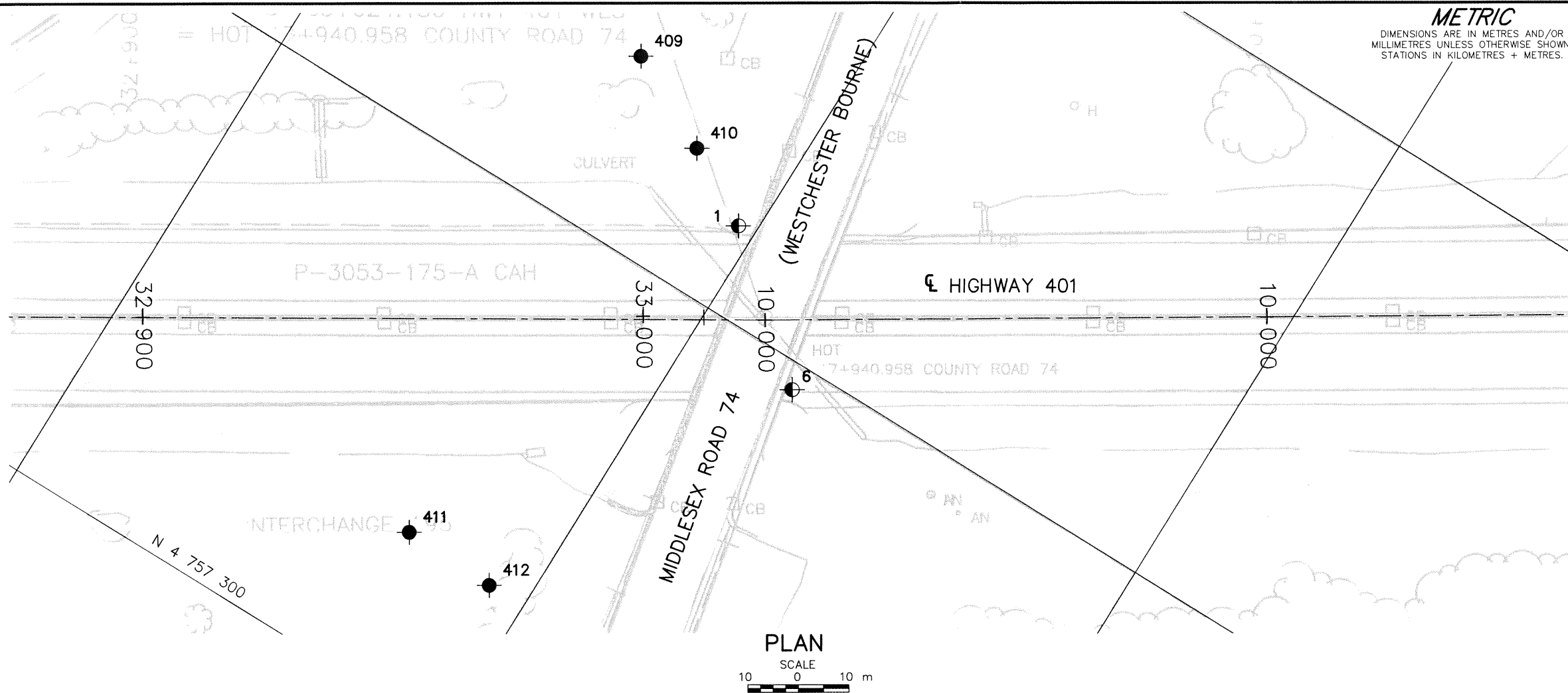
MIDDLESEX ROAD 74 UNDERPASS, SITE 19-375
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3053-11-00

TITLE

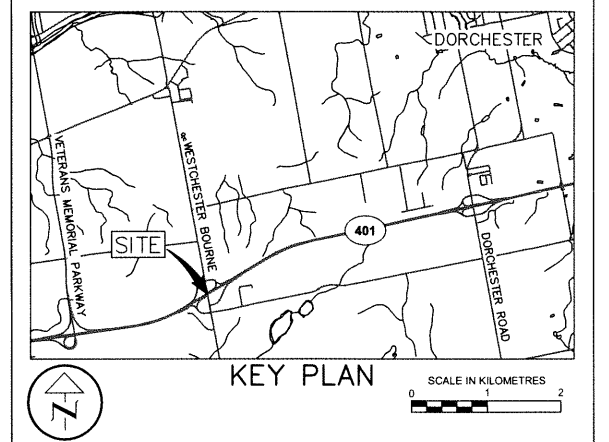
KEY PLAN



PROJECT No.		12-1132-0076	FILE No.		1211320076-2001-F01001
CADD	WDF/LMK	Apr. 15/15	SCALE	AS SHOWN	REV. 0
CHECK	NAG		FIGURE 1		

CONT No.
WP No. 3053-11-00MIDDLESEX ROAD 74
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

**Golder Associates Ltd.**
LONDON, ONTARIO, CANADA

LEGEND

- Borehole - Current Investigation
- Augerhole (Geocres 40114-110)
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on April 1, 2015
- WL encountered during drilling
- DRY Water level not established
- R Refusal

No.	ELEVATION	CO-ORDINATES (MTM ZONE 11)	
		NORTHING	EASTING
409	283.82	4 757 435.5	418 662.2
410	284.21	4 757 425.9	418 681.3
411	282.32	4 757 330.6	418 673.0
412	282.17	4 757 330.0	418 692.1
Geocres 40114-81			
1	280.42	4 757 417.1	418 696.5
6	280.11	4 757 394.9	418 722.7

NOTES

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans based on ETR Plate 92-401/17-0.

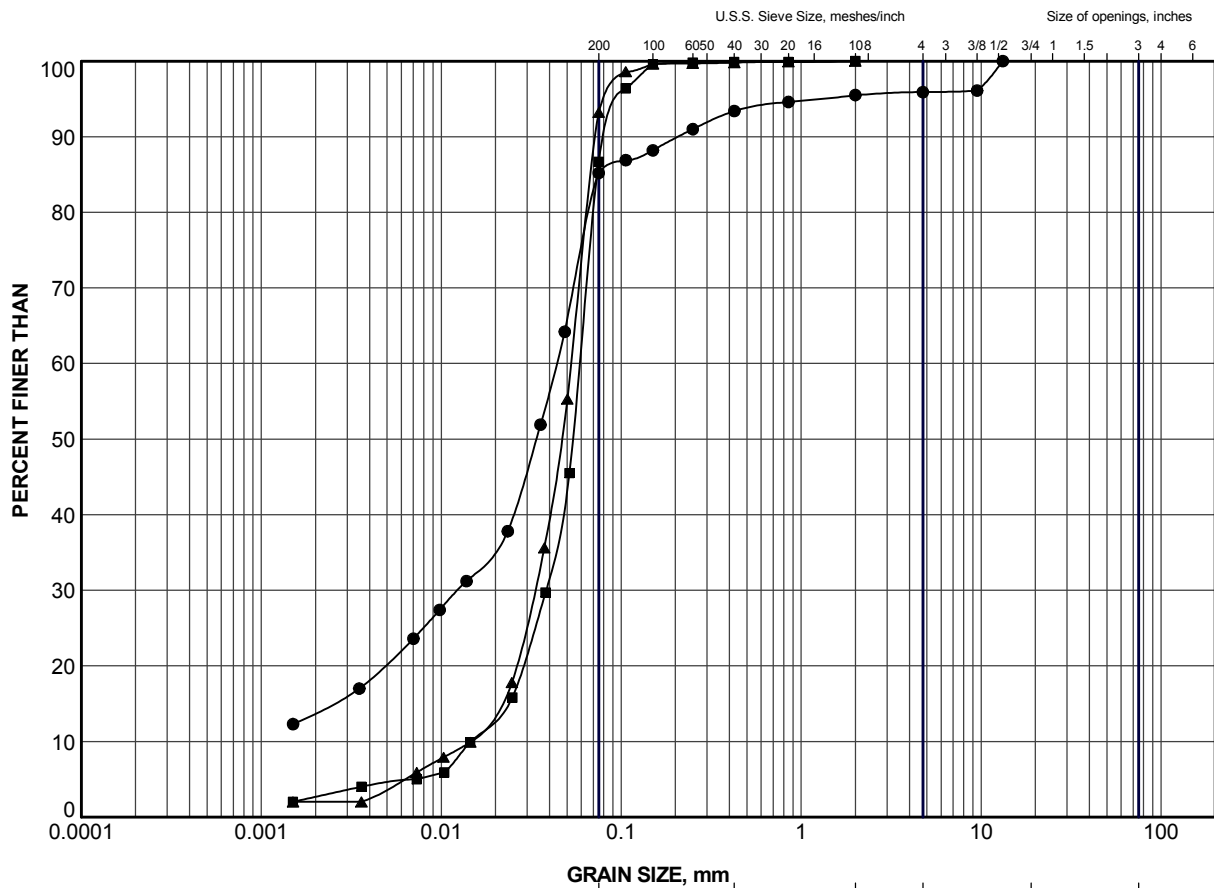
NO.	DATE	BY	REVISION
Geocres No. 40114-162			
HWY.	401	PROJECT NO.	12-1132-0076
SUBM'D.	NG	CHKD.	NAG
DRAWN:	LMK	CHKD.	DUP
DATE:	Mar 28/17	APPD.	FJH
SITE:	19-375	DWG.	1





APPENDIX A

Laboratory Test Data



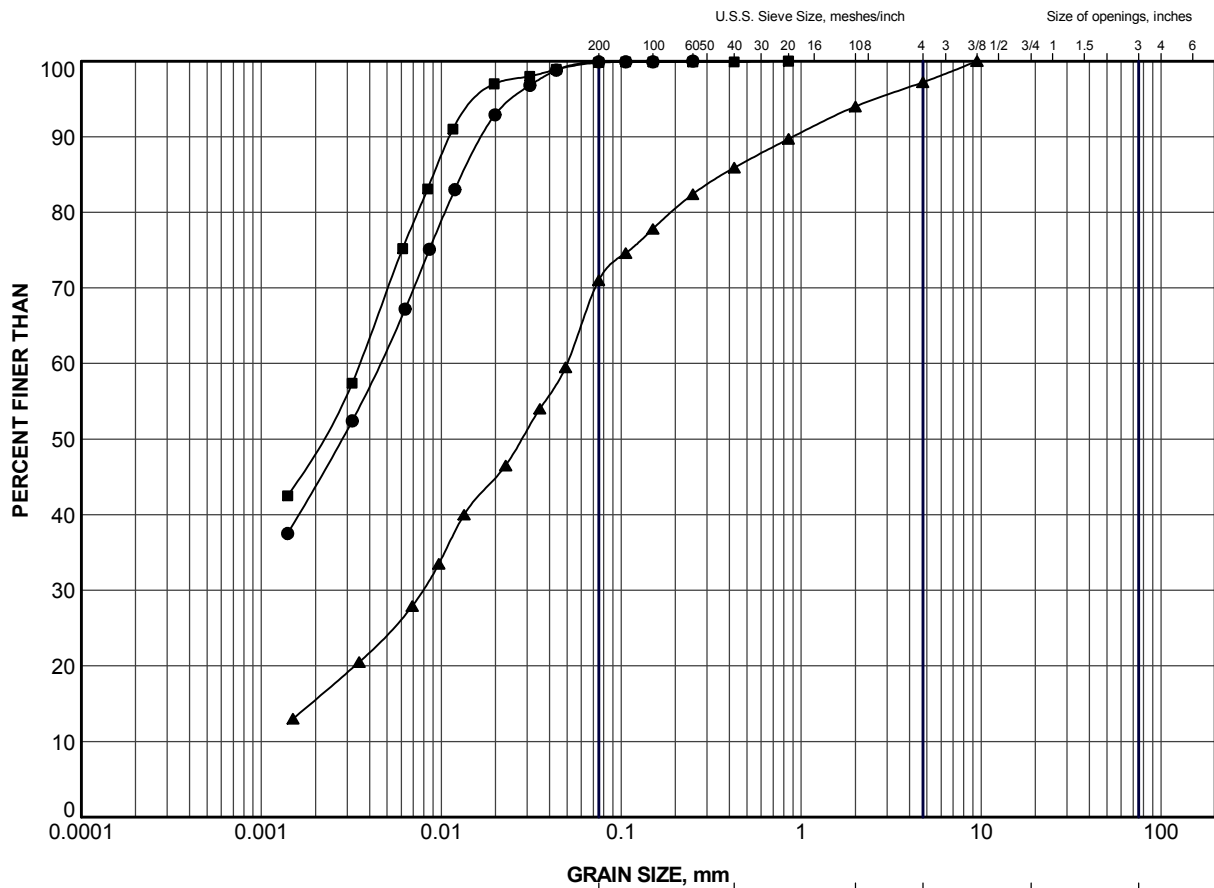
GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	409	1	282.8
■	410	14	268.7
▲	411	13	268.4

PROJECT					MIDDLESEX ROAD 74 UNDERPASS, SITE 19-375 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00				
TITLE					GRAIN SIZE DISTRIBUTION SILT				
PROJECT No.		12-1132-0076		FILE No. 1211320076-2001-F010A1		SCALE		N/A	
DRAWN		LMK		Apr 15/15		CHECK		NAG	
								FIGURE A-1	





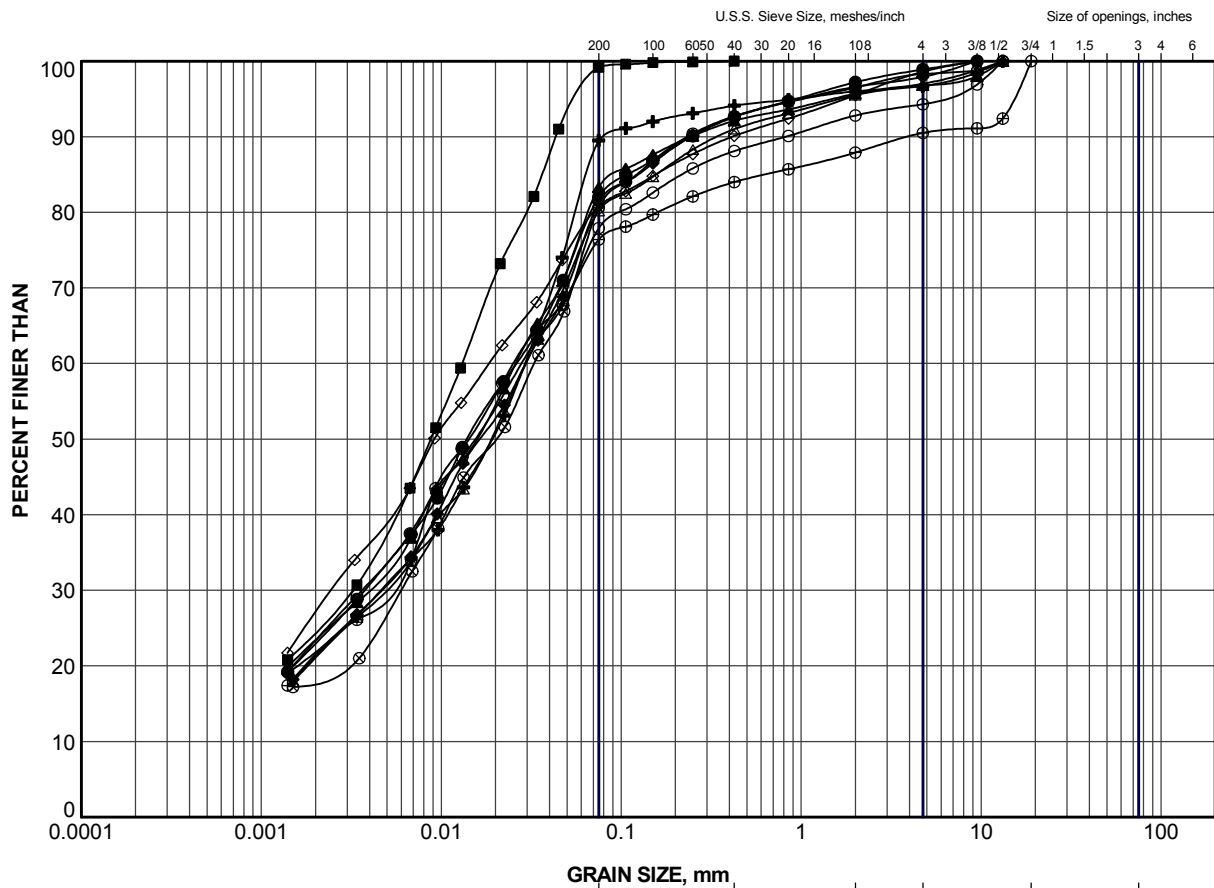
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	410	16	265.7
■	411	16	263.8
▲	412	2	280.4

PROJECT				MIDDLESEX ROAD 74 UNDERPASS, SITE 19-375 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00			
TITLE				GRAIN SIZE DISTRIBUTION CLAYEY SILT			
PROJECT No.		12-1132-0076		FILE No. 1211320076-2001-F010A2			
DRAWN		LMK		Apr 21/15		SCALE N/A REV.	
CHECK		NAG				FIGURE A-2	





LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	409	6	279.0
■	410	3	281.7
▲	410	10	274.8
+	410	13	270.3
◆	410	22	256.6
◇	411	2	280.6
○	411	5	278.3
△	411	19	259.2
⊗	411	21	256.2
⊕	412	8	274.3

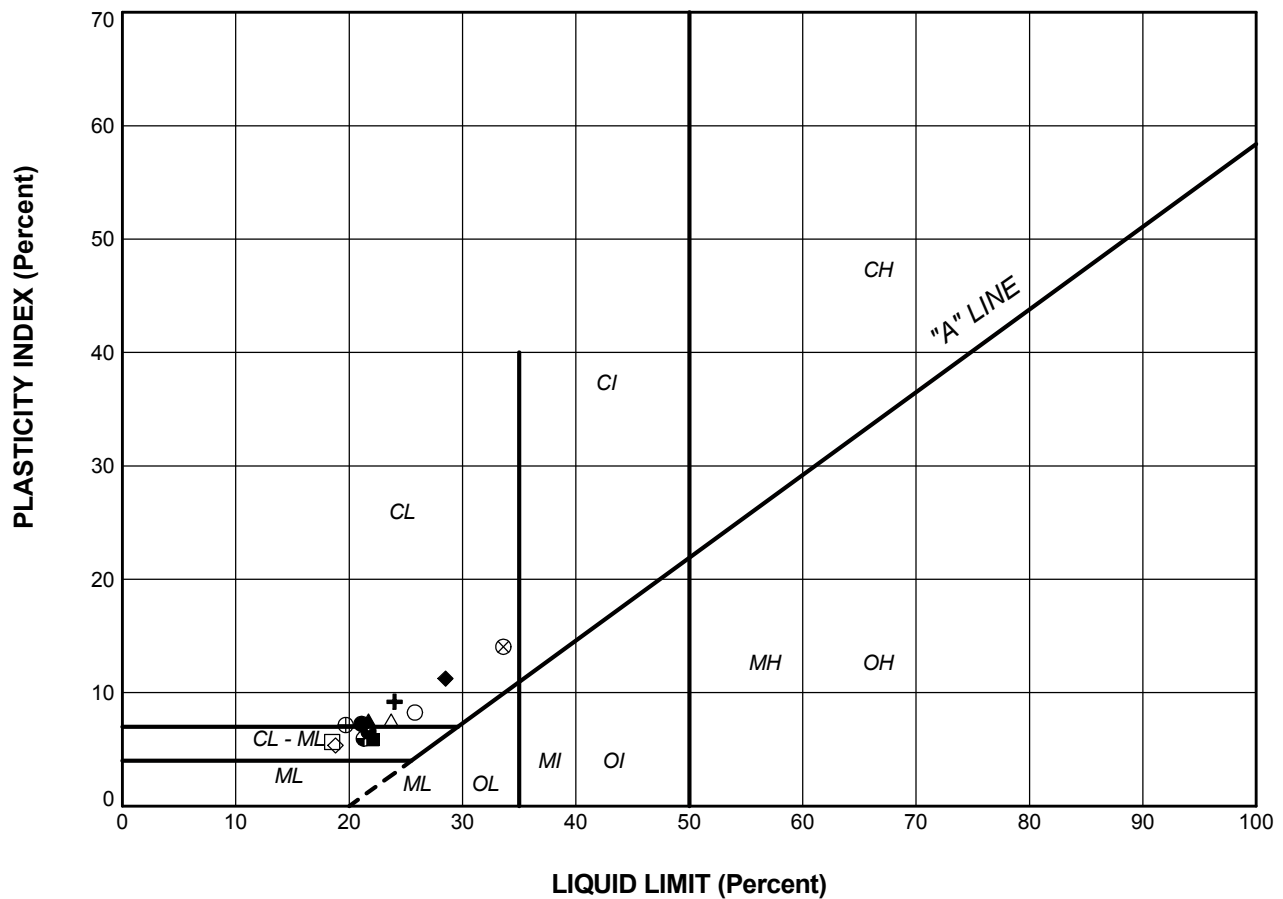
PROJECT MIDDLESEX ROAD 74 UNDERPASS, SITE 19-375
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3053-11-00

GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL



PROJECT No.	12-1132-0076	FILE No. 1211320076-2001-F010A3
DRAWN	LMK	Apr 22/15
CHECK	NAG	
SCALE	N/A	REV.

FIGURE A-3



LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	409	6	21.1	13.9	7.3
■	410	3	22.1	16.3	5.9
▲	410	10	21.7	14.2	7.6
+	410	13	24.0	14.8	9.2
◆	410	16	28.5	17.3	11.3
◇	410	22	18.8	13.5	5.4
○	411	2	25.8	17.6	8.3
△	411	5	23.7	16.2	7.6
⊗	411	16	33.6	19.6	14.1
⊕	411	19	19.7	12.6	7.2
□	411	21	18.5	12.9	5.7
⊙	412	2	21.7	15.1	6.6
⊛	412	8	21.3	15.4	6.0

PROJECT: MIDDLESEX ROAD 74 UNDERPASS, SITE 19-375
 HIGHWAY 401 INTERCHANGE IMPROVEMENTS
 GWP 3053-11-00

TITLE

PLASTICITY CHART



PROJECT No.	12-1132-0076	FILE No. 1211320076-2001-F010A4
DRAWN	LMK	Apr 15/15
CHECK	NAG	
SCALE	N/A	REV.

FIGURE A-4



APPENDIX B

Geocres Report No. 40I14-81

54 F-208C



"Franki Piles carry more tons per Pile"

CABLEGRAMS:
"FRANKIPILE"

FRANKI COMPRESSED PILE COMPANY OF CANADA LIMITED

1835 YONGE STREET



TELEPHONE:
HUDSON 8-9009

TORONTO 7, ONT.

Our Reference:

PC 304

September 15th, 1954.

SOIL INVESTIGATION REPORT for DEPARTMENT OF HIGHWAYS OF ONTARIO at HIGHWAY 401 AND HIGHWAY 74.

As requested, we carried out a soil investigation at the junction of Highway 401 and Highway 74.

REPORT OF INVESTIGATION:

Six percussion tests were made at the locations shown on the location sketch. The results are shown on the accompanying diagrams and are summarized as follows.

S U M M A R Y

Hole No:	Ground Surface	2,000 lbs per sq. ft.	4,000 lbs per sq. ft.	Refusal
1	920	927	910	898
2	920	915	912	897
3	920	915	912	901
4	919	915	910	898
5	919	914	910	896
6	920 919	913	907	895

Two auger borings were made with the following results.

LOG OF AUGER BORINGS

Hole #1

Ground Surface Elevation: 920

0' - 1'	Sandy clay top soil
1' - 6'	Sandy clay, moist
6' - 11'	Sandy clay, moist
11' - 16'	Light clay, slightly sandy and stoney
16' - 24'	Light clay, slightly sandy and stoney
24' /	Light clay, slightly sandy and stoney

- 2 -

Hole #6.

Ground Surface Elevation: 919

0' - 1'	Hard dry sandy clay
1' - 8'	Sandy clay, moist
8' - 13'	Sandy clay, moist
13' - 16'	Light clay, slightly sandy and stoney
16' - 21'	Light clay, slightly sandy and stoney
21' - 26'	Light clay, slightly sandy and stoney
26' 1/2	Light clay, slightly sandy and stoney

CONCLUSION:

Footings may be designed for 4,000 psf bearing
at the elevations shown.

W H T Wilson

W. H. T. Wilson, P. Eng.

FRANKI PILES

MONTREAL

PERCUSSION TEST DIAGRAM

FOR: DEPARTMENT OF HIGHWAYS OF ONTARIO

AT: HIGHWAY 401 & HIGHWAY #74

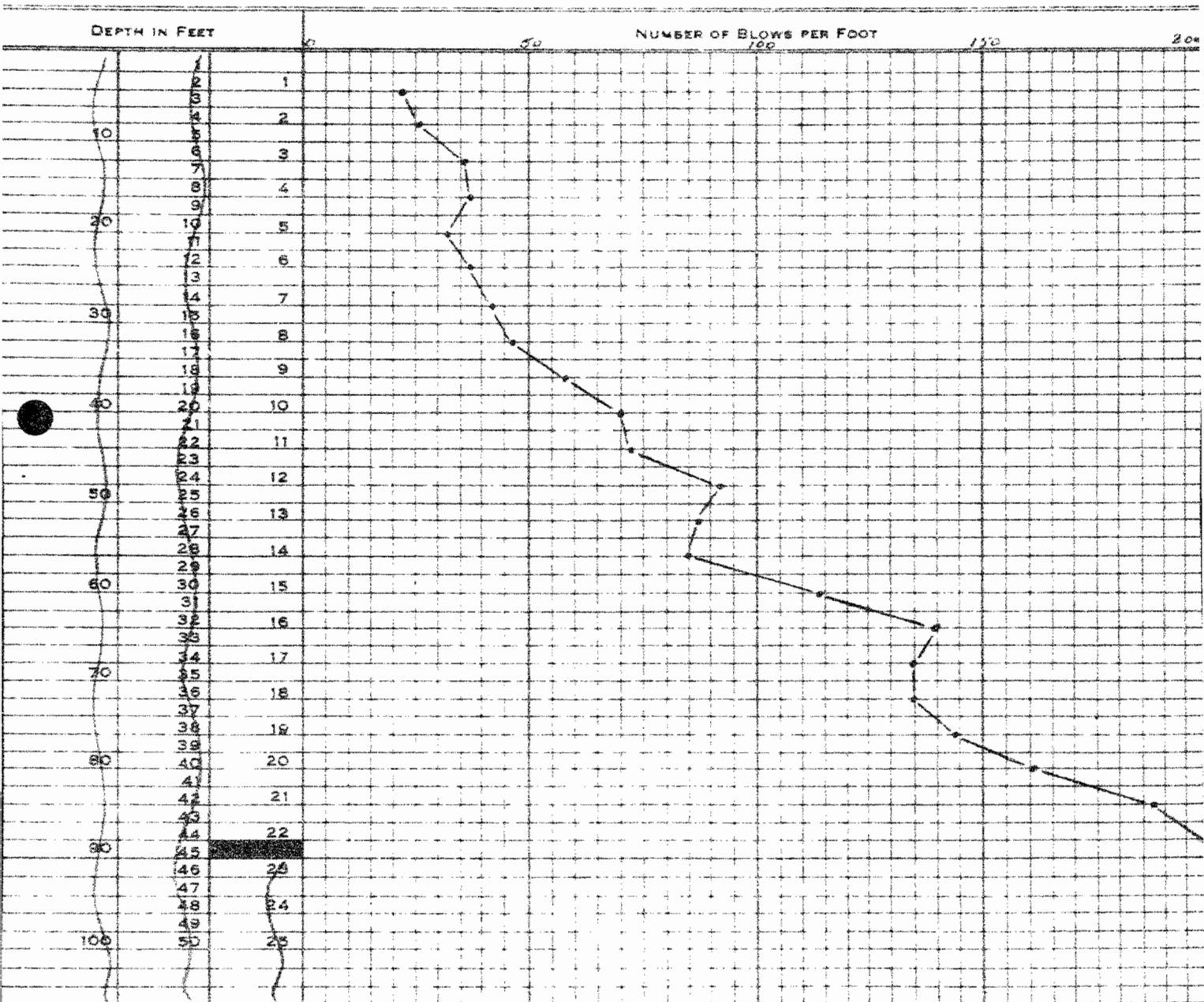
JOB NO. PC 304

DATE SEPTEMBER 7, 1954.

TEST NO.: 1

WEIGHT OF HAMMER 225#

DROP: 3 Ft



Ground Surface Elevation 920
Refusal Elevation 298
Number of Blows 200 for 11"

Signed

FRANKI PILES

MONTREAL

PERCUSSION TEST DIAGRAM

FOR: DEPARTMENT OF HIGHWAYS OF ONTARIO

AT: HIGHWAY 401 & HIGHWAY #74

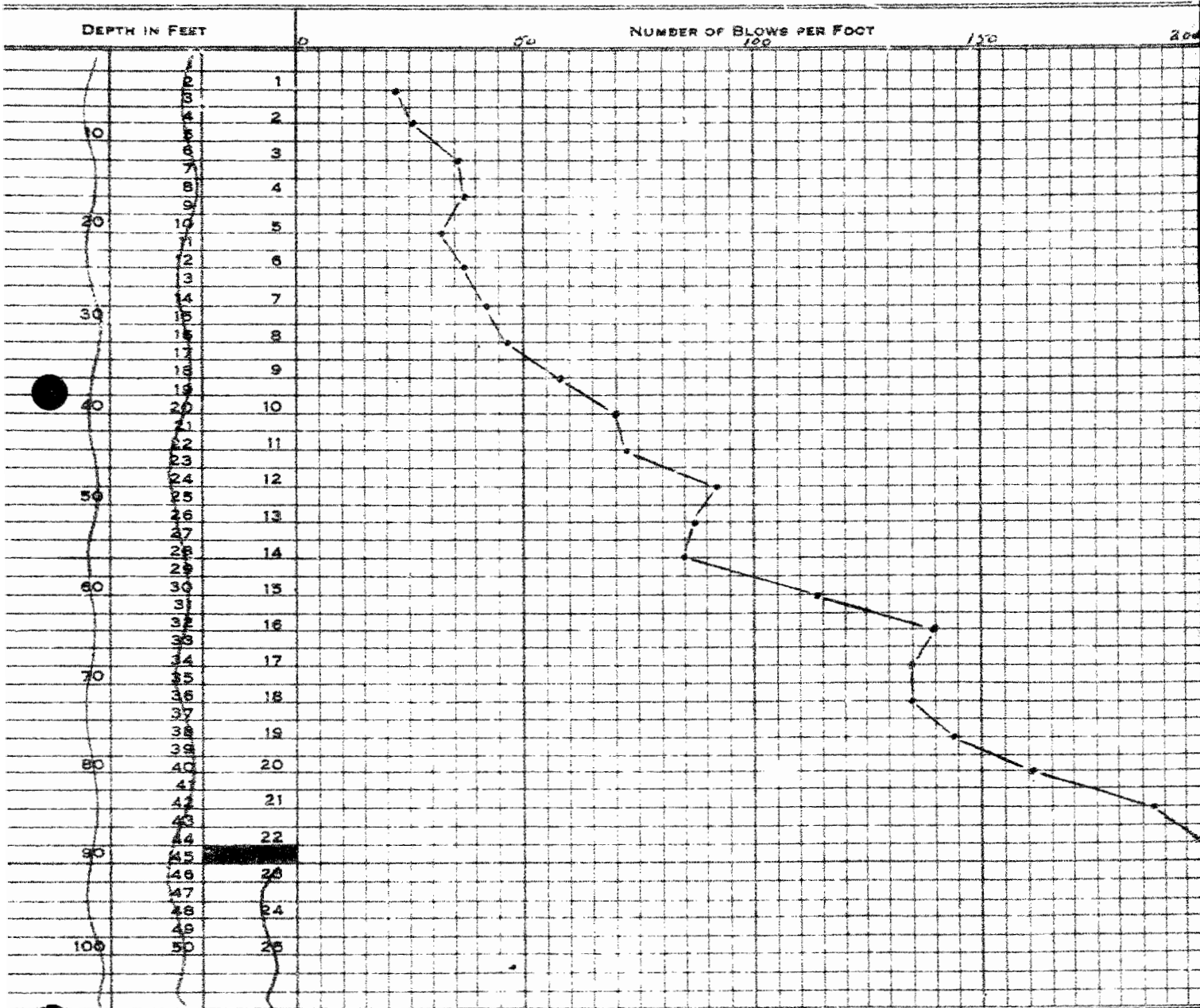
DATE SEPTEMBER 7, 1954.

TEST NO.: 1

WEIGHT OF HAMMER 225#

JOB NO. PG 304

DROP: 3 Ft.



Ground Surface Elevation 920
Refusal Elevation 898
Number of Blows 200 for 11"

Signed *W. H. White*

FRANKI PILES

MONTREAL

PERCUSSION TEST DIAGRAM

FOR: DEPARTMENT OF HIGHWAYS OF ONTARIO

AT: HIGHWAY 401 AND HIGHWAY #74

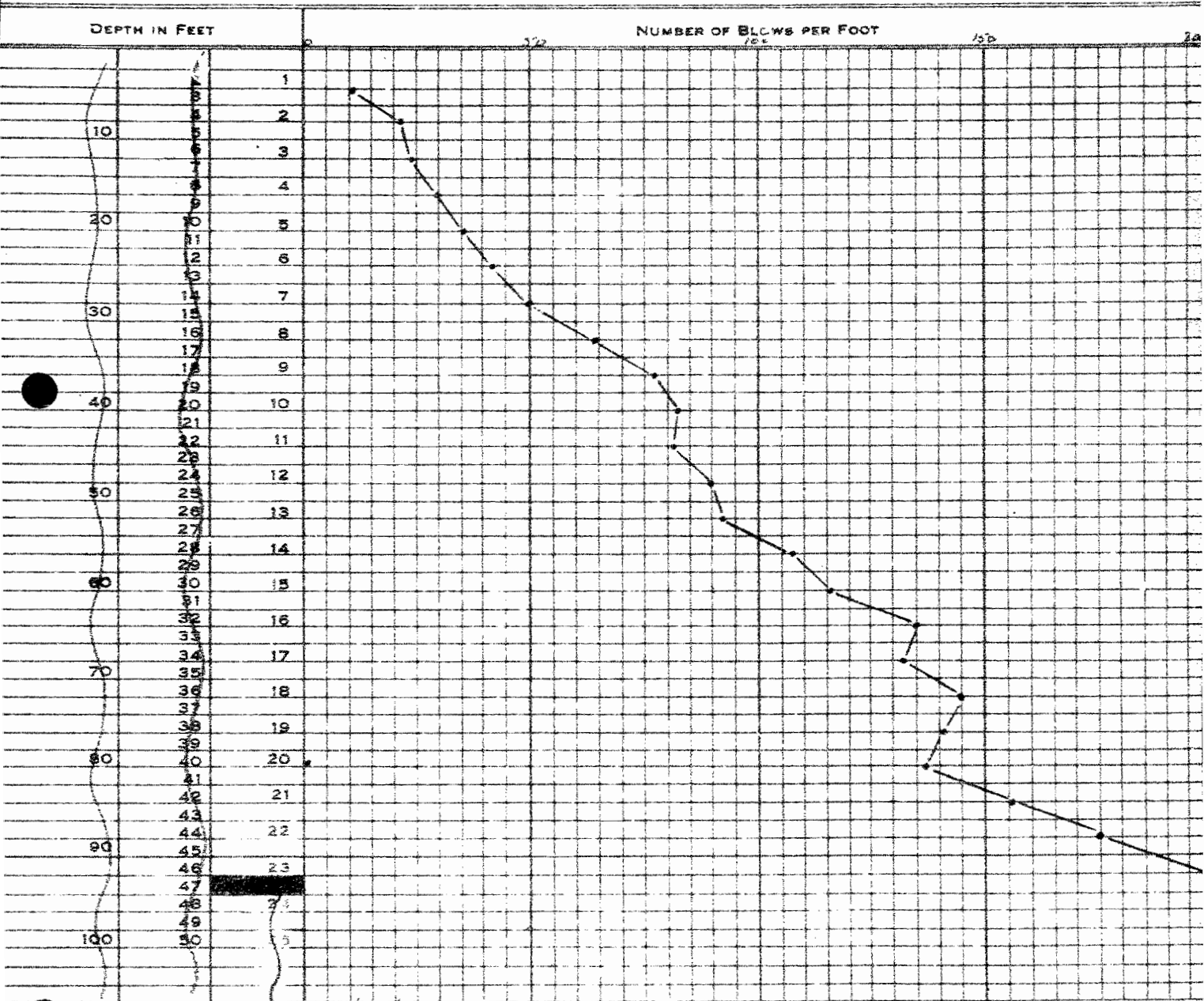
DATE: SEPTEMBER 8, 1954.

TEST NO.: 2

WEIGHT OF HAMMER 225#

JOB NO. FC 304

DROP: 3 Ft.



Ground Surface Elevation 920
Refusal Elevation 897
Number of Blows 202

Signed *W. F. L. L.*

FRANKI PILES

MONTREAL

PERCUSSION TEST DIAGRAM

FOR: DEPARTMENT OF HIGHWAYS OF ONTARIO

AT: HIGHWAY 401 AND HIGHWAY #74

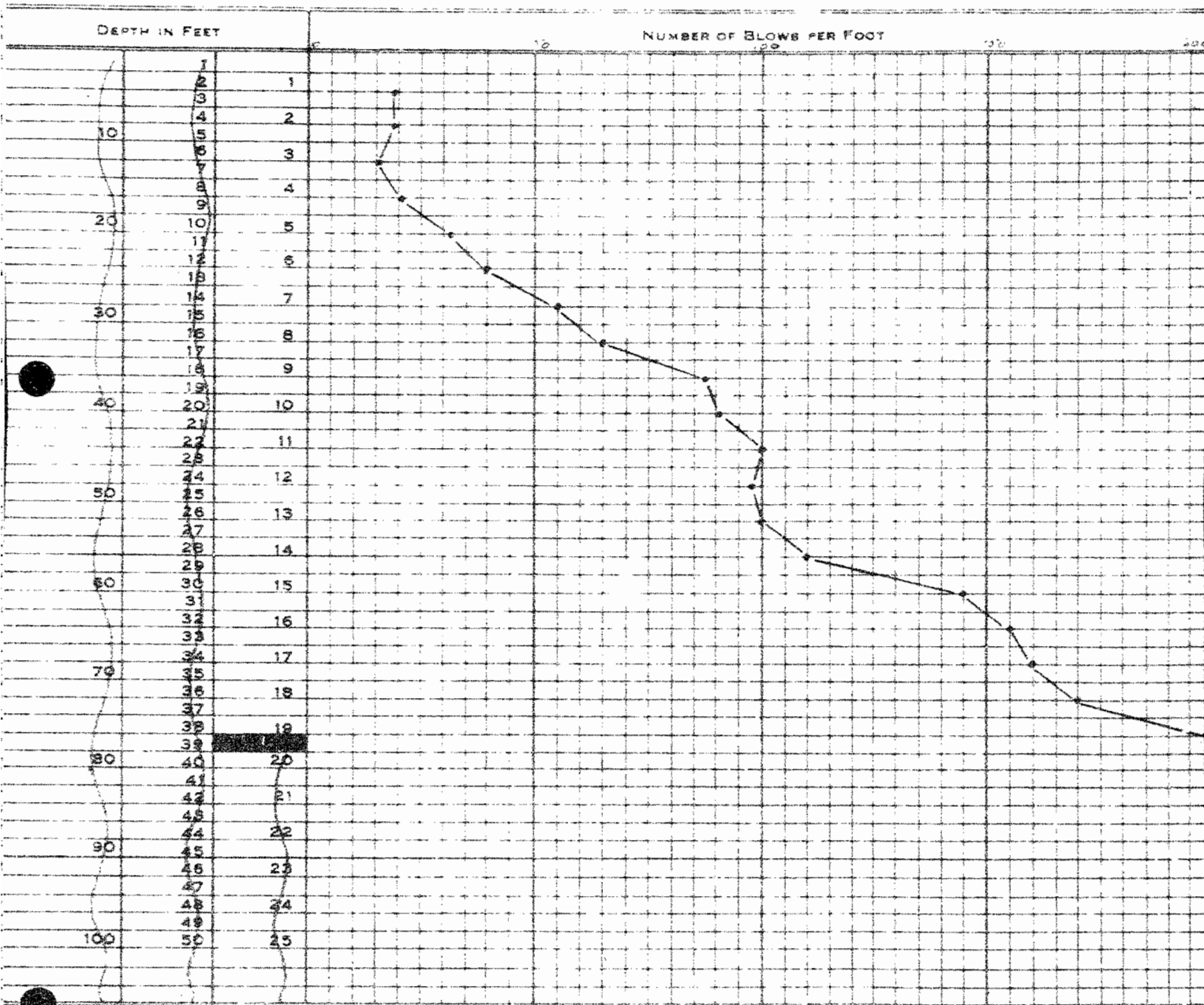
JOB NO. PC 304

DATE SEPTEMBER 8, 1954.

TEST NO.: 3

WEIGHT OF HAMMER 225#

DROP: 3 Ft.



Ground Surface Elevation 900
Refusal Elevation 901
Number of Blows 200 for 10"

Signed

FRANKI PILES

MONTREAL

PERCUSSION TEST DIAGRAM

FOR: DEPARTMENT OF HIGHWAYS OF ONTARIO

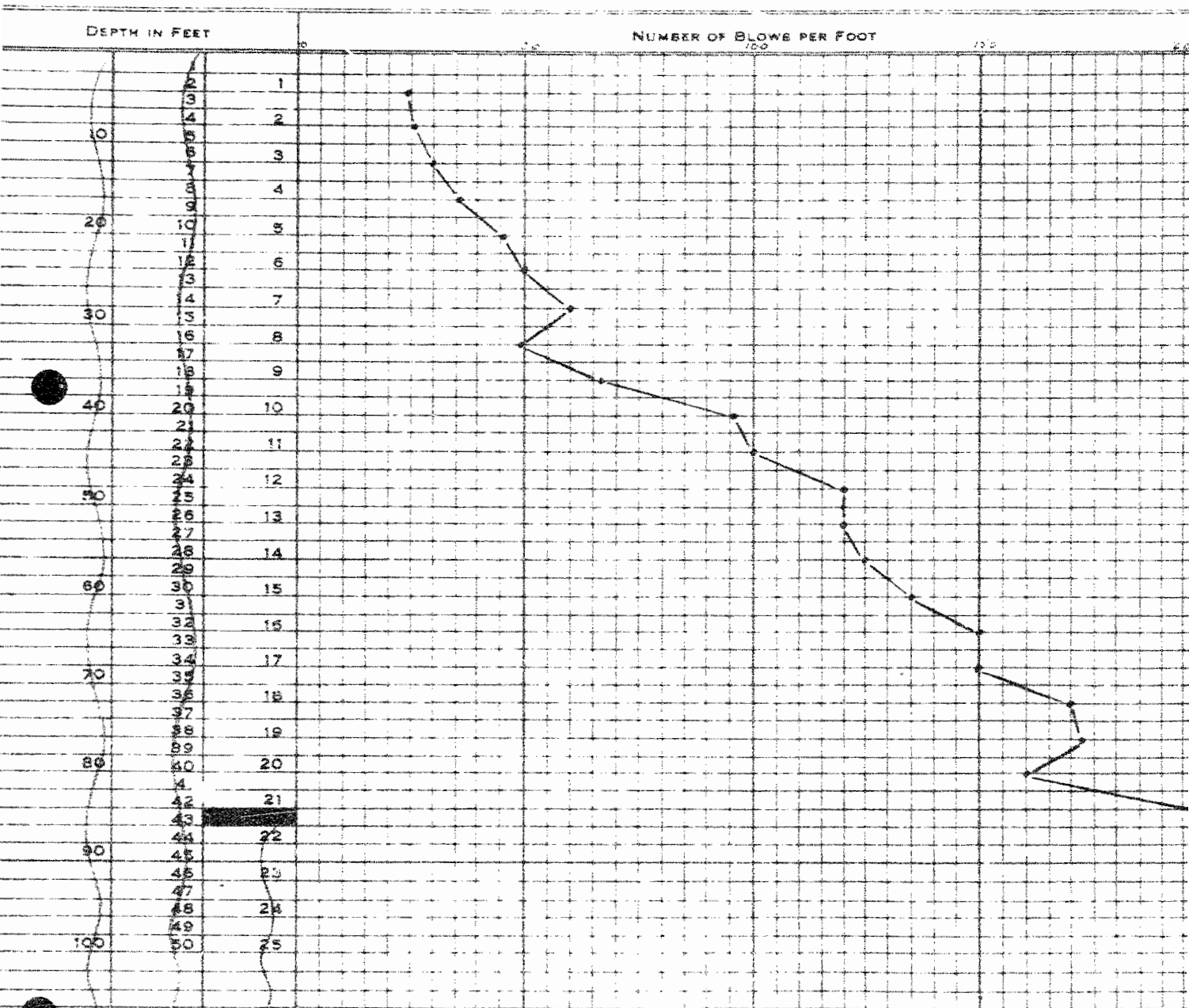
AT: HIGHWAY 401 AND HIGHWAY #74

JOB NO. PC 304

DATE SEPTEMBER 9, 1954.

TEST NO.: 4 WEIGHT OF HAMMER 225#

DROP: 3 Ft.



Ground Surface Elevation 919
Refusal Elevation 898
Number of Blows 200 for 10"

Signed

FRANKI PILES

MONTREAL

PERCUSSION TEST DIAGRAM

FOR: DEPARTMENT OF HIGHWAYS OF ONTARIO

AT: HIGHWAY 401 AND HIGHWAY #74

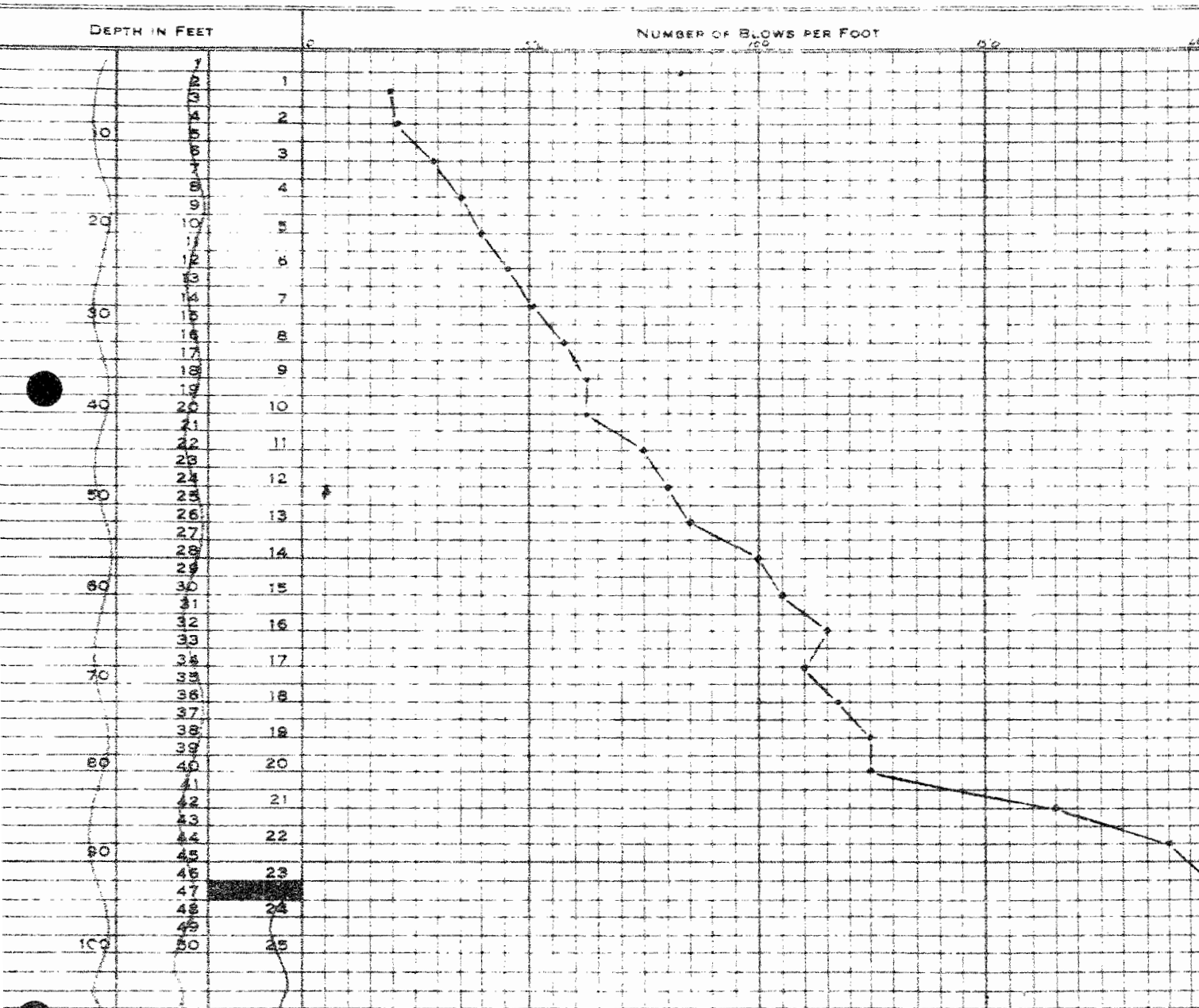
JOB NO. FC 304

DATE SEPTEMBER 9, 1954

TEST NO.: 5

WEIGHT OF HAMMER 225#

DROP: 3 Ft.



Ground Surface Elevation 919
Refusal Elevation 906
Number of Blows 200 for 11"

Signed *[Signature]*

FRANKI PILES

MONTREAL

PERCUSSION TEST DIAGRAM

FOR: DEPARTMENT OF HIGHWAYS OF ONTARIO

AT: HIGHWAY 401 AND HIGHWAY #74

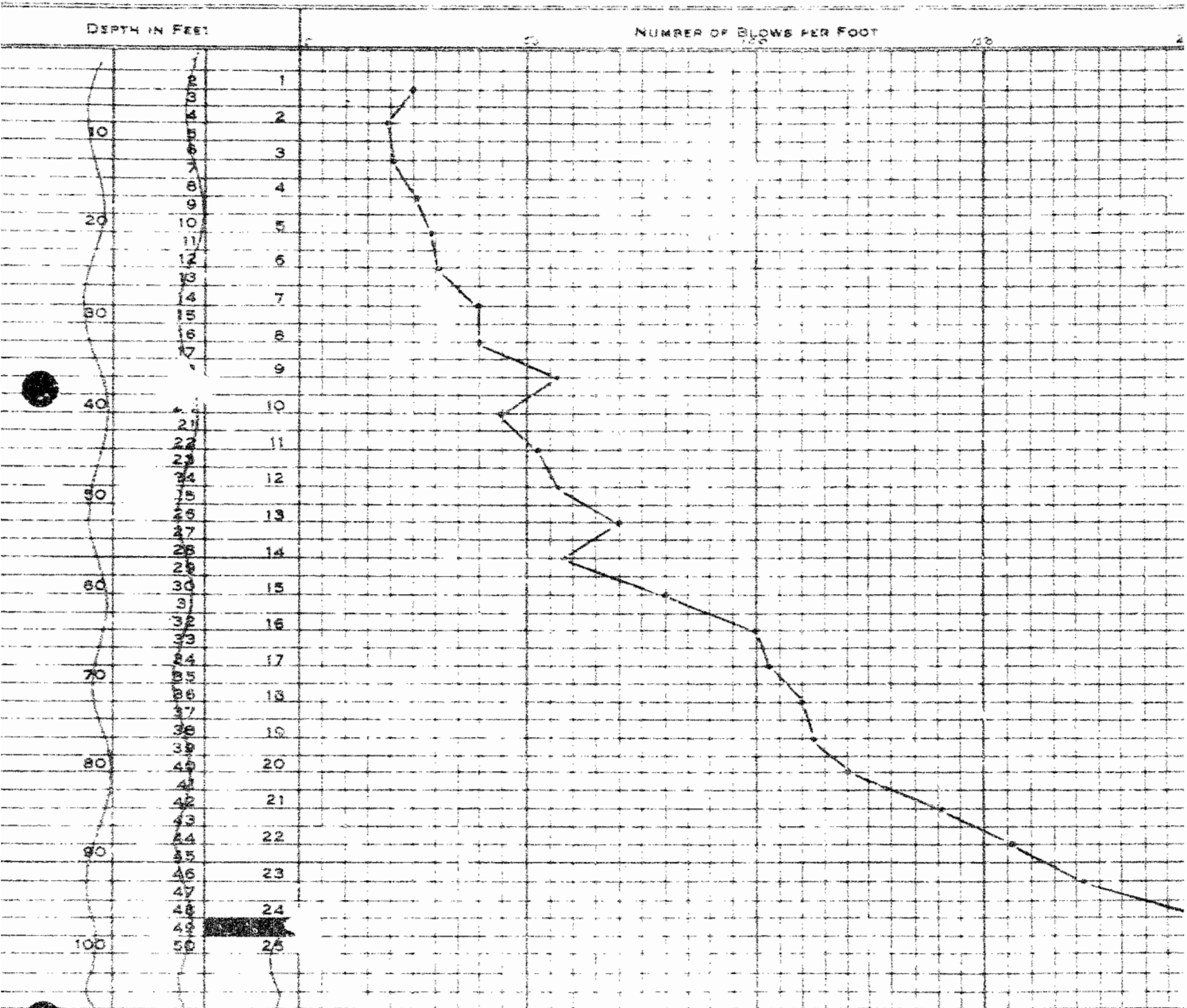
DATE SEPTEMBER 9, 1954.

TEST NO. 6

WEIGHT OF HAMMER 225#

JOB NO. PC 304

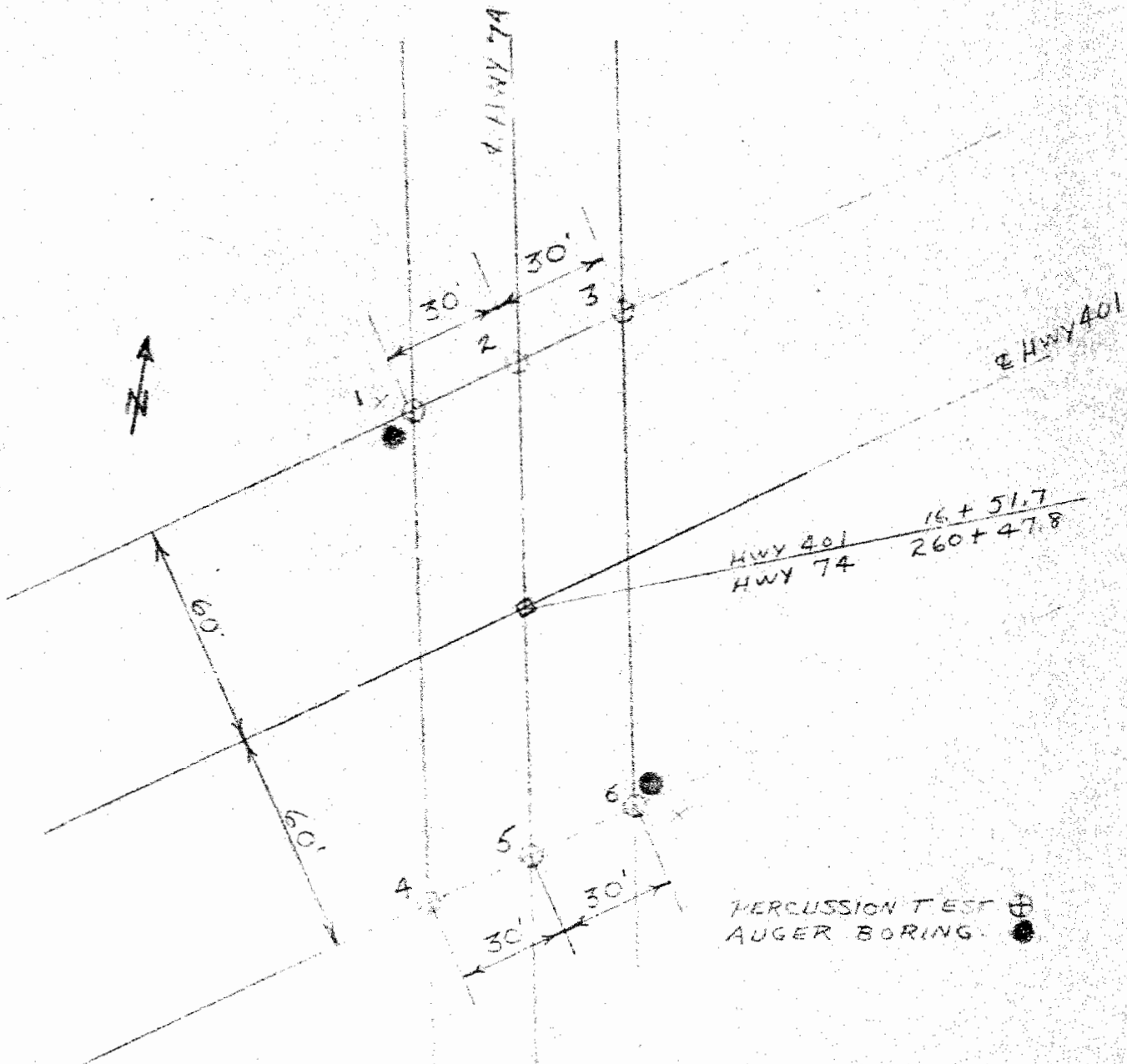
DROP: 3 Ft.



Ground Surface Elevation 919
Refusal Elevation 895
Number of Blows 212

Signed *W. J. J. J.*

LOCATION SKETCH



FRANKI COMPRESSED PILE COMPANY OF CANADA LIMITED
for
DEPARTMENT OF HIGHWAYS OF ONTARIO
at
HWY 401 AND HWY #74



APPENDIX C

DHO Drawings



D.H.O. REFERENCE DRAWINGS:
D.H.O. PROFILE NO. F.9529-B (D.M. 4544) 165UED; 29 JULY. 5.
D.H.O. PLAN NO. F.9529-10 (D.M. 4544) 165UED; 29 JULY. 5.

WESTMINSTER TWP. BRIDGE NO. 1		Job No. 910-17
M. M. DILLON & CO. LTD. CONSTRUCTION TORONTO		
DEPARTMENT OF HIGHWAYS: ONTARIO- BRIDGE OFFICE-TORONTO		
WESTMINSTER TWP. UNDERPASS HIGHWAY 74		
THE KING'S HIGHWAY NO. 401		DIV. NO. 2
CO. MIDDLESEX		
TWP. WESTMINSTER	LOT 1 & 24	CON. 1 & 2
GENERAL LAYOUT		
APPROVED		
<i>John L. ...</i> CHIEF BRIDGE ENGINEER		CHIEF ENGINEER
DESIGN	J. L. M. CHECK <i>Exp</i>	CONTRACT NUMBER 57-4352 GENERAL
DRAWING	D. H. H. CHECK <i>ED</i>	55-192 55-78
TRACING	D. H. H. CHECK <i>ED</i>	LOADING
DATE JANUARY 1962	4-20-54	MARKING

Site: 19-375

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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