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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

**Middlesex Road 32 (Dorchester Road) Underpass
Site Number 19-303**

**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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PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT

**MIDDLESEX ROAD 32 (DORCHESTER ROAD) UNDERPASS
SITE NUMBER 19-303
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 32 (Dorchester Road) Underpass (Site 19-303) 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 32 (Dorchester Road) Underpass is located south of the Village of Dorchester in the Municipality of Thames Centre, Ontario. The structure is located about 3.7 kilometres east and west of Westchester Bourne and Elgin Road, respectively. The location of the project is shown on the Key Plan, Figure 1.

For the purposes of this report, Highway 401 and Middlesex Road 32 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. Highway 401 was constructed in a partial cut at this location. The highway surface is at approximate elevation 279 metres at the underpass location. Middlesex Road 32 has a pavement surface near elevation 285 metres. The existing underpass was constructed in 1955 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. It is anticipated that the existing structure will be demolished and replaced with a new structure erected at the same location as the existing bridge.

2.2 Site Geology

This project lies within the physiographic region known as the Westminster Moraine. The physiographic mapping indicates that the Middlesex Road 32 Underpass site is situated on a till moraine.¹ Geological mapping indicates that the surficial material consists of the Erie Lobe of Port Stanley silty clay till and clayey silt till, in places covered by thin patches of lacustrine silt.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 229 metres, some 49 metres below the approximate Highway 401 ground surface 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 March 13, 2013 during which time four boreholes were drilled at the locations shown on the Borehole Location Plan, Drawing 1. The table below summarizes the borehole locations, ground surface elevations at the borehole locations, and borehole depths.

| Borehole | Location (m) | | Ground Surface Elevation (m) | Borehole Depth (m) |
|----------|--------------|---------|------------------------------|--------------------|
| | Northing | Easting | | |
| 401 | 4 758 600 | 422 221 | 285.51 | 30.63 |
| 402 | 4 758 590 | 422 223 | 285.34 | 9.60 |
| 403 | 4 758 679 | 422 217 | 286.14 | 9.60 |
| 404 | 4 758 667 | 422 220 | 286.17 | 28.65 |

The 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 D 1586. 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 are not factored to account for the use of an automatic hammer.

The boreholes were terminated between about 10 and 31 metres below the existing pavement surface. Groundwater conditions in the boreholes were observed throughout the drilling operations and piezometers were installed in boreholes 401 and 402 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).

The field work was monitored on a full-time basis by experienced Golder Associates staff members 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 the pavement structure followed by sand and gravel embankment fill, silt, clayey silt till, clayey silt, sand and sand and gravel.

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.

4.1.1 Pavement Structure

Each of the boreholes was advanced through the paved shoulder of Dorchester Road. Approximately 210 to 240 millimetres of asphaltic concrete was encountered from the ground surface. Approximately 60 to 400 millimetres of granular base material was encountered beneath the asphalt. A single measured N value, as determined by the standard penetration testing, carried out in the very dense granular base material, was 65 blows per 0.3 metres.

4.1.2 Fill

Layers of loose to compact sand and gravel fill were encountered in each of the boreholes under the pavement structure from elevation 284.9 to 285.9 metres. The fill thickness ranged from 0.5 metres to 4.6 metres near the abutments. The fill had measured N values ranging from 6 to 28 blows per 0.3 metres. The water content of a single sample of the fill materials was 7 per cent. The presence of cobbles in the sand and gravel fill material was noted during drilling based on augering resistance. The results of a grain size determination carried out on a sample of the fill material are shown on Figure A-1.

It should be noted that, based on the period that this overpass was built, there may be remnants of temporary works buried in the fill. Also, Department of Highway (DHO) Drawing No. D-3492-2 entitled "Abut. and Handrail Details – Fin. Road Elevs." dated Dec. 8, 1954 indicated that mass concrete with a compressive strength of 10



megapascals was placed below the west half of the north abutment. The mass concrete extends to about 0.6 to 1.8 metres below the underside of the abutment footing. DHO Drawing No. D-3492-2 has been included in Appendix B for reference.

4.1.3 Buried Topsoil

Approximately 180 millimetres of topsoil was encountered beneath the fill material in borehole 404. 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.

4.1.4 Silt

Layers of compact to very dense silt were encountered beneath the fill materials in boreholes 402 and 403. The silt layers were 2.9 and 0.8 metres thick and were encountered at about elevations 284.6 and 284.8 metres, respectively. Measured N values for the silt layers ranged from 10 to 57 blows per 0.3 metres. A sample of the silt had a measured water content of about 20 per cent. The results of a grain size determination carried out on a sample of the silt are shown on Figure A-2.

4.1.5 Clayey Silt

Layers of firm to hard clayey silt were encountered beneath the silt in borehole 403 at elevation 284.0 metres and in boreholes 401 and 404 beneath the clayey silt till, at elevations 264.8 and 267.4 metres, respectively. The measured N values for the clayey silt ranged from 8 to 38 blows per 0.3 metres. Samples of the clayey silt had measured water contents of between about 18 and 24 per cent. The clayey silt had plastic limits ranging between 14 and 21 per cent, liquid limits between 24 and 35 per cent, and plasticity indices of 10 to 15 per cent, based on three Atterberg limits determinations, the results of which are shown on Figure A-7. The results of grain size determinations carried out on samples of the clayey silt are shown on Figure A-3.

4.1.6 Clayey Silt Till

Stiff to hard clayey silt glacial till was encountered in borehole 401 beneath the fill materials, in borehole 402 beneath the silt, in borehole 403 beneath the clayey silt, and in borehole 404 beneath the buried topsoil. Where fully penetrated, the clayey silt till layers were 13.6 and 15.6 metres thick. The clayey silt till was encountered



between elevations of 280.3 and 283.2 metres. Boreholes 402 and 403 were terminated in the clayey silt till after exploring the layer for 5.9 and 6.7 metres, respectively.

The clayey silt till had measured N values ranging from 14 to 100 blows per 0.3 metres, being more typically in the range of 20 to 30 blows per 0.3 metres in the upper portion and 30 to 48 blows per 0.3 metres with depth. An N value in excess of 100 blows per 0.3 metres was recorded in borehole 401 near elevation 279.3 metres likely due to possible cobbles or boulders. The presence of cobbles should be anticipated throughout the clayey silt till deposit due to the depositional history of glacial materials. The water contents of the clayey silt till ranged from 11 to 18 per cent. The clayey silt till had plastic limits ranging from 14 to 18 per cent, liquid limits of 22 to 29 per cent, and plasticity indices of 7 to 11 per cent, based on ten Atterberg limits determination, the results of which are shown on Figure A-7. The results of grain size determinations carried out on samples of the clayey silt till are shown on Figure A-4.

4.1.7 Sand and Gravel

An approximately 2.9 metre thick layer of compact to very dense sand and gravel was encountered in borehole 401 beneath the clayey silt at elevation 260.7 metres. Measured N values for the sand and gravel were 19 and 53 blows per 0.3 metres. A sample of the sand and gravel had a measured water content of 14 per cent. The results of a grain size determination carried out on a sample of the sand and gravel are shown on Figure A-5. The presence of cobbles was noted in the sand and gravel layer based on drilling resistance.

4.1.8 Sand

Dense to very dense sand with cobbles was encountered in borehole 401 beneath the sand and gravel, and in borehole 404 beneath the clayey silt at elevations 257.8 and 264.5 metres, respectively. Boreholes 401 and 404 were terminated in the sand after exploring it for about 2.9 and 7.0 metres. The sand had measured N values ranging from 30 to greater than 100 blows per 0.3 metres. Samples of the sand had measured water contents of 16 and 19 per cent. The results of grain size determinations carried out on samples of the sand are shown on Figure A-6.

4.2 Groundwater Conditions

Groundwater conditions were observed in the boreholes during drilling. Also, a piezometer and a groundwater monitoring standpipe were installed in boreholes 401 and 402, respectively, as shown on the Record of Borehole sheets; however, no groundwater measurements were taken in the standpipe installed in borehole 402 as it was destroyed prior to any measurements. Encountered and measured groundwater levels are summarized in the following table.



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| Borehole | Ground Surface Elevation (m) | Encountered Groundwater Level | | Measured Groundwater Elevation (m) | | |
|----------|------------------------------|-------------------------------|---------------|------------------------------------|----------------|--------------|
| | | Depth (m) | Elevation (m) | April 3, 2013 | April 25, 2013 | June 5, 2013 |
| 401 | 285.5 | 21.7 | 263.8 | 267.5 | 267.6 | 267.5 |
| 402 | 285.3 | * | * | - | - | - |
| 403 | 286.1 | * | * | - | - | - |
| 404 | 286.2 | 21.6 | 264.5 | - | - | - |

* Groundwater level not established.

The above-noted encountered water levels are not considered to be representative of the long-term, stabilized groundwater conditions as the readings were taken only during the relatively short duration of drilling. Based on the measured and encountered groundwater levels and the soil colour change from brown to grey, the inferred perched groundwater level in the clayey silt till is at about elevation 279 metres. Groundwater in the underlying granular deposits is inferred at elevation 267.5 metres. 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 licensed well contractor. The field operations were supervised by Mr. Brett Thorner and Mr. Daniel Hyland 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. Azmi M. Hammoud, 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 DESIGN REPORT

MIDDLESEX ROAD 32 (DORCHESTER ROAD) UNDERPASS
SITE NUMBER 19-303

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 32 (Dorchester Road) Underpass (Site 19-303). The recommendations are 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.

Preliminary design information was not available at the time of this report; however, it is anticipated that the replacement structure may be lengthened by up to about 14 metres to accommodate the Highway 401 driving lanes, or greater if speed change lanes are to be present beneath the structure. It is understood that three alignment alternatives are being considered for the replacement structure. The alternatives to maintaining the existing alignment include: realignment approximately 25 metres to the east and realignment approximately 25 metres to the west. It is currently not known if the replacement structure will be single or multi-span, though it has been indicated that consideration is being given to designing a single span post-tensioned rigid frame structure with abutments supported by shallow spread/strip foundations.

6.2 Existing Structure

It is understood that the existing underpass structure, constructed in 1955, is a single span bridge over Highway 401, located east of London, Ontario. The underpass is a rigid frame structure with a single span of 32.9 metres and a curb-to-curb width of 13.41 metres. The deck is composed of eight reinforced concrete "Tee" girders at 2.15 metre spacing on centres.

Detail design information and as-built drawings regarding the foundations of the existing underpass are not available; however, based on Geocres Report No. 40114-9⁴ and the original contract drawings, it appears that the abutments and wing walls are founded on shallow footings. Based on DHO Drawing Nos. D-3498-1 and D-3498-2, dated December 8, 1954, the underside of the footing elevation for the abutments and retaining walls are summarized as follows:

⁴ Geocres Report No. 40114-9 "Soil Investigation for Department of Highways of Ontario (M.M. Dillon, Consulting Engineers) at Dorchester #10" dated September 22, 1954.



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| Bridge Element | Founding Elevation (m) | Founding Material |
|--------------------------|------------------------|-----------------------------|
| North Abutment | 276.49 | hard clayey silt till |
| Northwest Retaining Wall | 277.68 | very stiff clayey silt till |
| Northeast Retaining Wall | 276.49 | hard clayey silt till |
| South Abutment | 276.10 | very stiff clayey silt till |
| Southwest Retaining Wall | 277.70 | very stiff clayey silt till |
| Southeast Retaining Wall | 276.91 | very stiff clayey silt till |

It should be noted that the north abutment elevation drawing indicated that mass concrete with a compressive strength of 10 megapascals was to be placed in a sub-excavation to depths of 1.8 metres at the west end of the abutment footing to 0.6 metres at 7.3 metres east. The mass concrete was stepped up twice with 2.4 metres long segments. DHO Drawing Nos. D-3498-1 and 2 have been included in Appendix B for reference. Geocres Report No. 40I14-9 noted that a “soft” area existed at the west half of the north abutment.

It appears that the original designers gave consideration to founding the structure on Franki piles. Eight percussion tests carried out by the Franki Compressed Pile Company of Canada Limited for Geocres Report No. 40I14-9 experienced refusal between elevations 269.7 and 272.2 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 existing pavement structure, overlying fill and topsoil materials, overlying layers of silt, clayey silt till, clayey silt, sand, and sand and gravel. Native soils were encountered in the boreholes between about elevations 284.8 and 280.3 metres. The elevation of Highway 401 at the site is known to be approximately 279 metres. The perched groundwater level in the clayey silt till has been inferred to be at or near approximately elevation 279 metres. The inferred groundwater level in the granular deposits underlying the till is near elevation 267.5 metres.

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, or drilled caissons 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 driven H-piles or concrete filled steel tube piles. It may be cost-effective to support the piers on shallow foundations, but noting the presence of softer layers in the area of the existing north abutment, further investigation is needed in the area of any proposed piers.

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. For a multi-span structure, the width of the abutment and pier footings is expected to be in the range of about 3.5 to 5 metres. For this range in footing size a factored geotechnical resistance at Ultimate Limit States (ULS) of 400 kilopascals and a geotechnical reaction at Serviceability Limit States (SLS) of 300 kilopascals may be used for footings founded on the very stiff to hard clayey silt till.

It is anticipated that for a single span structure with retaining wall type abutments, such as the post-tensioned rigid frame structure currently under consideration, larger shallow spread footings will be required. For this case, it is anticipated that footings on the order of 6 to 8 metres wide with a length equivalent to the bridge width of 15 metres will be required. Footings within this width range founded on the very stiff to hard clayey silt till at about the existing footing elevations may be designed using a factored geotechnical resistance at ULS of 400 kilopascals and a geotechnical reaction at SLS of 200 kilopascals.

Depending on the selected final design, higher geotechnical resistances/reactions may be available if footings are placed at depths below that of the existing footings, or if different footings sizes are required, such as for a single span structure with retaining wall type abutments.

The SLS values correspond to an estimated total settlement of 25 millimetres. The above geotechnical resistances/reactions 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 resistances/reactions 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). The eccentricity of the resultant load shall be limited as discussed in Clause 6.7.3.4 of the CHBDC. 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 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. 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.



A single span, post-tensioned rigid frame structure is being considered for this site. Foundation reactions will include both vertical and horizontal reaction. The horizontal reactions to lateral forces acting on the soils behind the abutment walls and footings can be estimated using the following relation:

$$k_h = \text{coefficient of horizontal subgrade reaction (MPa/m)} = \frac{1}{3} k_s \frac{1}{D}$$

where:

D = embedment depth of the wall or footing (m)

k_s = modulus of subgrade reaction (MPa/m)

A preliminary range of the modulus of subgrade reaction for the native clayey silt till of 12 to 46 megapascals per metre (MPa/m) may be used for preliminary structural analysis. Higher values correspond to stiffer soils and/or narrower footings. The modulus values should be reviewed once specific load information and structural displacement estimates are available. The modulus of subgrade reaction is dependent upon the engineering properties of both the soil and the engineering characteristics (stiffness) and dimensions of the structure. Therefore, achieving an efficient abutment design that resists lateral forces may require an iterative process whereby the soil-structure interaction parameters are reviewed in conjunction with structural designs and displacement estimates and adjusted if appropriate.

Frost Protection

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

Construction Considerations

It may be possible to place footings constructed for the west area of the north abutment on the existing mass concrete; however, this may not be necessary due to the significantly lengthened span. The mass concrete should be inspected to confirm that the structural integrity is intact and the dimensions are compatible with the new 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.



6.3.2 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 Steel H-Piles

For design, the factored axial geotechnical resistances at ULS and geotechnical reaction at SLS for HP 310 x 110 piles driven 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 Location | Assumed Cut-off Elevation (m) | Founding Strata | Maximum Founding Elevation (m) | Factored Geotechnical Resistance at ULS (kN) | Geotechnical Reaction at SLS (kN) |
|----------------|-------------------------------|-----------------|--------------------------------|----------------------------------------------|-----------------------------------|
| South Abutment | 282.0 | Very dense sand | 256.0 | 1,600 | 1,300 |
| North Abutment | 282.0 | Very dense sand | 260.0 | 1,600 | 1,300 |

The above cut-off elevations have been assumed based on the existing elevations at the site, a soil cover of about 1.2 metres for frost protection, and a stick up of 3 metres for integral abutments. Lower cut-off elevations will likely be used if other abutment types are selected resulting in shorter piles. However, the consequent reduction in capacity is expected to be minimal.

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 – Driven Steel Tube Piles

For design, the factored axial geotechnical resistances at ULS and geotechnical reaction at SLS for concrete filled steel tube piles with a 324 millimetre O.D. and a 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 of 25 millimetres of settlement.

| Pile Location | Assumed Cut-off Elevation (m) | Founding Strata | Maximum Founding Elevation (m) | Factored Geotechnical Resistance at ULS (kN) | Geotechnical Reaction at SLS (kN) |
|----------------|-------------------------------|-----------------|--------------------------------|----------------------------------------------|-----------------------------------|
| South Abutment | 282.0 | Very dense sand | 257.8 | 1,500 | 1,200 |
| North Abutment | 282.0 | Very dense sand | 261.0 | 1,500 | 1,200 |

Geotechnical Axial Resistance – Drilled Caissons

The vertical load carrying capacity of the caissons may be calculated using the following equation:

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

Where Q_s is the nominal side resistance in kilonewtons (kN), 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 side resistance in kilopascals (kPa). 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. Cast-in-place concrete drilled piers or caissons founded in the hard clayey silt till may be designed using the nominal unit side resistance (f_{SN}) as summarized in the table below:

| Soil Type | f_{SN} (kPa) | Unit Weight (kN/m ³) |
|-----------------------------------|----------------|----------------------------------|
| Fill | - | 19.0 |
| Clayey Silt Till (South Abutment) | | |
| - Above elevation 270.5 m | 90 | 21.0 |
| - Below elevation 270.5 m | 105 | 21.5 |



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT MIDDLESEX ROAD 32 UNDERPASS, SITE NUMBER 19-303

| Soil Type | f_{SN} (kPa) | Unit Weight (kN/m ³) |
|-----------------------------------|-------------------|-------------------------------------|
| Clayey Silt Till (North Abutment) | | |
| - Above elevation 276.5 m | 90 | 21.0 |
| - Below elevation 276.5 m | 115 | 21.5 |

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

$$Q_b = q_{BN} A_t$$

Where Q_b is the toe resistance in kN, q_{BN} is the nominal unit base resistance in kPa, and A_t is the cross-sectional area of the caisson in square metres. The ultimate resistance Q_u is the sum of Q_s and Q_b . A resistance factor of 0.5 should be applied to Q_u to obtain the factored axial resistance at ULS. The ULS values will govern design since the axial resistance at SLS for 25 millimetres of settlement is greater than at ULS.

The top of caisson elevations for the proposed abutments and piers are not known at this time. However, it has been assumed that the top of caisson elevation at the abutments will correspond to the top of clayey silt till elevation. For the purposes of preliminary design, the following factored geotechnical resistances may be used for caissons with a nominal diameter of 1.2 metres founded at the elevations recommended in the table below:

| Caisson Location | Assumed Cut-off Elevation (m) | Design Tip Elevation (m) | Founding Strata | q_{BN} (kPa) | Factored Geotechnical Reaction at ULS (kN) |
|------------------|-------------------------------|--------------------------|-----------------------|-------------------|--------------------------------------------|
| South Abutment | 281.0 | 270.5 | Hard Clayey Silt Till | 1,290 | 2,500 |
| North Abutment | 282.2 | 274.5 | Hard Clayey Silt Till | 1,500 | 2,500 |

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)

It is anticipated that a relatively low grade raise of up to about 1 metre will be constructed at the approach embankments in conjunction with the bridge replacement. Considering the relatively low grade raise, negligible negative skin friction is expected to develop for the new piles at both abutments.

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 where:

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

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)

The range in values 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.

| Location | Soil Type | Elevation (m) | n_h (MPa/m) | S_u (kPa) |
|-----------------------------|-----------------------------------------------------------|------------------|---------------|-------------|
| CSPs for integral abutments | Granular backfill | Where applicable | 5 - 10 | - |
| South Abutment | Existing Granular Fill – loose to compact sand and gravel | 280.3 to 284.9 | 2 - 6 | - |
| | Very stiff to hard clayey silt till | 264.8 to 280.3 | - | 200 |
| | Firm to very stiff clayey silt | 260.7 to 264.8 | - | 100 |
| South Abutment | Very dense sand and gravel | 257.8 to 260.7 | 6 - 16 | - |
| | Very dense sand | Below 257.8 | 20 - 22 | - |
| North Abutment | Existing Granular Fill – loose to compact sand and gravel | 281.0 to 285.8 | 2 - 6 | - |
| | Very stiff to hard clayey silt till | 267.4 to 281.0 | - | 250 |
| | Very stiff to hard clayey silt | 264.5 to 267.4 | - | 225 |
| | Dense to very dense sand | Below 264.5 | 15 - 22 | - |



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT MIDDLESEX ROAD 32 UNDERPASS, SITE NUMBER 19-303

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 | 30 | * |
| - 324 mm OD x 9.5 mm tube | 35 | * |
| Semi-Integral or Conventional abutments | | |
| - HP 310 x 110, strong axis bending | 230 | 125 |
| - 324 mm OD x 9.5 mm tube | 170 | 110 |
| Concrete Caissons, 1.2 metre diameter | 900 | 400 |

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

The lateral resistances are based on Brom's 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 underside of abutment. 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 |

Construction Considerations

It should be noted that cobbles and boulders are present in the till soils and the underlying granular layers and may impact pile driving/caisson drilling 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. It is anticipated that the abutments for the new structure will be



offset from the existing abutment locations in order to accommodate the lengthening of the bridge span. However, it is possible that new piers may be placed close to the existing abutments. It may be necessary to remove the mass concrete placed beneath the west half of the north abutment footings as noted on the original 1954 design drawings. If piles are to be driven through the existing embankment fill near the present abutments, they may encounter remnants of temporary works buried in the fill.

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

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, 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 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 criteria. Analysis of bridges in SPZ 1 is not a requirement of the CHBDC. 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.⁵ 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.

⁵ 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:

- Select, free-draining granular fill meeting the specifications of 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 SP 105S10. 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 OPSD 3101.150 and 3190.100.
- A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the abutment walls in accordance with CHBDC Figure 6.6. Compaction equipment should be used in accordance with Special Provision (SP) 105S10.
- 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 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 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_o 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 Embankments

It is anticipated that only minimal grade raising of the embankments will be required to accommodate the repositioned abutments. Also, approach embankment widening is not expected. However, if the design of the replacement bridge does require grade raising or embankment widening the following should be considered. All surficial topsoil, organic, loose, soft, and/or otherwise deleterious materials should be stripped from areas of proposed embankment widenings. Prior to placement of embankment fill material, the exposed subgrade should be proofrolled under the direction of a geotechnical QVE. The embankment fills should consist of an approved granular borrow such as SSM, except for the top approximately 0.5 metres where pavement base and subbase materials will be placed. Embankment fill materials should be placed in maximum 300 millimetre thick loose lifts and properly benched into the existing embankments and compacted. Upon completion of filling to the pavement subgrade level, the embankment side slopes should be trimmed to a final inclination of two horizontal to one vertical or flatter. Due to the anticipated minimal grade raising of the embankment and the presence of compact silt over stiff to hard clayey silt till, instability is not expected. Embankments greater than 8 metres in height should be constructed with a 2 metre wide bench at mid-height.

6.7 Excavations and Temporary Cut Slopes

6.7.1 General

Excavations for the pile caps or shallow foundations will penetrate the existing fill and organic materials into the clayey silt till. The groundwater level in the till is expected to be at about elevation 279 metres and will fluctuate seasonally and due to climatic variations. The excavations are expected to extend below the groundwater level; however, minor seepage is anticipated. Seepage volumes are expected to be low due to the presence of clayey soil. If necessary, groundwater control may be 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. The fill materials at this



site would be classified as Type 3 soils as would any cohesionless soils below the groundwater level. The native clayey materials and properly dewatered cohesionless soils would be classified as Type 2.

6.7.2 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.

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

Driven steel H or concrete filled steel tube piles are considered the preferred foundation alternative for the replacement structure. 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 401 and 404, as well as at the location of any proposed central piers. The boreholes should be advanced to 3 metres below refusal, 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.

It is understood that, in addition to replacing the structure along its current alignment, consideration is being given to relocating the structure by approximately 25 metres to the east or west. If one of the realignment alternatives is selected for Detail Design, additional boreholes may be required at the approaches and/or abutments, in addition to those noted above.

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 central pier(s). If the vertical alignments of the highway or the approach embankments are to be significantly altered, embankment stability and settlement should be evaluated. Also, if staged construction is to be used for the construction of the replacement bridge, 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. Azmi M. Hammoud, 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 32, Site 19-303
 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"> • Feasible 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 into very dense sand. | <ul style="list-style-type: none"> • Preferred technical alternative 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:
- 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.
 - Table to be read in conjunction with accompanying report.

Prepared By: NG
 Checked By: DUP

LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

(b) Cohesive Soils

Consistency

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

IV. SOIL TESTS

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

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

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

- Notes:**
- 1 $\tau = c' + \sigma' \tan \phi'$
 - 2 shear strength = (compressive strength)/2
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

RECORD OF BOREHOLE No 401

1 OF 3

METRIC

PROJECT 12-1132-0076
W.P. 3053-11-00 LOCATION N 4758600.3 , E 422220.7 ORIGINATED BY BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM / WASH BORING, CASED COMPILED BY WF/LK/AG
DATUM GEODETIC DATE March 5, 2013 - March 6, 2013 CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) |
|----------------|-------------------------------------------------------------------------------------------------------------------------------------------|------------|---------|------|---------------|----------------------------|-----------------|---------------------------------------------|-------------------------|---------------------------------------------------------|-------------------|----|-------------------------|---------------------------------------------------|
| ELEV. DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | 20 40 60 80 100 | 20 40 60 80 100 | W _P W W _L | WATER CONTENT (%) | GR | | |
| 285.51 | PAVEMENT SURFACE | | | | | | | | | | | | | |
| 0.00 | ASPHALT | | | | | | | | | | | | | |
| 0.21 | FILL, granular base | | 1 | SS | 65 | | 285 | | | | | | | |
| 284.90 | | | | | | | | | | | | | | |
| 0.61 | FILL, sand and gravel, trace to some silt, with cobbles Loose to compact Brown | | 2 | SS | 25 | | | | | | | | | |
| | | | 3 | SS | 6 | | | | | | | | | |
| | | | 4 | SS | 12 | | | | | | | | | |
| | | | 5 | SS | 9 | | | | | | | | | |
| | | | 6 | SS | 8 | | | | | | | | | |
| | | | 7 | SS | 16 | | | | | | | | | |
| 280.33 | | | | | | | | | | | | | | |
| 5.18 | CLAYEY SILT TILL, trace to some sand, trace gravel, with cobbles Very stiff to hard Brown becoming grey at about elev. 279.0m | | 8 | SS | 21 | | 280 | | | | | | | |
| | Cobble/boulder at about elev. 279.3m | | 9 | SS | 100/ 140mm | | | | | | | | | |
| | | | 10 | SS | 29 | | | | | | | | | |
| | | | 11 | SS | 22 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 12 | SS | 22 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 13 | SS | 24 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 14 | SS | 25 | | | | | | | | | |
| | | | | | | | | | | | | | | |
| | | | 15 | SS | 29 | | | | | | | | | |
| | | | | | | | | | | | | | | |

Continued Next Page

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

| | | | | | |
|---------------------------------------|--------------------------------------------------------------------|----------------------------------|--|----------------------------------------|---------------|
| PROJECT <u>12-1132-0076</u> | | RECORD OF BOREHOLE No 401 | | 2 OF 3 | METRIC |
| W.P. <u>3053-11-00</u> | LOCATION <u>N 4758600.3 , E 422220.7</u> | | | ORIGINATED BY <u>BT</u> | |
| DIST <u> </u> HWY <u>401</u> | BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM / WASH BORING, CASED</u> | | | COMPILED BY <u>WF/LK/AG</u> | |
| DATUM <u>GEODETIC</u> | DATE <u>March 5, 2013 - March 6, 2013</u> | | | CHECKED BY <u> </u> | |

[illegible]

Continued Next Page

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

LDN_MTO_06 12-1132-0076-2001.GPJ LDN_MTO.GDT 20/11/13

| | | | | | |
|-----------------------------|--------------------------------------------------------------------|----------------------------------|--|--------|---------------|
| PROJECT <u>12-1132-0076</u> | | RECORD OF BOREHOLE No 401 | | 3 OF 3 | METRIC |
| W.P. <u>3053-11-00</u> | LOCATION <u>N 4758600.3 , E 422220.7</u> | ORIGINATED BY <u>BT</u> | | | |
| DIST <u></u> HWY <u>401</u> | BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM / WASH BORING, CASED</u> | COMPILED BY <u>WF/LK/AG</u> | | | |
| DATUM <u>GEODETIC</u> | DATE <u>March 5, 2013 - March 6, 2013</u> | CHECKED BY <u></u> | | | |

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|-----------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------|---------|------|------------|----------------------------|-----------------|---------------------------------------------|----|----|----|-----|---------------------------------------------------------|---|----------------|--------------------------------------|---------------------------------------------------|----|----|----|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | | GR | SA | SI | CL |
| | | | | | | | | 20 | 40 | 60 | 80 | 100 | W _p | W | W _L | | | | | |
| | | | 26 | SS | 83 | | | | | | | | | | | | | | | |
| 254.88 30.63 | END OF BOREHOLE | | 27 | SS | 106 | | 255 | | | | | | | | | | | | | |
| | Groundwater encountered at about elev. 263.8m during drilling on March 6, 2013. Water level in piezometer at elev. 267.46m on April 3, 2013. Water level in piezometer at elev. 267.56m on April 25, 2013. Water level in piezometer at elev. 267.54m on June 5, 2013. | | | | | | | | | | | | | | | | | | | |

RECORD OF BOREHOLE No 402

1 OF 1

METRIC

PROJECT 12-1132-0076
W.P. 3053-11-00 LOCATION N 4758589.5, E 422223.2 ORIGINATED BY BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WF/LK/AG
DATUM GEODETIC DATE March 7, 2013 CHECKED BY

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT w _p | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
|----------------|------------------------------------------------------------------------------------------------------|------------|---------|------|------------|----------------------------|-----------------|---------------------------------------------|-----------------|------------------|------------|-------------------|------------------------------------|-------------------------------------|-----------------------------------|------------------------------------------|------------------------------------------------------------------|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | | |
| | | | | | | | | ○ UNCONFINED | + FIELD VANE | ● QUICK TRIAXIAL | × LAB VANE | WATER CONTENT (%) | | | | | |
| 285.34 0.00 | PAVEMENT SURFACE ASPHALT | | | | | | 20 40 60 80 100 | 20 40 60 80 100 | 20 40 60 80 100 | 10 20 30 | | | | | | | |
| 0.27 | FILL, granular base Brown | | | | | | | | | | | | | | | | |
| 284.58 0.76 | FILL, sand and gravel, trace silt, with cobbles Brown | | 1 | SS | 10 | | | | | | | | | | | | |
| | SILT, trace to some sand, trace gravel, with clayey silt layers Compact to very dense Brown | | 2 | SS | 19 | | | | | | | | | | | | |
| | | | 3 | SS | 26 | | | | | | | | | | | | |
| | | | 4 | SS | 57 | | | | | | | | | | | | |
| 281.68 3.66 | CLAYEY SILT TILL, trace to some sand, trace gravel Stiff to very stiff Grey | | 5 | SS | 28 | | | | | | | | | | | | |
| | | | 6 | SS | 23 | | | | | | | | | | | | |
| | | | 7 | SS | 14 | | | | | | | | | | | | |
| | | | 8 | SS | 20 | | | | | | | | | | | | |
| | | | 9 | SS | 25 | | | | | | | | | | | | |
| 275.74 9.60 | END OF BOREHOLE | | 10 | SS | 23 | | | | | | | | | | | | |
| | Groundwater not established during drilling on March 7, 2013. | | | | | | | | | | | | | | | | |
| | Standpipe dry on March 8, 2013. | | | | | | | | | | | | | | | | |
| | Installation missing/destroyed on April 3, 2013. | | | | | | | | | | | | | | | | |

RECORD OF BOREHOLE No 403

1 OF 1

METRIC

PROJECT 12-1132-0076
W.P. 3053-11-00 LOCATION N 4758678.9 , E 422216.9 ORIGINATED BY BT
DIST HWY 401 BOREHOLE TYPE POWER AUGER, HOLLOW STEM COMPILED BY WF/LK/AG
DATUM GEODETIC DATE March 7, 2013 CHECKED BY

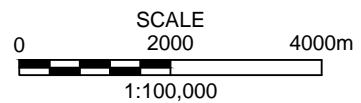
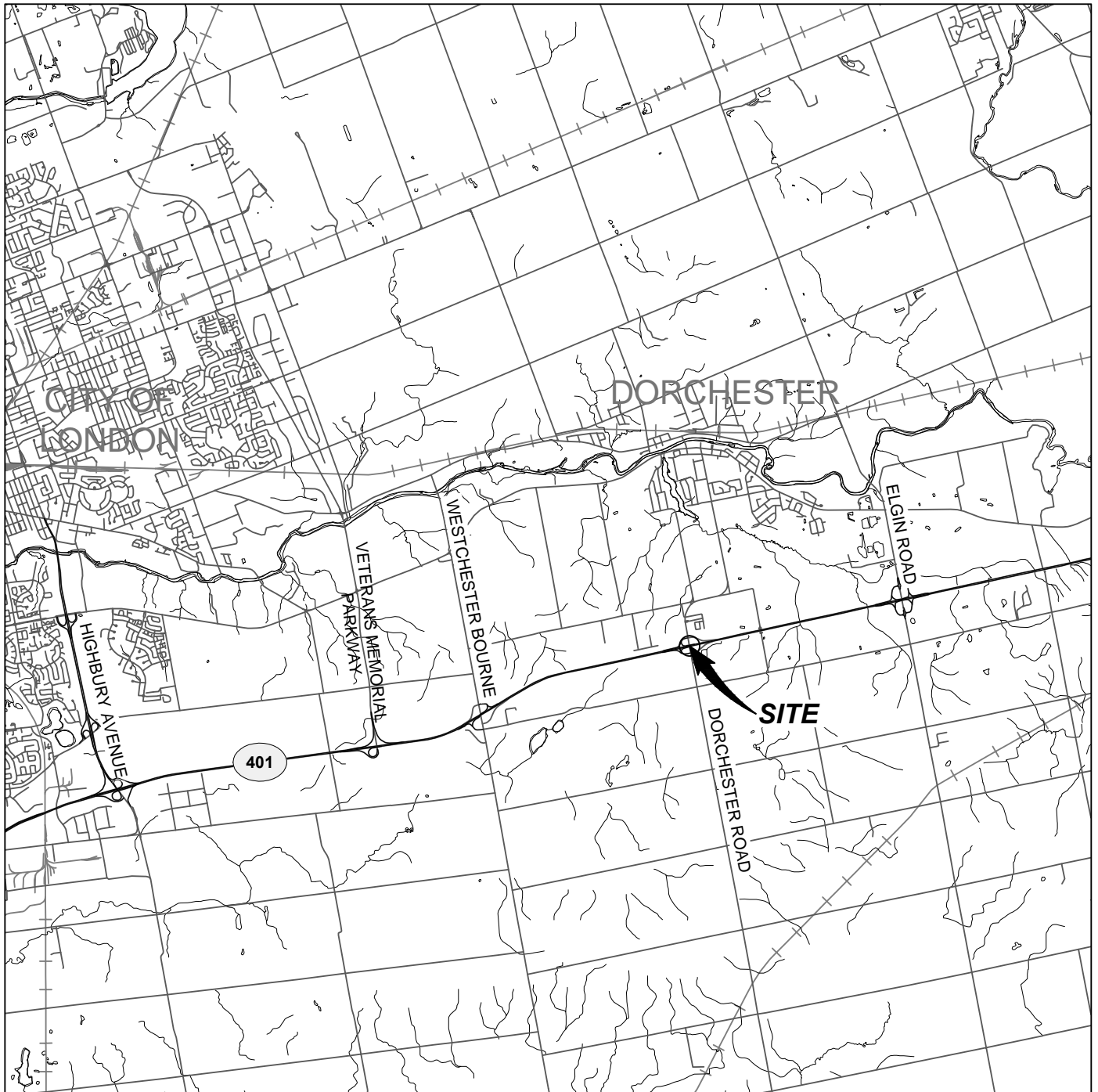
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT CONTENT | | | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | | | | |
|---------------|-----------------------------------------------------------------------------------------------------------------------------|------------|---------|------|------------|----------------------------|-----------------|---------------------------------------------|----|----|----|-----|--------------------------------------------------------------|--------------|------------------|-------------------------|---------------------------------------------------|----|----|----|----|------------|-------------------|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | W _P W W _L | | | | kN/m ³ | GR | SA | SI | CL | | |
| | | | | | | | | 20 | 40 | 60 | 80 | 100 | ○ UNCONFINED | + FIELD VANE | ● QUICK TRIAXIAL | | | | | | | × LAB VANE | WATER CONTENT (%) |
| 286.14 | PAVEMENT SURFACE | | | | | | | | | | | | | | | | | | | | | | |
| 0.00 | ASPHALT | | | | | | 286 | | | | | | | | | | | | | | | | |
| 0.24 | FILL, granular base | | | | | | | | | | | | | | | | | | | | | | |
| 0.43 | Brown | | | | | | | | | | | | | | | | | | | | | | |
| | FILL, sand and gravel, some silt, with cobbles | | 1 | SS | 20 | | | | | | | | | | | | | | | | | | |
| 284.77 | Compact Brown | | | | | | 285 | | | | | | | | | | | | | | | | |
| 1.37 | SILT, trace clay, trace sand Compact Brown | | 2 | SS | 13 | | | | | | | | | | | | | | | | | | |
| 284.01 | | | | | | | 284 | | | | | | | | | | | | | | | | |
| 2.13 | CLAYEY SILT, trace sand Very stiff Brown | | 3 | SS | 20 | | | | | | | | | | | | | 0 | 1 | 74 | 25 | | |
| 283.24 | | | | | | | 283 | | | | | | | | | | | | | | | | |
| 2.90 | CLAYEY SILT TILL, trace to some sand, trace gravel Very stiff to hard Brown becoming grey at about elev. 281.0m | | 4 | SS | 29 | | 282 | | | | | | | | | | | | | | | | |
| | | | 5 | SS | 31 | | | | | | | | | | | | | | | | | | |
| | | | 6 | SS | 31 | | | | | | | | | | | | | | | | | | |
| | | | | | | | 281 | | | | | | | | | | | | | | | | |
| | | | 7 | SS | 26 | | | | | | | | | | | | | | 1 | 8 | 52 | 39 | |
| | | | 8 | SS | 20 | | 280 | | | | | | | | | | | | | | | | |
| | | | | | | | 279 | | | | | | | | | | | | | | | | |
| | | | 9 | SS | 20 | | 278 | | | | | | | | | | | | | 3 | 11 | 51 | 35 |
| | | | | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | 277 | | | | | | | | | | | | | | | | |
| 276.54 | END OF BOREHOLE | | 10 | SS | 27 | | | | | | | | | | | | | | | | | | |
| 9.60 | Groundwater not established during drilling on March 7, 2013. | | | | | | | | | | | | | | | | | | | | | | |

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

| | | | | | |
|---------------------------------------|--------------------------------------------------------------------|----------------------------------------|--|--------|---------------|
| PROJECT <u>12-1132-0076</u> | | RECORD OF BOREHOLE No 404 | | 2 OF 2 | METRIC |
| W.P. <u>3053-11-00</u> | LOCATION <u>N 4758666.6 , E 422219.5</u> | ORIGINATED BY <u>DH</u> | | | |
| DIST <u> </u> HWY <u>401</u> | BOREHOLE TYPE <u>POWER AUGER, HOLLOW STEM / WASH BORING, CASED</u> | COMPILED BY <u>WF/LK/AG</u> | | | |
| DATUM <u>GEODETIC</u> | DATE <u>March 12, 2013 - March 13, 2013</u> | CHECKED BY <u> </u> | | | |

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT | | | UNIT WEIGHT γ kN/m³ | 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 _p | W | W _L | | |
| | CLAYEY SILT TILL, trace to some sand, trace gravel Very stiff to hard Brown becoming grey at about elev. 279.2m | | | | | ▽ | 271 | | | | | | | | | | |
| | | | 15 | SS | 48 | | 270 | | | | | | | | | | |
| | | | | | | | 269 | | | | | | | | | | |
| | | | 16 | SS | 37 | | 268 | | | | | | | | | | |
| 267.42 18.75 | CLAYEY SILT, trace sand Very stiff to hard Grey | | | | | | 267 | | | | | | | | | | |
| | | | 17 | SS | 38 | 266 | | | | | | | | | | | |
| | | | | | | 265 | | | | | | | | | | | |
| | | | 18 | SS | 26 | 264 | | | | | | | | | | | |
| 264.53 21.64 | SAND, fine to medium, some silt, trace to some gravel Dense to very dense Grey | | | | | | 265 | | | | | | | | 0 1 51 48 | | |
| | | | 19 | SS | 33 | 264 | | | | | | | | | | | |
| | | | | | | 263 | | | | | | | | | | | |
| | | | 20 | SS | 30 | 262 | | | | | | | | | | | |
| | | | | | | 261 | | | | | | | | | | | |
| | | | 21 | SS | 100/ 275mm | 260 | | | | | | | | | | 7 76 (17) | |
| | | | | | | 259 | | | | | | | | | | | |
| | | | 22 | SS | 100/ 275mm | 258 | | | | | | | | | | | |
| 257.52 28.65 | END OF BOREHOLE | | 23 | SS | 100 | | | | | | | | | | | | |
| | Groundwater encountered at about elev. 264.5m during drilling on March 13, 2013. | | | | | | | | | | | | | | | | |

LDN_MTO_06 12-1132-0076-2001.GPJ LDN_MTO.GDT 20/11/13



REFERENCE

PLAN BASED ON CANMAP STREETFILES V.2008.5.

NOTE

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

PROJECT MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303
HIGHWAY 401 INTERCHANGE IMPROVEMENTS
GWP 3053-11-00

TITLE

KEY PLAN



| | | | |
|--------------------------|---------|---------------------------------|----------------|
| PROJECT No. 12-1132-0076 | | FILE No. 1211320076-2001-F02001 | |
| CADD | LMK/AMG | AUG. 14/13 | SCALE AS SHOWN |
| CHECK | | | REV. 0 |

FIGURE 1

METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 3053-11-00



COUNTY RD. 32 (Dorchester Rd)
HIGHWAY 401 INTERCHANGE IMPROVEMENTS

SHEET

BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



KEY PLAN

SCALE IN KILOMETRES
0 1 2

LEGEND

- Borehole - Current Investigation
- Seal
- Standpipe
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL measured on June 5, 2013
- WL encountered during drilling
- DRY Water level not established

| No. | ELEVATION | CO-ORDINATES (MTM ZONE 11) | |
|-----|-----------|----------------------------|-----------|
| | | NORTHING | EASTING |
| 401 | 285.51 | 4 758 600.3 | 422 220.7 |
| 402 | 285.34 | 4 758 589.5 | 422 223.2 |
| 403 | 286.14 | 4 758 678.9 | 422 216.9 |
| 404 | 286.17 | 4 758 666.6 | 422 219.5 |

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.

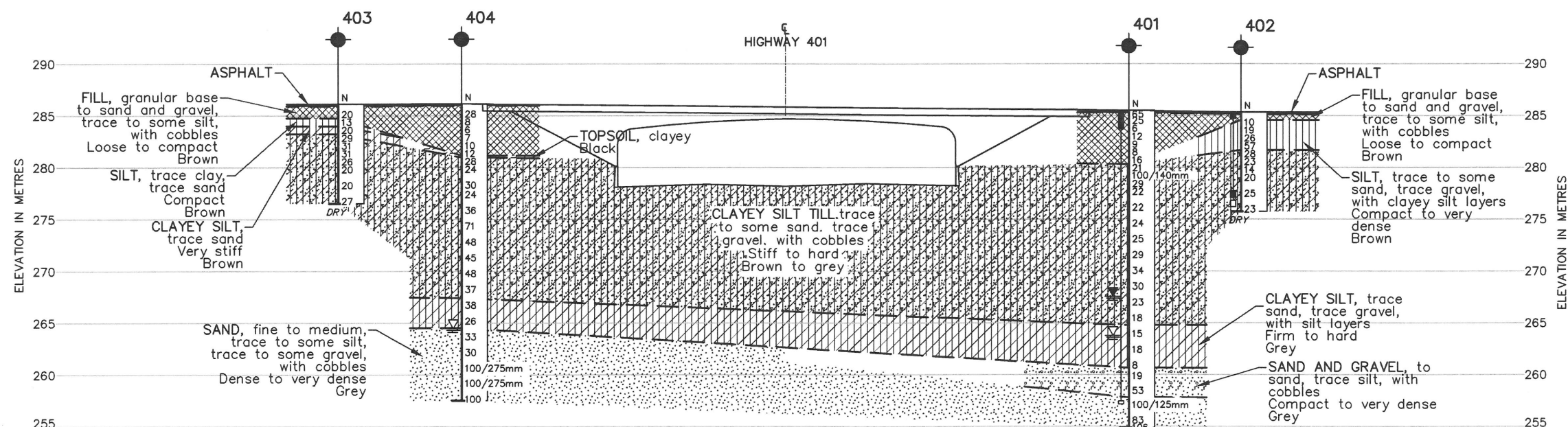
| NO. | DATE | BY | REVISION |
|-------------|-----------|-------------|--------------|
| Geocres No. | 40114-155 | | |
| HWY. | 401 | PROJECT NO. | 12-1132-0076 |
| SUBM'D. | NG | CHKD. | DUP |
| DRAWN: | WDF | CHKD. | AMH |
| | | DATE: | Apr 17/15 |
| | | APPD. | FJH |
| | | SITE: | 19-303 |
| | | DWG. | 1 |



PROFILE ALONG COUNTY ROAD 32 (DORCHESTER ROAD)

HORIZONTAL SCALE
5 0 5m

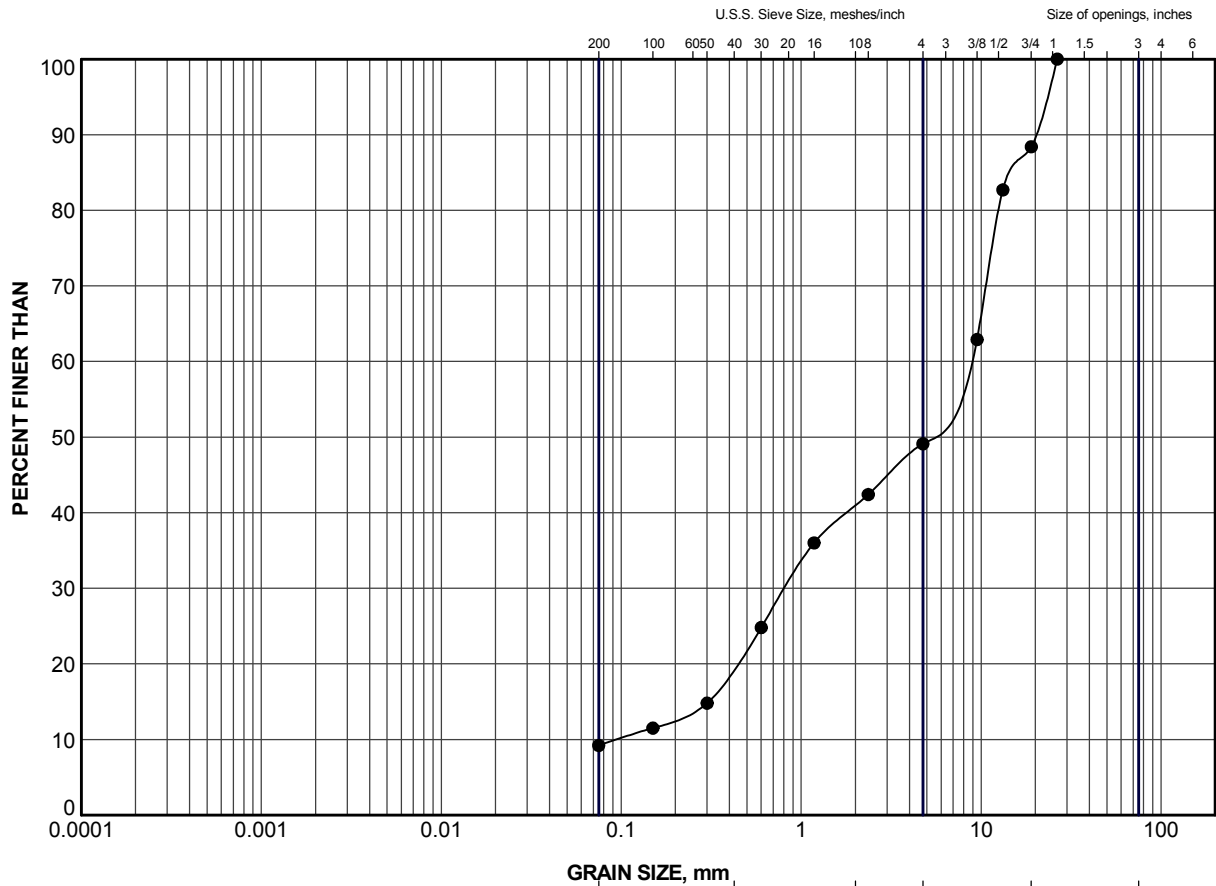
VERTICAL SCALE
5 0 5m





APPENDIX A


Laboratory Test Data

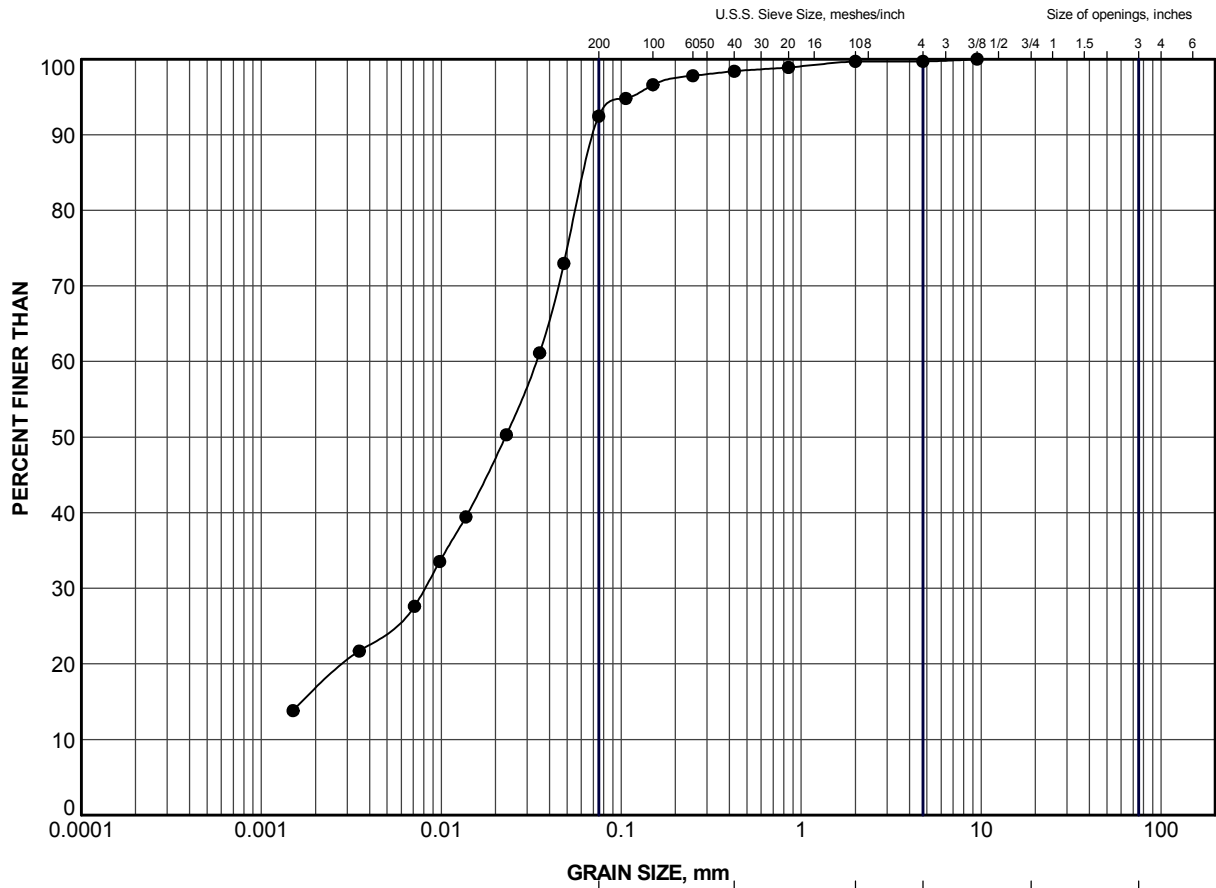


| | | | | | | |
|---------------|-----------|--------|--------|-------------|--------|----------------|
| CLAY AND SILT | fine | medium | coarse | fine | coarse | Cobble Size |
| | SAND SIZE | | | GRAVEL SIZE | | |

LEGEND

| | | | |
|--------|----------|--------|----------|
| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
| ● | 401 | 4 | 283.0 |

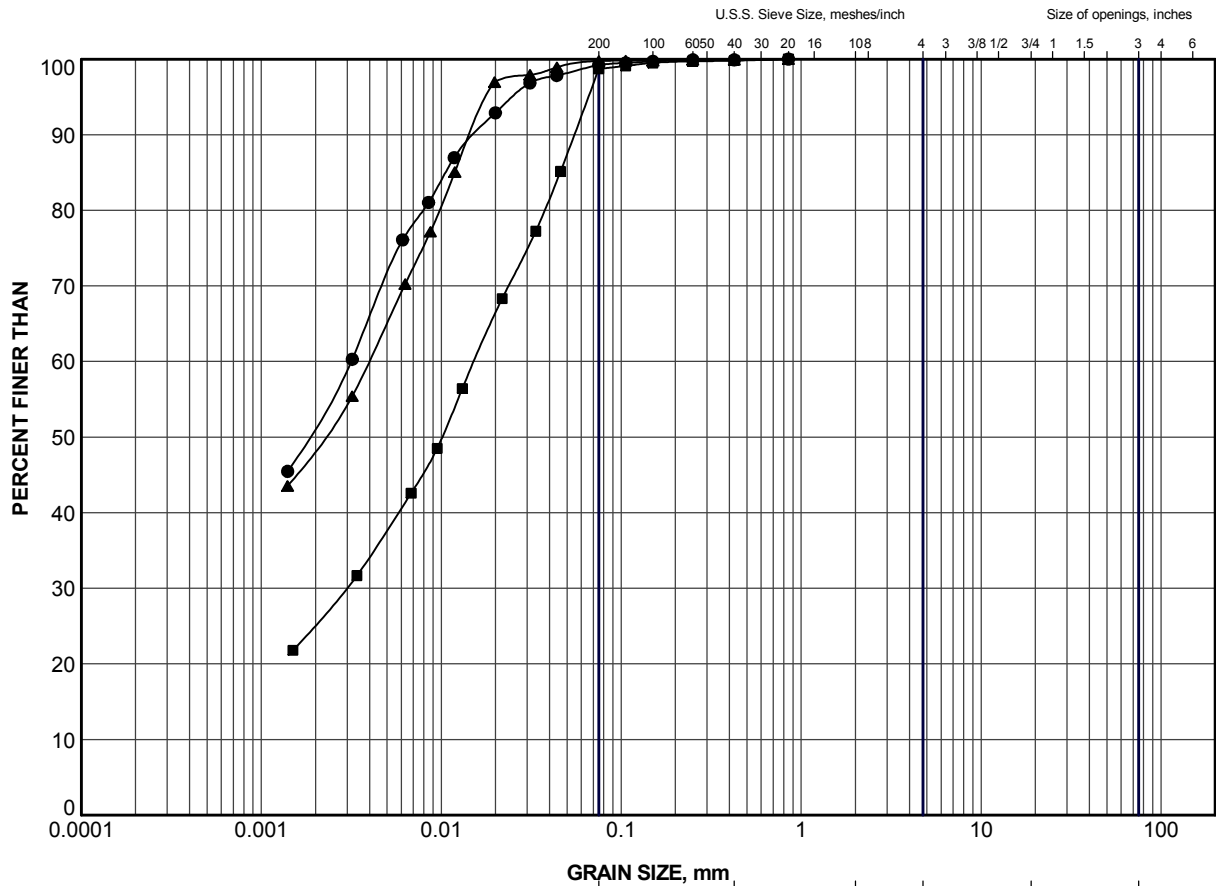
| | | | | | | | |
|---------------------------------------------------------------------------------------------------------------------------------|-----|---------------------------------|--|----------------------------------------------------------------------------------------------------|--|-----|--|
| PROJECT | | | | MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00 | | | |
| TITLE | | | | GRAIN SIZE DISTRIBUTION FILL | | | |
| PROJECT No.12-1132-0076-2001 | | FILE No. 1211320076-2001-F020A1 | | SCALE | | N/A | |
| DRAWN | WDF | Apr 04/13 | | REV. | | | |
| CHECK | | | | | | | |
|  Golder Associates LONDON, ONTARIO | | | | FIGURE A-1 | | | |

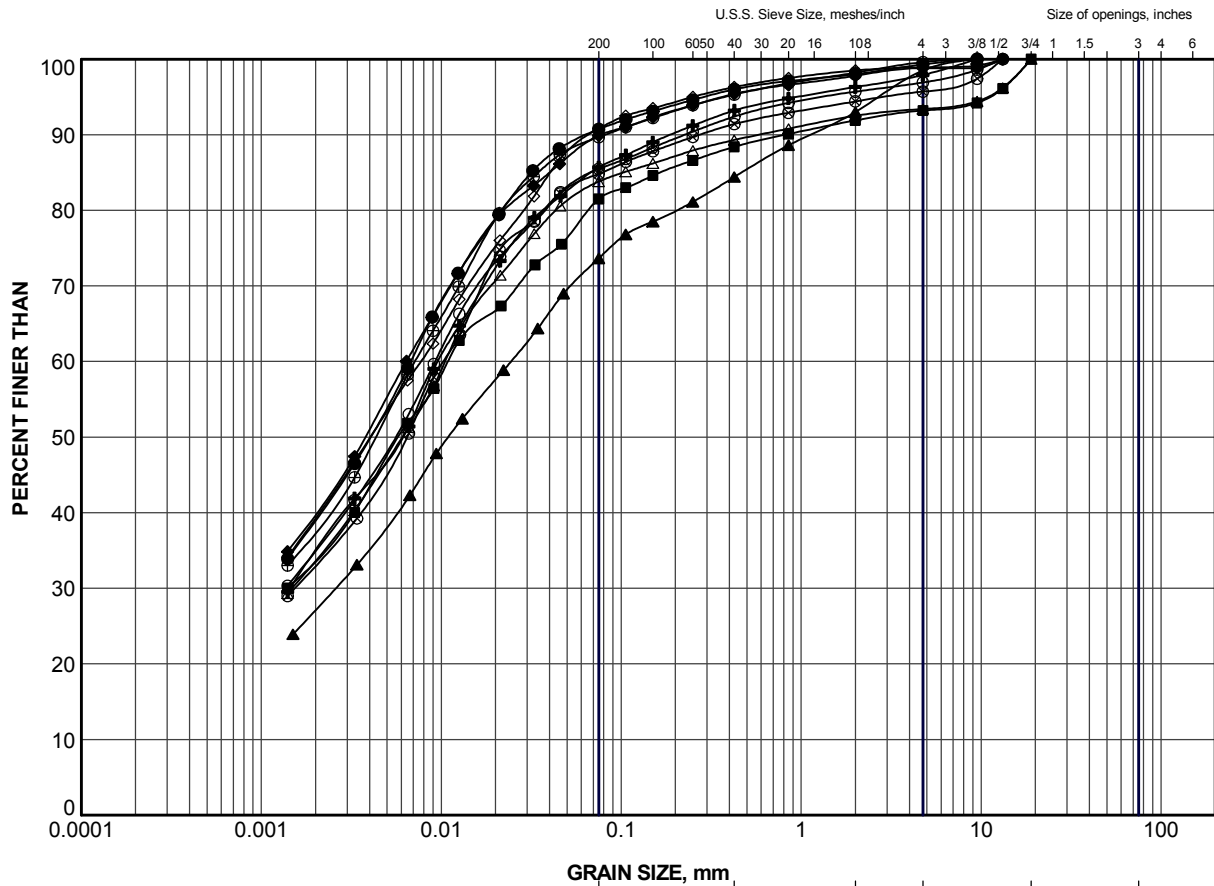


| | | | | | | |
|---------------|-----------|--------|--------|-------------|--------|----------------|
| CLAY AND SILT | fine | medium | coarse | fine | coarse | Cobble Size |
| | SAND SIZE | | | GRAVEL SIZE | | |
| | | | | | | |

| LEGEND | | | |
|--------|----------|--------|----------|
| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
| ● | 402 | 3 | 282.8 |

| | | | | | | | |
|---------------------------------------------------------------------------------------------------------------------------------|--|------------------------------|-----|----------------------------------------------------------------------------------------------------|-------------------|-----|------|
| PROJECT | | | | MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00 | | | |
| TITLE | | | | GRAIN SIZE DISTRIBUTION SILT | | | |
|  Golder Associates LONDON, ONTARIO | | PROJECT No.12-1132-0076-2001 | | FILE No. 1211320076-2001-F020A2 | | | |
| | | DRAWN | WDF | Apr 04/13 | SCALE | N/A | REV. |
| | | CHECK | | | FIGURE A-2 | | |





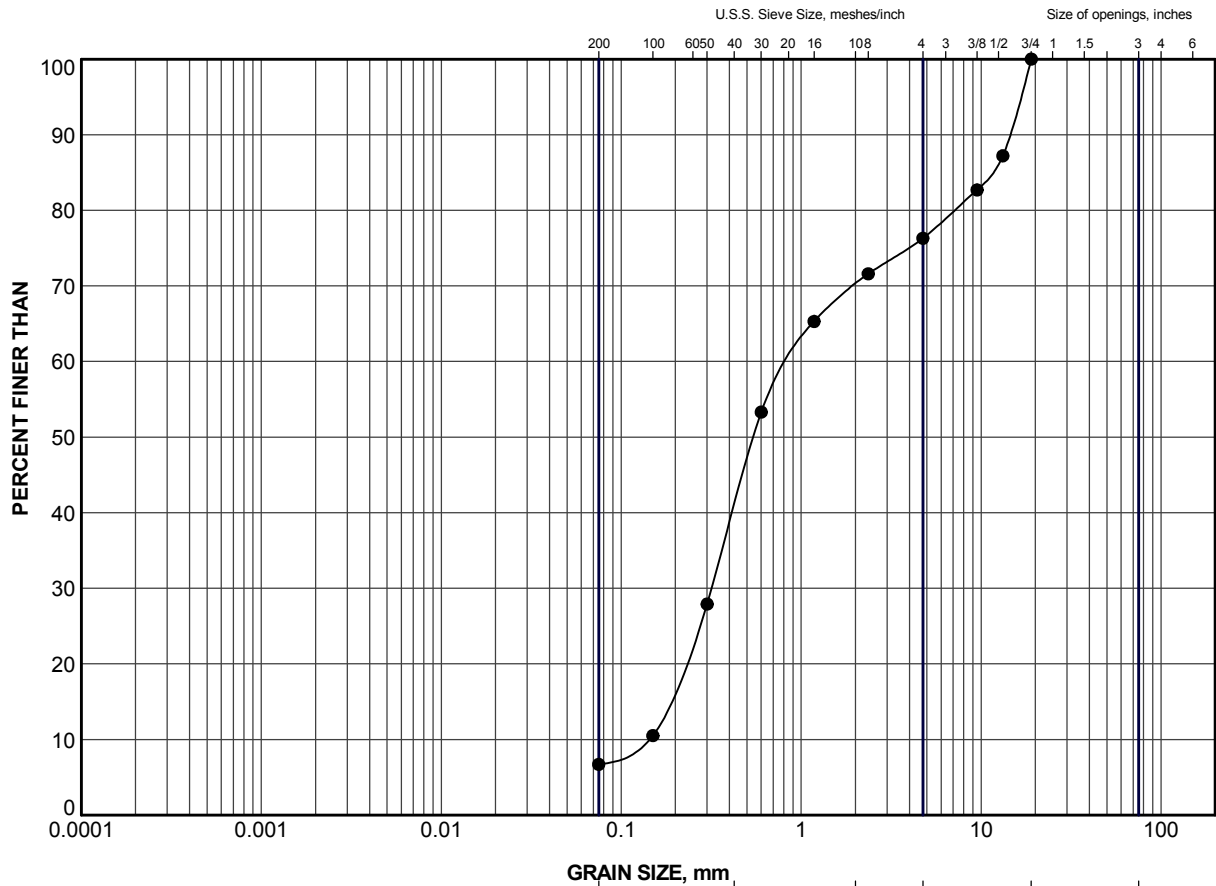
| CLAY AND SILT | GRAVEL SIZE, mm | | | | | Cobble Size |
|---------------|-----------------|--------|--------|-------------|--------|-------------|
| | fine | medium | coarse | fine | coarse | |
| | SAND SIZE | | | GRAVEL SIZE | | |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|--------|----------|--------|----------|
| ● | 401 | 8 | 279.9 |
| ■ | 401 | 14 | 273.1 |
| ▲ | 401 | 17 | 268.5 |
| + | 402 | 6 | 280.5 |
| ◆ | 402 | 8 | 279.0 |
| ◇ | 403 | 7 | 280.5 |
| ○ | 403 | 9 | 278.3 |
| △ | 404 | 8 | 279.9 |
| ⊗ | 404 | 11 | 275.9 |
| ⊕ | 404 | 14 | 271.3 |

| | | | | | | | |
|------------------------------|--|---------------------------------|--|----------------------------------------------------------------------------------------------------|--|------------|--|
| PROJECT | | | | MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00 | | | |
| TITLE | | | | GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL | | | |
| PROJECT No.12-1132-0076-2001 | | FILE No. 1211320076-2001-F020A4 | | SCALE | | N/A | |
| DRAWN | | WDF | | Apr 04/13 | | REV. | |
| CHECK | | | | | | FIGURE A-4 | |




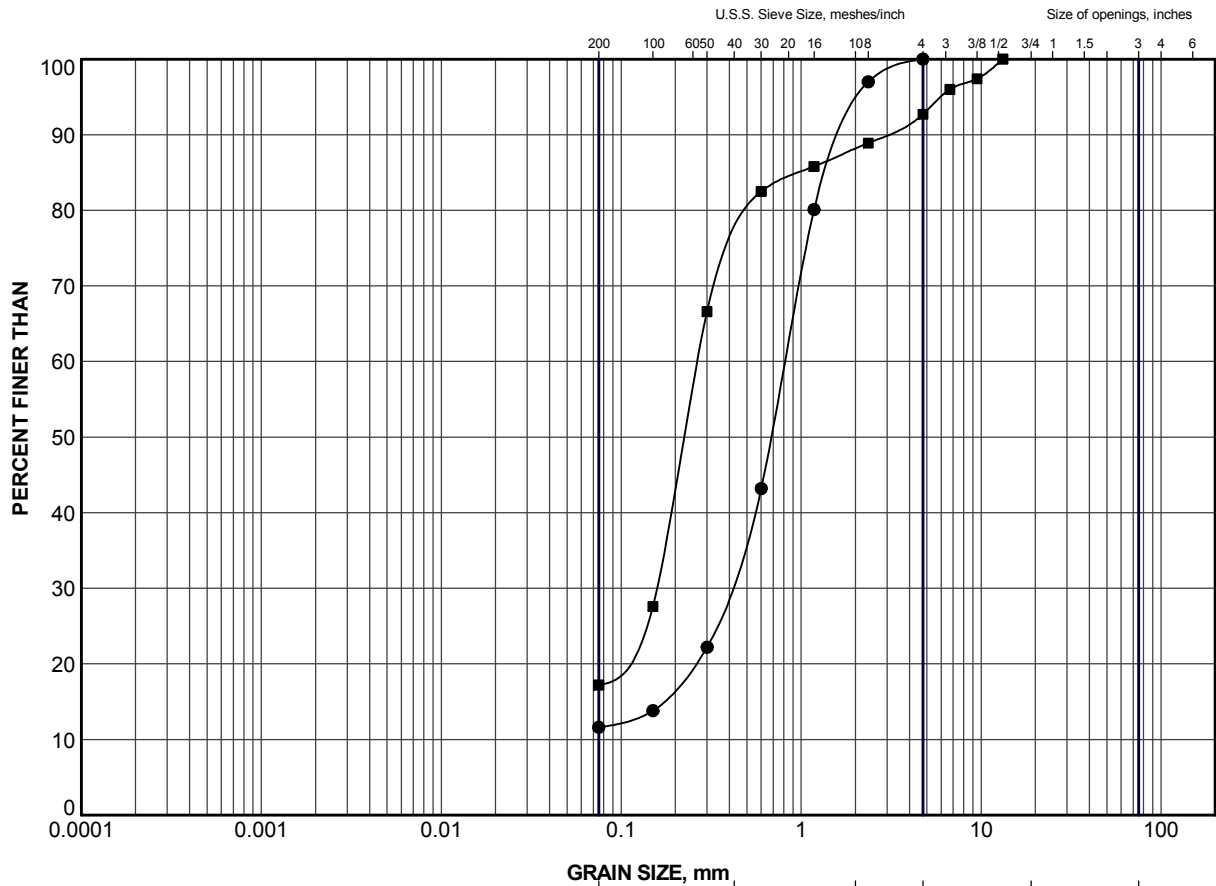


| | | | | | | |
|---------------|---------------|--------|--------|-------------|--------|-------------|
| CLAY AND SILT | SAND SIZE, mm | | | | | Cobble Size |
| | fine | medium | coarse | fine | coarse | |
| | SAND SIZE | | | GRAVEL SIZE | | |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|--------|----------|--------|----------|
| ● | 401 | 23 | 260.0 |


| | | | | | | | |
|---------------------------------------------------------------------------------------------------------------------------------|--|-----|--|----------------------------------------------------------------------------------------------------|--|----------------|--|
| PROJECT | | | | MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00 | | | |
| TITLE | | | | GRAIN SIZE DISTRIBUTION SAND AND GRAVEL | | | |
| PROJECT No.12-1132-0076-2001 | | | | FILE No. 1211320076-2001-F020A5 | | | |
| DRAWN | | WDF | | Apr 04/13 | | SCALE N/A REV. | |
| CHECK | | | | | | FIGURE A-5 | |
|  Golder Associates LONDON, ONTARIO | | | | | | | |

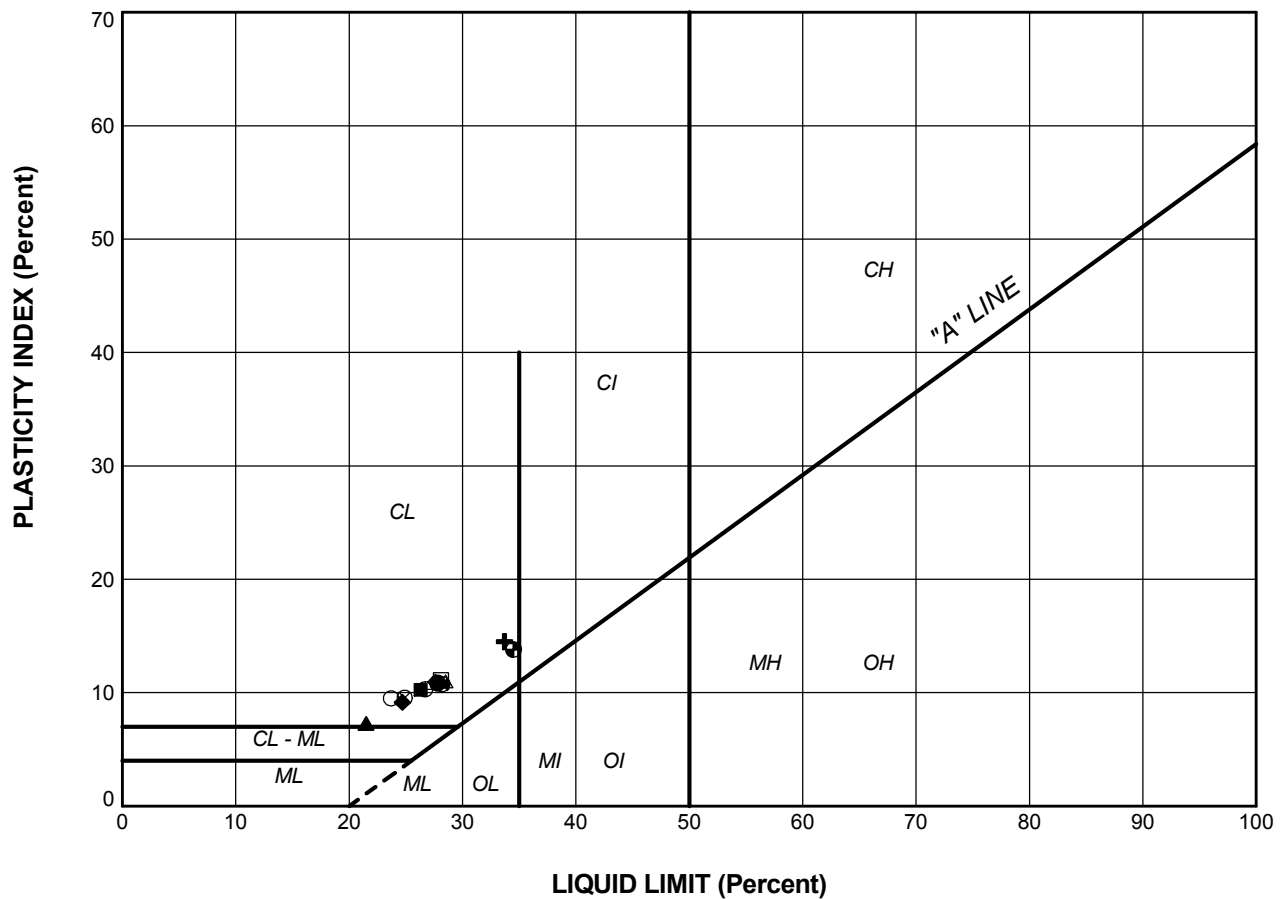


| | | | | | | |
|---------------|-----------------|--------|--------|-------------|--------|----------------|
| CLAY AND SILT | GRAVEL SIZE, mm | | | | | Cobble Size |
| | fine | medium | coarse | fine | coarse | |
| | SAND SIZE | | | GRAVEL SIZE | | |

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEV (m) |
|--------|----------|--------|----------|
| ● | 401 | 25 | 257.1 |
| ■ | 404 | 21 | 260.7 |

| | | | | | | | |
|---------------------------------------------------------------------------------------------------------------------------------|--|-----|--|----------------------------------------------------------------------------------------------------|--|----------------|--|
| PROJECT | | | | MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00 | | | |
| TITLE | | | | GRAIN SIZE DISTRIBUTION SAND | | | |
| PROJECT No.12-1132-0076-2001 | | | | FILE No. 1211320076-2001-F020A6 | | | |
| DRAWN | | LMK | | Jun 14/13 | | SCALE N/A REV. | |
| CHECK | | | | | | FIGURE A-6 | |
|  Golder Associates LONDON, ONTARIO | | | | | | | |



LEGEND

| SYMBOL | BOREHOLE | SAMPLE | LL(%) | PL(%) | PI |
|-------------------------|----------|--------|-------|-------|------|
| CLAYEY SILT TILL | | | | | |
| ● | 401 | 8 | 27.8 | 17.0 | 10.9 |
| ■ | 401 | 14 | 26.3 | 16.1 | 10.3 |
| ▲ | 401 | 17 | 21.5 | 14.2 | 7.3 |
| ◆ | 402 | 6 | 24.7 | 15.6 | 9.2 |
| ◇ | 402 | 8 | 27.6 | 16.7 | 11.0 |
| △ | 403 | 7 | 28.5 | 17.5 | 11.0 |
| ⊗ | 403 | 9 | 24.9 | 15.4 | 9.6 |
| ⊕ | 404 | 8 | 26.7 | 16.4 | 10.3 |
| □ | 404 | 11 | 28.1 | 17.0 | 11.1 |
| ⊙ | 404 | 14 | 28.2 | 17.5 | 10.8 |
| CLAYEY SILT | | | | | |
| + | 401 | 21 | 33.7 | 19.2 | 14.5 |
| ○ | 403 | 3 | 23.7 | 14.2 | 9.5 |
| ⊙ | 404 | 18 | 34.5 | 20.7 | 13.8 |

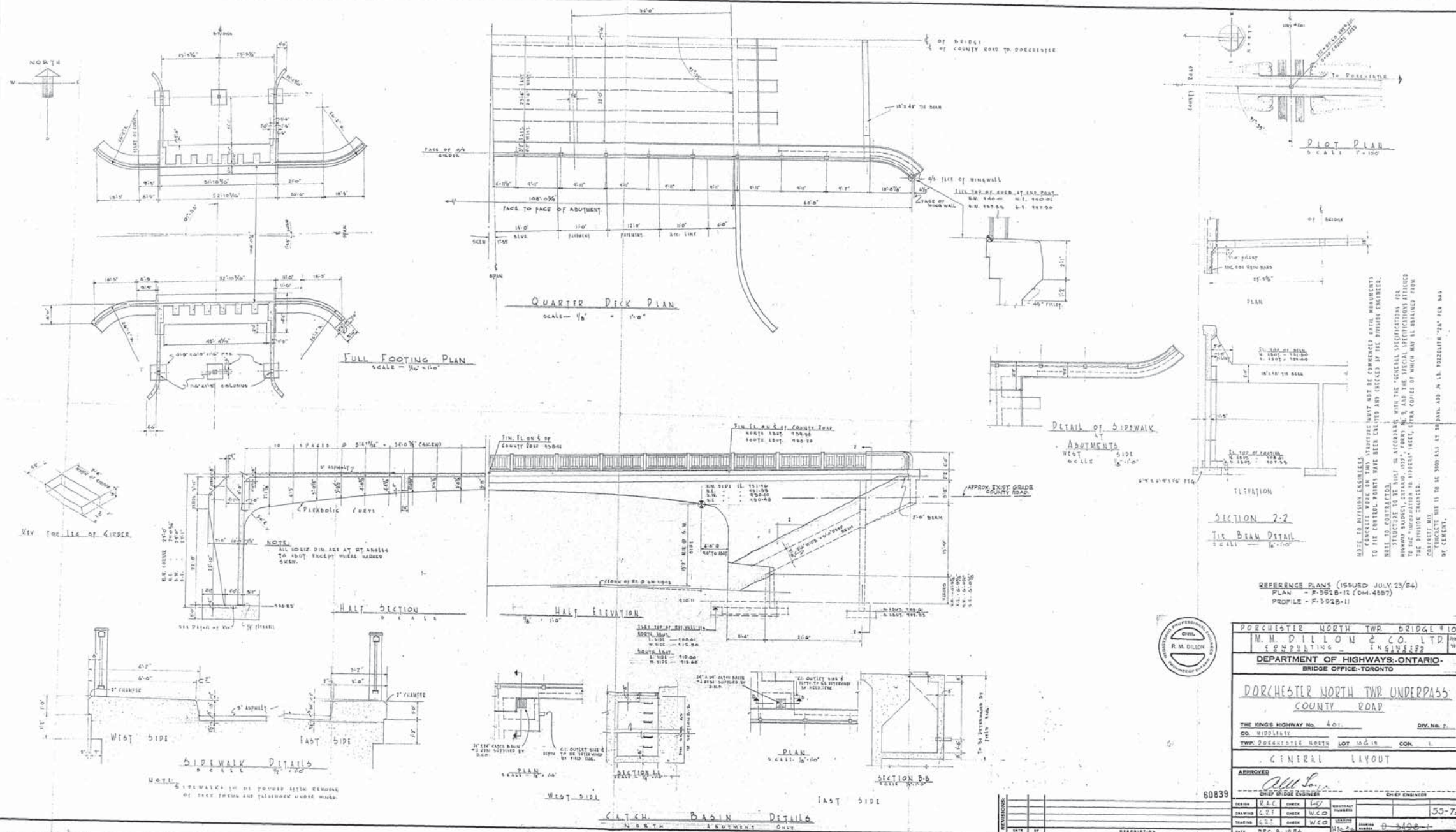
| | | | | | | | |
|------------------------------|---------|------------|-------------------|----------------------------------------------------------------------------------------------------|------|--|--|
| PROJECT | | | | MIDDLESEX ROAD 32 UNDERPASS, SITE 19-303 HIGHWAY 401 INTERCHANGE IMPROVEMENTS GWP 3053-11-00 | | | |
| TITLE | | | | | | | |
| PLASTICITY CHART | | | | | | | |
| PROJECT No.12-1132-0076-2001 | | | | FILE No. 1211320076-2001-F020A7 | | | |
| DRAWN | WDF/AMG | AUG. 14/13 | SCALE | N/A | REV. | | |
| CHECK | | | FIGURE A-7 | | | | |
| | | | | | | | |





APPENDIX B

DHO Drawing Nos. D-3498-1 and 2



PORCHESTER NORTH TWP BRIDGE # 10
 M. M. DILLON & CO. LTD.
 CONSULTING ENGINEERS
 BRIDGE OF HIGHWAYS-ONTARIO
 BRIDGE OFFICE-TORONTO

PORCHESTER NORTH TWP UNDERPASS
 COUNTY ROAD

THE KING'S HIGHWAY NO. 401. DIV. NO. 7
 CO. WINDSOR

TWP. 26 CANTONMENT SOUTH LOT 16 & 18 CON. 1

GENERAL LAYOUT

APPROVED
Wm. L. ...
 CHIEF BRIDGE ENGINEER

CHIEF ENGINEER

| | | | | | |
|---------|-----|------|---|----------|---|
| DRAWN | RAC | CHEK | W | DESIGNED | W |
| DRAWN | W | CHEK | W | DESIGNED | W |
| TRACING | W | CHEK | W | DESIGNED | W |

55-72

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.

| | |
|---------------|-------------------|
| Africa | + 27 11 254 4800 |
| Asia | + 852 2562 3658 |
| Australasia | + 61 3 8862 3500 |
| Europe | + 356 21 42 30 20 |
| North America | + 1 800 275 3281 |
| South America | + 55 21 3095 9500 |

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