

PRELIMINARY
FOUNDATION INVESTIGATION
AND DESIGN REPORT

PROPOSED HIGHWAY 401 UNDERPASS STRUCTURE
AT Highbury Avenue
CITY OF LONDON, COUNTY OF MIDDLESEX

G.W.P. 3032-11-00
Agreement # 3011-E-0019



I.E.
Group

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PROPOSED HIGHWAY 401 & Highbury Avenue
Interchange Reconfiguration
City of London, County of Middlesex

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Agreement # 3011-E-0019

Prepared for:

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MTO GEOCREs No. 40I14-148

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PART A – FOUNDATION INVESTIGATION

1.0 INTRODUCTION

Infrastructure Engineering Group Inc. (IE Group) has been retained by the Ministry of Transportation Ontario (MTO) to prepare a preliminary foundation investigation and design report for the proposed interchange improvements of the Highway 401 underpass structure at Highbury Avenue. It is understood that the new structure will be located offset to the west of the existing alignment to facilitate traffic staging.

Four (4) boreholes were drilled between February 25 and 27, 1959 by MTO for the existing structure and reported in the MTO GEOCREs Report 40I14-63.

The Client Supplied Materials consists of the following:

- 1) Cover Sheet of Book 1, Metric 94-401, MTO Engineering & Title Records, King's Highway 401, Geographic Township Westminster, County Middlesex, 10+000.000 to 33+023.841;
- 2) ETR Plate No. 94-401/54-0, 56-0 and 58-0, WP PHOTO DTM, Survey May 2006;
- 3) MTO GEOCREs Report No. 40I14-63;
- 4) Figure 11 titled "Highway 401 Improvements Planning Study, Highway 4 Easterly to Highbury Avenue, G.W.P. 476-89-00 - Highbury Avenue Interchange Recommended Plan (2021)", prepared by URS, not dated;
- 5) ETR Plate Sheet with no Plate No., Sheet 43 for Contract No. 2008-3011, Profiles for Highbury Avenue between STA 19+540 to 20+100, prepared by Dillon Consulting signed and sealed on March 10, 2008;

2.0 SITE DESCRIPTION

2.1 Site Location

The original proposed new Highbury Avenue underpass structure was to be located at approximately Station 26+600, approximately 40 m west of the existing structure. Since preparation of the draft report, the preferred location for the proposed new bridge is on the existing alignment.

Photographs of the subject site are presented in Appendix E.

Based on the Highbury Avenue profiles provided (Sheet 43), the existing pavement of Highway 401 at this location is near Elevation 276 m. The existing pavement of Highbury Avenue at the center of the existing structure is near Elevation 283 m. It is assumed that the proposed new underpass structure and Highway 401 will be maintained after for the interchange reconfiguration.

The existing structure consists of four (4) spans. It is assumed that the abutments and three piers of the existing structure are supported on spread footings founded near Elevation 273.71 m (898.0 ft) or lower, as recommended in the MTO GEOCREs Report No. 40I14-63.

2.2 Physiography and Topography

The area of the site is located in the western limit of the physiographic region known as the Westminster Moraine. Geographic information indicates that the general soil conditions at the site consist of the Port Stanley silty clay and clayey silt till with localized lacustrine deposits. The bedrock in this area of the site consists of limestone, dolostone and shale belonging to the Dundee Formation of Middle Devonian Age. The bedrock surface is estimated to be about Elevation 205 m, some 70 m below ground surface.

The existing ground surface on the shoulders of Highway 401, at the location of the proposed structure, is between Elevation 275.0 and 275.5 m. It is assumed that the finished grades of the new Highbury Avenue underpass structure will be similar to those of the existing pavement surface, with an estimated finished pavement at Elevation 283 m.

3.0 INVESTIGATION PROCEDURES

3.1 Field Investigation

Two (2) boreholes were drilled and sampled to obtain data for foundation design of the proposed interchange improvement work. The locations of the boreholes are shown on Drawing 1.

A Diedrich D-120 truck-mounted drill rig was used, supplied by CT Soil between February 14 and 16, 2012, for drilling and Standard Penetration Testing (SPT, following the procedures of ASTM D 1586). The boreholes were drilled using continuous flight hollow stem augers and cased wash-boring technique. Soil samples were retrieved at selected intervals throughout the depths of the boreholes in conjunction with Standard Penetration Tests (SPT). Samples were generally taken at intervals of depth of 1.5 m to 15 m and opened up to 3.0 m to the maximum depth of exploration.

Field pocket penetrometer was used on the retrieved SPT samples, where applicable, to determine the undrained shear strength of the cohesive soil deposits. These undrained shear strengths are used to supplement the properties of the cohesive soils. It is noted that the

measured shear strength value would be slightly lower than the actual value due to sampling disturbance.

The soil samples obtained were placed in labeled containers and transported to IEG's London laboratory for further examination and laboratory testing.

The locations of the boreholes are shown on Drawing 1 and the depths of sampling were as follows:

Borehole No.	Depth of Sampling (m)
1	30.94
2	15.70

A monitoring well was installed in each borehole for future monitoring of the groundwater levels. The monitoring wells were constructed as per the requirements of O.Reg. 903, with a 1.5 m sand packed screen and grouted to the ground surface with a combination of Quickgrout and Holeplug. A flush mount protective casing set in concrete is also provided at the ground surface for protection.

Our field engineer, Mr. Ralph Billings, P. Eng., supervised the fieldwork and worked under the direction of the project engineer, Mr. Eric Chung, P. Eng. Our field staff cleared the location of buried utilities and logged the boreholes.

The MTM coordinates and ground surface elevations at the as drilled borehole locations were surveyed by AGM, OLS, our surveying sub-consultant.

The results of the drilling, sampling, in-situ testing and groundwater observations are summarized on the Record of Borehole sheets and enclosed in Appendix "B".

3.2 Laboratory Analysis

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all retrieved soil samples. In addition, grain size analyses and Atterberg Limit tests were performed on selected soil samples.

The results of the laboratory testing are presented on the Record of Borehole sheets (Appendix "B"), and Laboratory Test Results (Figures 1 to 10, Appendix "C").

4.0 SUBSURFACE CONDITIONS

4.1 General Subsurface Conditions

Reference is made to the Record of Borehole sheets (Appendix “B”) and Laboratory Test Results (Appendix “C”) for detailed subsurface soil and groundwater conditions encountered in the boreholes. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and, consequently, represent transitions between soil types rather than exact planes of geological change. The soil profiles depicting the subsurface conditions on Drawing 1 will vary between and beyond the borehole locations.

The existing ground surface on the shoulders of Highway 401, at the location of the proposed structure, is between Elevation 275.0 and 275.5 m. It is assumed that the finished grades of the new Highbury Avenue underpass structure will be similar to those of the existing pavement surface, with an estimated finished pavement at Elevation 283 m.

The subsurface deposits at the site consist of loose to compact fill (bottom Elevation between 272.75 m and 273.69 m) placed on an upper deposit of compact to very dense silty sand to silt deposit. The upper silty sand to silt deposit extends to a depth of 14.48 m below the ground surface of Borehole 1, and extends beyond the vertical limit of Borehole 2 at a maximum depth of 15.70 m (Elevation 259.67 m).

The upper silty sand to silt layers in Borehole 1 is underlain by a dense to very dense/hard layered silt and clayey silt deposit which extends to 18.84 m below the present ground surface (Elevation 256.20 m).

The layered silt and clayey silt deposit in Borehole 1 is underlain by a very dense lower sand to silty sand deposit which extends to a depth of 27.43 m below the present ground surface (Elevation 247.61 m).

The lower sand to silty sand deposit in Borehole 1 is further underlain by a stiff to hard clayey silt which extends beyond the vertical limit of the borehole at a maximum depth of 30.94 m (Elevation 244.10 m).

Groundwater was measured at a depth of 5.30 m (Elevation 269.74 m) in the lower sand to silty sand layer in Borehole 1 and 3.50 m (Elevation 271.87 m) in the upper silty sand to sand layer in Borehole 2.

The following is a detailed description of the subsurface conditions encountered.

4.1.1 Asphalt and Fill

Borehole 1 penetrated 200 mm thick layer of asphalt underlain by a 710 mm thick layer of granular fill. The ground surface at the location of Borehole 2 is covered with a 460 mm thick layer of silty sand fill.

The granular and silty sand fill in both boreholes are underlain by a layer of brown silty clay fill, with some sand to some sand and gravel.

A single grain size distribution analyses and a single Atterberg limits determination were carried out on the silty sand fill and the results are shown on Figures 1 and 2 of Appendix "C", respectively. Atterberg limit determination carried out on the silty sand fill yielded a liquid limit, plastic limit and a plasticity index of 22%, 19% and 3% and the fill material is classified as a silty sand material with little plasticity (SM).

A single grain size distribution analyses and a single Atterberg limits determination were carried out on the silty clay fill and the results are shown on Figures 1 and 2 of Appendix "C", respectively. Atterberg limit determination carried out on the silty clay fill yielded a Liquid Limit, Plastic Limit and a Plasticity Index of 42%, 19% and 23% and the silty clay fill material is classified as a silty clay material with intermediate plasticity (CI).

Standard penetration tests taken on the fill layers yielded "N"-values of between 5 and over 200 blows per 0.3 m. The single "N"-value of over 100 blows per 0.3 m was likely inflated due to frost penetration. The remaining two "N"-values of 5 and 8 blows per 0.3 m indicate that the fill material is generally loose.

Moisture content of two samples of the silty sand fill material yielded results of 3 and 15%. A single moisture content of the silty clay fill yielded a result of 26%.

4.1.2 Upper Silty Sand to Silt

The fill layers are underlain by an upper silty sand to silt deposit which extends to a depth of 14.48 m below the ground surface of Borehole 1, and extends beyond the vertical limit of Borehole 2 at a maximum depth of 15.70 m (Elevation 259.67 m). This upper silty sand to silt deposit has a brown colour which changes to grey at depths of between 8.8 m and 11.3 m below the present ground surface.

Seventeen (17) grain size distribution analyses were carried out on the silty sand to silt deposit and the results are shown on Figures 3, 4 and 5 of Appendix "C".

Standard penetration tests taken on the upper silty sand to silt deposit yielded "N"-values from 19 to over 100 blows per 0.3 m. In general, "N"-values were from 50 blows to over 100 blows per 0.3 m, with localized lower values of between 19 and 48 blows per 0.3 m within the upper 1.5 m of the deposit, and generally over 50 blows per 0.3 m below

this depth. These values, together with a visual and tactile examination of the soil samples indicated that the upper silty sand to silt deposit is generally very dense, with compact to dense zones within the top 1.5 m of the deposit.

Moisture content determinations carried out on the upper silty sand to silt deposit yielded results of between 16 and 25%, indicating a moist to saturated condition.

Practical refusal with "N"-values of over 100 blows per 0.3 m was achieved at depths of between 4.57 m and 9.60 m (Elevations 270.47 m and 265.90 m) below the present ground surface of Borehole 1. Borehole 1 was extended to a maximum depth of 30.94 m (Elevation 244.10 m) to determine the underlying soil strata.

Borehole 2 was terminated based on practical refusal at depths of between 10.67 m and 14.17 m (Elevations 264.70 m and 261.20 m), with an additional sample taken below to confirm compactness condition of the material.

4.1.3 Layered Silt and Clayey Silt

The upper silty sand to silt deposit in Borehole 1 is underlain by a 4.36 m thick layer of grey layered silt and clayey silt. Two (2) grain size distribution analyses and two (2) Atterberg Limits determinations were carried out on this deposit and the results are presented on Figures 6 and 7 of Appendix "C". The results indicate Liquid Limit of 15 to 20%, Plastic Limit of 12 to 14% with Liquidity Index of 3 to 6%.

Standard penetration tests taken on the layered silt and clayey silt deposit yielded "N"-values of 46 to over 100 blows per 0.3 m, indicating a dense to very dense compactness condition or a hard consistency.

Natural moisture contents of 20 to 22% were obtained from the layered silt to clayey silt, indicating a moist to saturated condition.

4.1.4 Lower Sand to Silty Sand

The layered silt and clayey silt in Borehole 1 is underlain by a lower grey sand to silty sand deposit which extends to a depth of 27.43 m (Elevation 247.61 m). Three (3) grain size distribution analyses were carried out on the lower sand to silty sand deposit and the results are presented on Figure 8 of Appendix "C".

Standard penetration tests taken on the lower sand to silt deposit yielded "N"-values of 70 to over 100 blows per 0.3 m, indicating a very dense compactness condition. Moisture content determinations carried out on the lower sand to silty sand yielded results of between 22% and 24%, indicating a saturated condition.

4.1.5 Lower Clayey Silt

The lower sand to silty sand deposit in Borehole 1 is further underlain by a grey clayey silt deposit with embedded sand and gravel (till). Frequent wet silt seams are present within the upper 1.5 m of the deposit. The lower clayey silt extends beyond the vertical limit of the borehole at a maximum depth of 30.94 m (Elevation 244.10 m).

Two (2) grain size distribution analyses and two (2) Atterberg Limits determinations were carried out on this deposit and the results are presented on Figures 9 and 10 of Appendix "C". The results indicate Liquid Limit of 20 to 21%, Plastic Limit of 12 to 14% with Liquidity Index of 7 to 8%.

Standard penetration tests taken on the lower clayey silt deposit yielded "N"-values of 12 and 52 blows per 0.3 m. These results, together with a visual and tactile examination indicate that the clayey silt is stiff to hard.

Moisture content determinations carried out on the clayey silt yielded results of 27% and 14% indicating a moist to saturated condition.

4.2 Groundwater Conditions

Groundwater was measured in the installed monitoring wells on February 15 and 17, 2012 and on May 31, 2012. The water levels were measured on May 31, 2012 before bailing, with groundwater levels recovered to a similar elevation within 30 minutes after well development by bailing in both boreholes. The following table summarizes the water levels recorded:

Borehole	Date of Monitoring	Water Level in Borehole, m Depth (Elevation)	Remarks
1	Feb. 15, 2012 May 31, 2012	5.30 (269.74) 11.50 (263.54)	Monitoring well installed with screen between 24.84 m and 26.37 m below ground surface (Elevations 250.20 m and 248.67 m). Top of casing at Elevation 274.98 m.
2	Feb. 17, 2012 May 31, 2012	3.50 (271.87) 4.45 (270.92)	Monitoring well installed with screen between 13.87 m and 15.39 m below ground surface (Elevations 261.50 and 259.98 m). Top of casing at Elevation 275.23 m.

The upper silty sand to silt deposit changes in colour from brown to grey at depths of between 8.8 m and 11.3 m below the present ground surface (Elevations 266.57 m and 263.74 m). The brown to grey interface likely reflects a level of permanent saturation. It should be noted that the groundwater level will fluctuate seasonally and in response to weather events.

4.3 Comparison with MTO Geocres Report No. 40I14-63

Two sampled boreholes (MTO BH 1 and 2) and two sets of dynamic cone penetration tests (MTO BH 2 and 4) were carried out for MTO Geocres Report No. 40I14-63.

Borehole No.	Ground Surface Elevation m (ft.)	Depth of Sampling m (ft.)
MTO BH 1	275.23 (903.0)	DCPT to 4.27 (14.0) Sampling to 8.08 (26.5)
MTO BH 2	275.23 (903.0)	DCPT to 3.35 (11.0)
MTO BH 3	275.54 (904.0)	DCPT to 6.10 (20.0) Sampling to 8.08 (26.5)
MTO BH 4	275.54 (904.0)	DCPT to 5.49 (18.0)

MTO BH 1 and 3 penetrated a 0.9 m to 1.4 m thick layer of brown clay fill underlain by dense to very dense light brown fine sand extending beyond the vertical limit of the boreholes at a maximum depth of 8.08 m (Elevations of between 267.16 m and 267.46 m). The recorded standard penetration test results (N-values) are between 60 and 70 blows per 300 mm. Dynamic cone penetration tests (DCPT) were also carried out from the ground surface of MTO BH 1 and 3 and the results are presented as Nc-values. The Nc values of MTO BH 1 and 3 range from 5 and over 150 blows per 300 mm with refusal at depths of 4.3 m and 6.1 m (14 ft. and 20 ft.), i.e. Elevations 270.97 m and 269.44 m, respectively. MTO BH 2 and 4 were put down to depths of (11 ft. and 18 ft.) below the ground surface, with Nc-values ranging from 10 to over 250 blows/300 mm, and refusal with Nc-values of 187 blow per 225 mm and 107 blows per 100 mm, at Elevations 271.88 m and 270.05 m, respectively.

Groundwater was not encountered in any of the 1959 MTO boreholes.

Grain size distribution analyses were not carried out for the 1959 MTO investigation.

Both Boreholes 1 and 2 of the current foundation investigation were put down west of the existing structure, with Borehole 1 simulating the conditions of MTO BH 3 and Boreholes 2 simulating MTO BH 2.

The light brown fine sand layer referred to in the 1959 MTO investigation is identified in the current foundation investigation as silty sand to silt. Additional grain size distribution analyses were carried out for the current investigation for the purpose of soil modelling. Since there is limited geotechnical data from the MTO 1959 Investigation, the geotechnical data collected from this foundation investigation are used for foundation design in this report.

PART B – FOUNDATION DESIGN

5.0 DISCUSSION AND RECOMMENDATIONS

5.1 General

This section of the report provides our recommendations on the geotechnical aspects of foundation design of the proposed replacement of the proposed Highbury Avenue underpass structure over Highway 401 based on our interpretation of the factual information obtained during this 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 to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction method and scheduling.

The original proposed new Highbury Avenue underpass structure was to be located at approximately Station 26+600, approximately 40 m west of the existing structure. Since preparation of the draft report, the preferred location for the proposed new bridge is on the existing alignment.

Based on the Highbury Avenue profiles provided (Sheet 43), the existing pavement of Highway 401 at this location is near Elevation 276 m. The existing pavement of Highbury Avenue at the center of the existing structure is near Elevation 283 m. It is assumed that the proposed new underpass structure and Highway 401 will be maintained after for the interchange reconfiguration.

The existing structure consists of four (4) spans. It is assumed that the abutments and three piers of the existing structure are supported on spread footings founded near Elevation 273.71 m (898.0 ft) or lower, as recommended in the MTO GEOCREs Report No. 40I14-63.

The subsurface deposits at the site consist of loose to compact fill (bottom Elevation between 272.75 m and 273.69 m) placed on an upper deposit of compact to very dense silty sand to silt deposit. The upper silty sand to silt deposit extends to a depth of 14.48 m below the ground surface of Borehole 1, and extends beyond the vertical limit of Borehole 2 at a maximum depth of 15.70 m (Elevation 259.67 m).

The upper silty sand to silt layers in Borehole 1 is underlain by a dense to very dense/hard layered silt and clayey silt deposit which extends to 18.84 m below the present ground surface (Elevation 256.20 m). The layered silt and clayey silt deposit in Borehole 1 is underlain by a very dense lower sand to silty sand deposit which extends to a depth of 27.43 m below the present ground surface (Elevation 247.61 m). The lower sand to silty sand deposit in Borehole 1 is further underlain by a stiff to hard clayey silt which extends beyond the vertical limit of the borehole at a maximum depth of 30.94 m (Elevation 244.10 m).

Groundwater was measured at a depth of 5.30 m (Elevation 269.74 m) in the lower sand to silty sand layer in Borehole 1 and 3.50 m (Elevation 271.87 m) in the upper silty sand to sand layer in Borehole 2.

The type of structure for the proposed Highway 401 underpass at the Highbury Avenue interchange is not known at the time of preparing this preliminary report. It is assumed that the structure will likely be constructed with integral abutments based on current MTO practices. It is anticipated that pile foundations for integral abutments will likely be founded 6 to 10 m below existing ground surface with a pile cap elevation in the order of Elevation 273 m or slightly higher for false abutment construction is chosen.

Shallow spread footing foundations are preferred based on the lower cost. However, integral abutments are not feasible with this foundation option.

5.2 Spread Footing Foundation

It is noted that the results of the MTO and IE Group boreholes correspond well. Spread footing foundations founded in the upper very dense silty sand to silt deposit can be used to support the proposed underpass structure. For preliminary design purposes and assuming a 6 m wide footing, the recommended footing bearing elevation and corresponding bearing resistances are shown below:

Elevation	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
272.75 m	600	350

For this option to be considered, additional boreholes and laboratory testing will need to be carried out at the final location of the proposed underpass structure to determine the competency and consistency of the upper silty sand to silt deposit within the zone of influence of the footing loads.

The sliding resistance of the cast-in-place footings should be checked. The unfactored horizontal resistance (Clause 6.7.5, CAN/CSA-S6-06) against sliding between concrete and undisturbed, silty sand to silt can be calculated using a coefficient of friction of 0.40 as per Table 24.4 CFEM 4th Edition, 2006.

5.3 Driven Pile Foundation

The results of Boreholes 1 and 2 of the present investigation indicate that driven piles can be founded in the very dense upper silty sand to silt deposit. H-piles can be driven to proper set and the following axial resistances are recommended for HP310x110 steel pile:

Factored Axial Resistance at ULS	= 1,600 kN
Axial Resistance at SLS	= 1,100 kN

Borehole Location	Estimated Pile Tip Depth (from present ground surface)	Estimated Pile Tip Elevation
Borehole 1 (SW of existing structure)	6.5 m	268.5 m
Borehole 2 (NW of existing structure)	10.0 m	264.4 m

For this option to be considered, additional deep boreholes will need to be carried out at the final location of the proposed underpass structure to confirm and identify any changes in the pile bearing stratum.

As the site is generally underlain by dense to very dense granular soils and hard cohesive deposits, the potential for down drag (negative skin friction) is not a concern.

The pile foundation should be designed and constructed as per OPSS 903. The Hiley Formula (SS103-11) is applicable for pile driving Control.

Resistance to lateral loads may be provided by mobilization of passive resistance along the pile. Considering the soils condition and the proposed founding elevation of the pile cap, the passive resistance will be mobilized mostly in the compact to very dense upper silty sand to silt deposit and well-compacted granular fill if false abutments are used. For false abutments, passive resistance should not be relied on if the upper 3 m of the piles are surrounded by CSPs and filled with loose uniform sand.

For preliminary estimates, the Lateral Resistance at SLS can be taken as 70 to 80 kN for HP310x110 steel pile along the strong axis. The SLS resistance is the lateral load causing a lateral movement of 10 mm. The estimated Factored Lateral Resistance at ULS is 160 kN.

The horizontal subgrade reaction method can be used to calculate the lateral capacity of the pile. The recommended ranges of modulus of horizontal subgrade reaction, k_h , are provided in the following formula. If the lateral pile capacity is not sufficient to support the lateral loading, battered piles can be employed.

$$k_h = 67 (S_u/d) \quad \text{for cohesive soil, MPa/m}$$

$$= n_h (Z/d) \quad \text{for cohesionless soil, MPa/m}$$

Where d = pile width or diameter (m)
 n_h = constant of horizontal subgrade reaction, MPa/m
 S_u = undrained shear strength of soil, MPa
 Z = depth below ground surface, m

Soil Stratum	Elevation Range (m)	n_h (MPa/m)	Undrained Shear Strength (S_u), (MPa)
Well-compacted Granular Fill	Above existing grades	10	
Clay/Silt Fill	275 to 272		0.05
Silty Sand to Silt	272 to 264	5	

The values for k_h must be reduced based on the closeness of the piles and the direction of the applied lateral loading. The reduction factors are listed in the following table.

Reduction Factors for Pile Spacing in Pile Groups

Pile Spacing Perpendicular to Direction of Loading		Pile Spacing in Direction of Loading	
centerline to centerline distance between Piles	reduction factor (multiply k_h by factor)	centerline to centerline distance between Piles	reduction factor (multiply k_h by factor)
4 d and more	1	8 d and more	1
3.5 d	0.69	6 d	0.7
2.5 d	0.63	4 d	0.4
2.0 d	0.5	3 d and less	0.25

5.4 Lateral Earth Pressures

The lateral earth pressures acting on the abutments, wing walls and retaining walls will depend on the type and method of placement of the backfill materials and on the subsequent lateral movement of the structure whether it is restrained or unrestrained. The lateral earth pressures to be used in the design should be computed in accordance with Section 6.9 of the CAN/CSA-S6-06.

The granular backfill should conform to OPSS 1010 for either Granular “A” or Granular “B” Type III. To maintain free draining characteristics in granular fill materials, the maximum percentage passing the 75 µm (No. 200 sieve) should be limited to 5% (SP110S13).

The backfill should be constructed as per OPSS 902 and 501. A perforated subdrain should be installed behind the walls with a positive outlet or wall drains as per OPSD-3190.100.

The lateral earth pressure, P_h , acting on the abutment walls, wing walls and retaining walls may be computed using the equivalent fluid pressures presented in Clause 6.9.2.3 of the CAN/CSA-S6-06, or employing the following equation based on unfactored earth pressure distributions:

$$P_h = K (\gamma h + q)$$

Where:

- K = earth pressure coefficient, use value from table below
 γ = unit weight of soil, = 21.2 kN/m³ for Granular “B”
= 22.8 kN/m³ for Granular “A”
 h = depth below top of wall, m
 q = live load surcharge pressure, equivalent fill height of 0.8 m
as per Clause 6.9.5 of CHBDC and CAN/CSA-S6-06

Wall Type	Earth Pressure Coefficient (K)	
	Granular “A” $\phi = 35^\circ$	Granular “B” $\phi = 30 \text{ to } 35^\circ$
Restrained Wall (K_o)	0.43	0.50 to 0.43
Unrestrained Wall (K_a)	0.27	0.33 to 0.27

The above parameters are based on a horizontal back slope (not exceeding 5 degrees) behind the headwalls. A compaction surcharge equal to 12 kPa should be included in the lateral earth pressures for the structural design of the headwalls and retaining walls in accordance with Clause 6.9.3 of the CAN/CSA-S6-06.

Vibratory equipment for use behind the abutment walls, wing walls and retaining walls should be restricted in size as per current MTO practices, and should conform to OPSS 501 and SSP105S10.

5.5 Retained Soil System (RSS)

The approximately 5 to 6 m high abutment wall could be constructed as a false abutment wall or Retained Soils System (RSS). The global stability of the RSS walls is expected to have a factor of safety of over 1.5 against global stability failure in light of the typically compact to very dense silty sand to silt deposit underlying the site.

These retaining structures should be designed in accordance with the parameters given in Sections 5.2 and 5.4. The approved supplier of the RSS walls (refer to the DSM listing) should be responsible for design of the structure such as backfill, reinforcement, and internal and external stability. The designer should consult the MTO Designated Sources of Materials (DSM) list for the approved RSS.

The following information should be included in the contract drawing:

- length and location
- height and space constraints
- elevation of top and bottom of RSS
- performance requirement
- appearance requirement

The sliding resistance of the retaining walls should be checked. It is assumed that a Granular “A” bedding layer, compacted to a minimum of 98% of the material’s standard Proctor maximum dry density (SPMDD), will be placed beneath the false abutment wall foundations or RSS structures. The unfactored horizontal resistance (Clause 6.7.5, CAN/CSA-S6-06) against sliding between the concrete and Granular “A” bedding can be calculated using a coefficient of friction of 0.6 as per Table 24.4 CFEM 4th Edition, 2006. The RSS wall design should be carried out as per the MTO RSS Design Guidelines (September 2008).

The backfill to be used for embankment construction will likely consist of imported, free draining Granular “B” material with soil unit weights and earth pressure coefficients provided in Section 5.4.

5.6 Excavation, Groundwater Control and Temporary Shoring

Excavation for this project will involve the excavation for the spread footings or installation of piles and pile caps, and the construction of abutment walls and retaining walls. The depth of excavation is expected to be 2 to 3 m deep below present grade.

Excavation to depths of up to 3 m should not present any special difficulties using heavy excavation equipment, provided it is constructed in accordance with OPSS 501, 517, 518, 539, 577 and 902, and OPSD-803.010 and 3121.150. However, the buried utilities alongside the roadways will likely be in conflict with the excavation. Excavation and protection procedures shall conform to OPSS 539 and should be reviewed with the utility companies or authorities prior to construction.

It should be pointed out that if the founding soil is disturbed, excessive settlements could occur after structural loads are applied. Therefore, a minimum 50 mm thick lean concrete working mat

should be placed immediately after excavation and subgrade preparation for footings to protect the integrity of the bearing surface and to facilitate placement of reinforcing steel. All foundation excavations, bearing surfaces, and placement of lean concrete mat should be inspected and approved by the geotechnical engineer.

Groundwater is not expected to be encountered within the anticipated depth of excavation of up to 3 m. Surface water and seepage water (perched within the upper fill materials) that inadvertently enter the excavation can be handled by pumping from filtered sump pits.

All excavation must be carried out in compliance with the requirements of the Occupational Health and Safety Act (OHSA). For this purpose, the loose fill and the upper compact silty sand and silt materials encountered at this site are classified as Type 3 soils. Saturated cohesionless soils (if encountered) are classified as Type 4 soils.

Excavation within the Type 3 soils shall be cut to no steeper than 1H : 1V throughout. Side slopes of 3H : 1V or flatter shall be used for excavation within Type 4 soils.

Temporary support may be required to facilitate the underpass construction and to maintain access for construction and local traffic, and emergency vehicles. The staging of different phases of this work should be examined to determine if roadway protection is required. Roadway protection is generally a contractor design/build item in accordance with OPSS 539, SP105S19 and current MTO practices.

The use of temporary shoring support, such as soldier piles and lagging or sheet piles, for the utilities or existing embankments may be required depending the final layout of the proposed structure. Geotechnical parameters for the design of temporary support structures are provided in Sections 5.2 and 5.4. In addition, a unit weight of 21 kN/m^3 and an internal friction angle (ϕ) of 30° for the compact silty sand to silt deposit can be adopted for design.

5.7 Frost Protection

This project is located in the London Operations District. The design frost penetration depth for this project is 1.2 m in accordance with OPSD 3090.101. The pile caps and spread footings should be provided with at least 1.2 m of soil cover for adequate frost protection. Alternatively, frost protection can be provided by equivalent thermal insulation.

5.8 Ramp/Embankment Construction

The new/relocated ramps resulted from the interchange improvements will be in the order of 5 to 6 m high. Based on the findings of the field investigation, the previous borehole information and the performance of the existing embankments, no foundation stability or settlement problems are anticipated for embankment slope of between 2H:1V.

For the construction of the embankment, the surficial topsoil and any deleterious materials should be stripped or excavated prior to placing fill materials. The embankment widening should then be constructed as per OPSD-202.010, 202.030 and 208.010, with emphasis on adequate benching of the subgrade for receiving the embankment fill. The fill to be used for embankment construction can either be site or imported silt or clay soils or granular materials, but granular materials are preferred for compaction and drainage.

Backfill adjacent to the structure should be carried out in conformance with OPSS 902 and OPSD-3121.150, and the fill should be placed and compacted in accordance with OPSS 501 and SSP105S10.

The fill placement should begin at the toe of the embankment, in leveled lifts and each lift compacted to at least 98% SPMDD. Benching into the existing embankment slope at 1 m high steps is recommended as per OPSD 208.010.

After stripping, the exposed subgrade should be inspected and approved by the geotechnical engineer. The approved subgrade should then be proof-rolled using a heavy compactor, as directed by the engineer. Unless the excavation is carried out in wet weather conditions, no unusual dewatering is anticipated during stripping and preparation of the subgrade to receive the embankment fills. Where necessary, dewatering can be carried out using gravity drainage and pumping from open filtered sumps in accordance with OPSS 517 and 902, with emphasis on the requirements of OPSS 518.

Measures should be incorporated into the design and staging to ensure that the slope surfaces are protected from surface erosion in accordance with the requirements of OPSS 577. Proper erosion control measures should be implemented both during construction of the embankment fills and permanently. Erosion control during construction should be carried out by installing silt fences. Properly designed erosion control blankets could also be placed on any new embankments and adjacent disturbed embankments after completion of fill placement. A vegetative cover should be established as soon as practical upon completion of fill placement to minimize the chances of surface erosion.

Maximum settlement resulted from the 6 m high embankment fill is estimated to be less than 5 mm.

6.0 STATEMENT OF LIMITATION

A foundation investigation should be carried out during the detail design stage as per MTO standards and to confirm the preliminary recommendations provided in this report.

The Limitations of Report, as Quoted in Appendix D, is an integral part of this report.

We trust that we have completed the assignment within the Terms of Reference for this project. If there are any questions concerning this report, please do not hesitate to contact our office.

Yours truly,
Infrastructure Engineering Group Inc.

Eric Y. Chung, M.Eng., P.Eng.
Designated MTO Contact

Joseph Law, P.Eng.
Project Manager

Tom O'Dwyer, P. Eng.
Quality Review Engineer

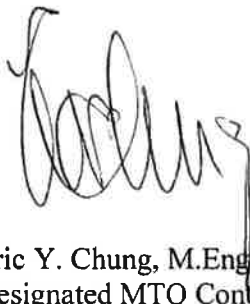
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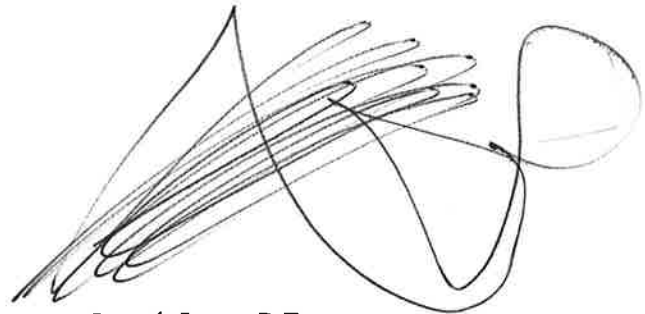
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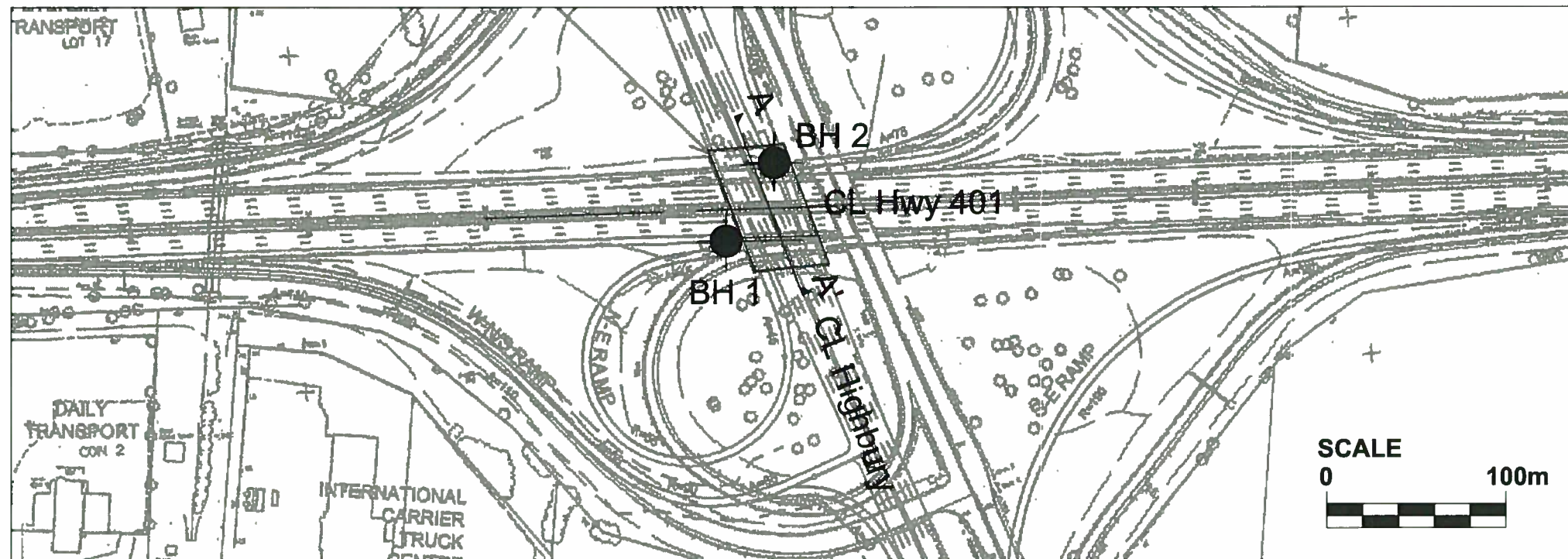


Ministry of Transportation Ontario
G.W.P. 3032-11-00
Proposed Highway 401 Underpass at Highbury Avenue
Agreement # 3011-E-0019
MTO GEOCREs No. 40I14-148

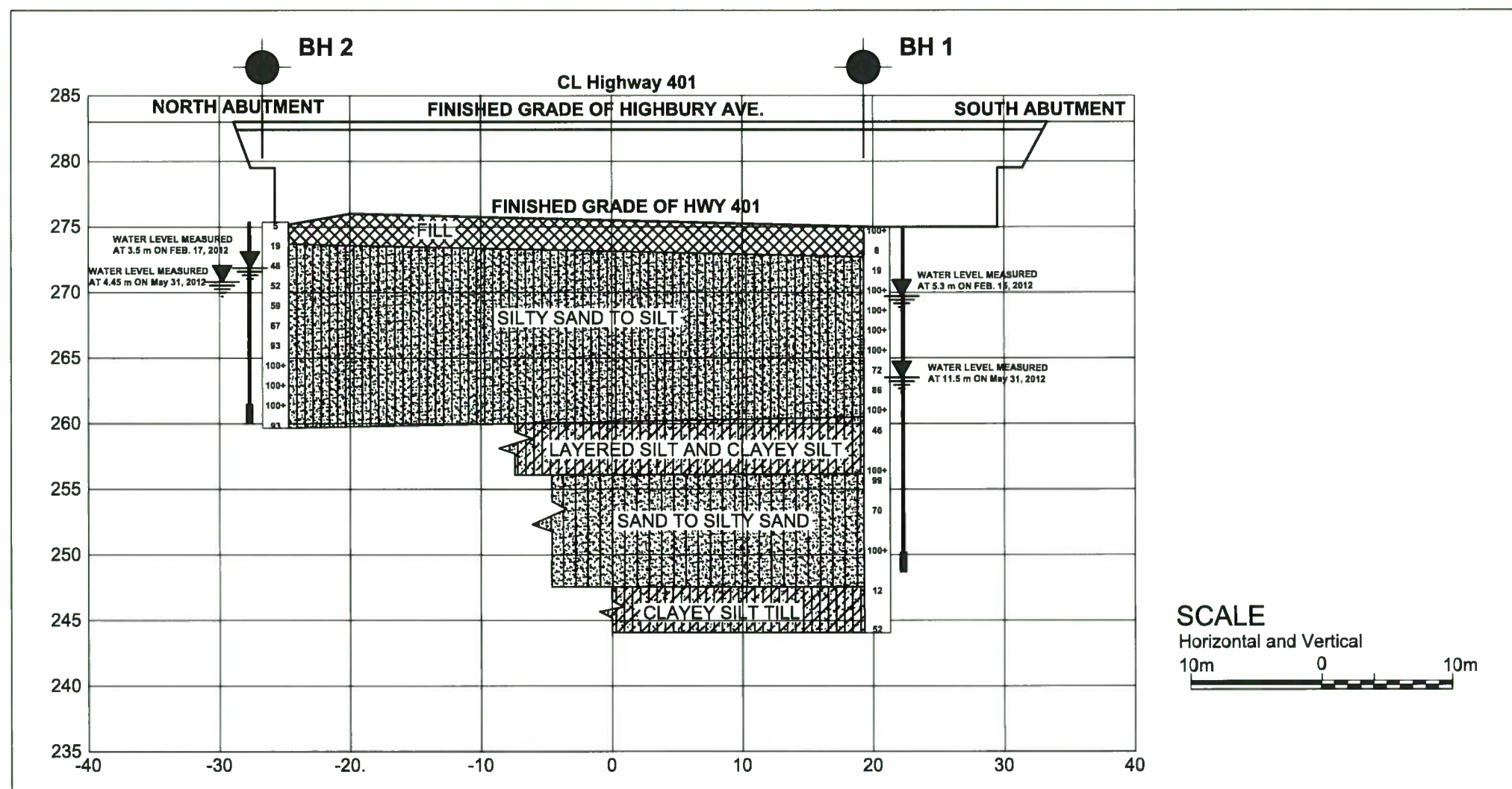
12-1-IEG1-Highbury
Final Report
Drawing 1
July 23, 2012

Drawing 1

Borehole Locations and Soil Strata



BOREHOLE LOCATION PLAN



SECTION A-A'

- NOTES
1. THE COMPLETE FOUNDATION INVESTIGATION AND DESIGN REPORT FOR THIS PROJECT AND OTHER RELATED DOCUMENTS MAY BE EXAMINED AT THE ENGINEERING MATERIALS OFFICE, DOWNSVIEW. INFORMATION CONTAINED IN THIS REPORT AND RELATED DOCUMENTS ARE SPECIFICALLY EXCLUDED IN ACCORDANCE WITH THE CONDITIONS OF SECTION GC2.01 OF OPS GEN. COND.
 2. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES AND BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.
 3. SUBGRADE ELEVATION OF THE EXISTING FOOTING NOT KNOWN AND IS ESTIMATED TO BE AT A MINIMUM OF 1.2m BELOW THE FINISHED GRADE.
 4. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

BOREHOLE NO.	ELEVATION	MTM-11 CO-ORDINATES	
		NORTH	EAST
1	275.04	4755997	412530
2	275.37	4756050	412547

MTO GEORES No. 4014-148	
HWY No.	HWY 401
SUBM'D J.L.	CHECKED E.C.
DRAWN J.L.	CHECKED J.L.

DATE	BY	DESCRIPTION
16/07/12	J.L.	Final
02/06/12	J.L.	Draft

DIST	LONDON
SITE	Not Known
DWG	1

METRIC

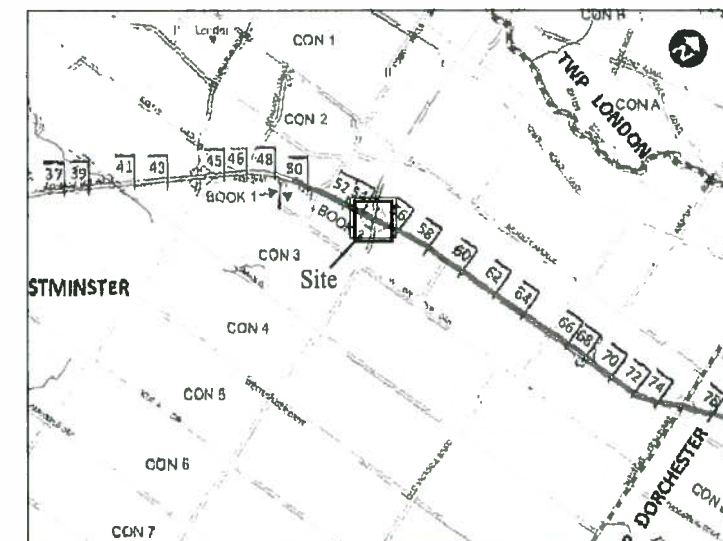
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No xxxx-xxxx
WP NoGWP 3032-11-00

Highbury Avenue
Highway 401 Underpass
BOREHOLE LOCATION PLAN & PROFILE

I.E. Infrastructure Engineering Group Inc.
Pavement & Construction Materials Consulting Engineers
GTA • Kitchener • London • Windsor

KEYPLAN NTS



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊙ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation
- Standpipe

APPENDIX A

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Soils Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

The benchmark and elevations mentioned in this report were obtained strictly for use in the geotechnical design of the project and by this office only, and should not be used by any other parties for any other purposes.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Infrastructure Engineering Group Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

This report does not reflect the environmental issues or concerns unless otherwise stated in the report.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, IEG recommends that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Ministry of Transportation Ontario
G.W.P. 3032-11-00
Proposed Highway 401 Underpass at Highbury Avenue
Agreement # 3011-E-0019
MTO GEOCRES No. 40I14-148

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Appendix A

MTO GEOCRES REPORT NO. 40I14-63

#59-F-15

W.P.# 31-55

Hwy. # 401

CROSSING

HIGHBURY AVE. EXT.

WESTMINSTER TWP.

FOUNDATION REPORT

on

New Bridge at Highway #401 and
Highbury Ave. Extension (Line 'A')
Crossing in Westminster Township.

Profile No: F-3775-2

Plan No: F-3775-1

Chainage: Station 223+75.

Distribution:

Mr. A. M. Toye, Bridge Engineer.	(2)
Mr. H. A. Tregaskes, Construction Engineer.	(1)
Mr. D. G. Ramsay, Design Engineer.	(1)
Mr. H. Orlando, Project Design Engr., London.	(1)
Mr. W. L. Fraser, District Engineer, London.	(1)
Mr. J. Roy, Regional Soils Engr., London.	(1)
Mr. A. Watt, Ont. Water Resources Commission.	(1)
Foundation Section.	(1)
Gen. Files.	(1)

W.P. 31-55

W.J. F-59-15.

INTRODUCTION:

Reported herein are the results of a foundation investigation recently completed at the above noted site. A brief description of the field work carried out and our recommendations pertaining to footing design based upon the factual data obtained are given in the following paragraphs:-

SITE LOCATION AND DESCRIPTION:

The structure is located at the intersection of Highway #401 and the proposed extension of Highbury Avenue (Line 'A') in Westminster Township (see Profile No. F-3529-12, chainage - 223+75).

The site is located in the physiographic region known as the Carodoc Sand Plains which is a well drained formation with gently rolling to flat topography.

INVESTIGATION PROCEDURE:

Field work consisted of two detailed sampled borings and four dynamic cone penetration tests located on either side of the centre line of Hwy. 401 as shown on enclosed plan numbered F 59-15A.

Samples were recovered at a maximum depth interval of five feet in each boring. The granular nature of the subsoil precluded the taking of relatively undisturbed samples; disturbed samples were recovered using a 2-inch diameter split barrelled sampling spoon. The driving energy used in driving the sampler conformed to the requirements of the empirical Standard Penetration Test, and 'N' values were recorded and have been presented on the data sheets included with this report.

INVESTIGATION PROCEDURE: (cont'd.) ...

Borings were terminated in the underlying sand stratum at a depth of 26 1/2 feet below existing ground surface.

Elevation of the top of the borings and the chainage locations have been noted on the profiles appended to this report.

SOIL TYPES ENCOUNTERED:

In order of stratigraphic succession from existing ground surface to the depth investigated by the borings, the following subsoil strata were encountered:-

(1) Fill: A surface layer of brown silty clay, containing some gravel sizes was intersected in each of the two borings. This material is believed to be embankment fill placed during grading operations during construction of Hwy. 401. The thickness of this fill layer varied from 4 1/2 feet at the location of Hole No. 1, to 3 feet at the location of Hole No. 2.

(2) Dense Brown Fine to Medium Sand: Underlying the surface veneer of recently placed fill material, a deep deposit of dense fine to medium dry sand was intersected at each borehole location. Standard Penetration test resistance values (N) are in excess of 25 blows/ft. throughout this stratum. Ground water was not encountered in either of the two borings during the period of the field work.

Representative strength and in-situ density values for this sand stratum are as follows:-

'N' value - 30 blows/ft.

Unit Weight in-situ - 115 p.c.f.

Angle of Shearing Resistance = $\phi = 36^{\circ}$.

cont'd. /3 ...

APPENDIX I.

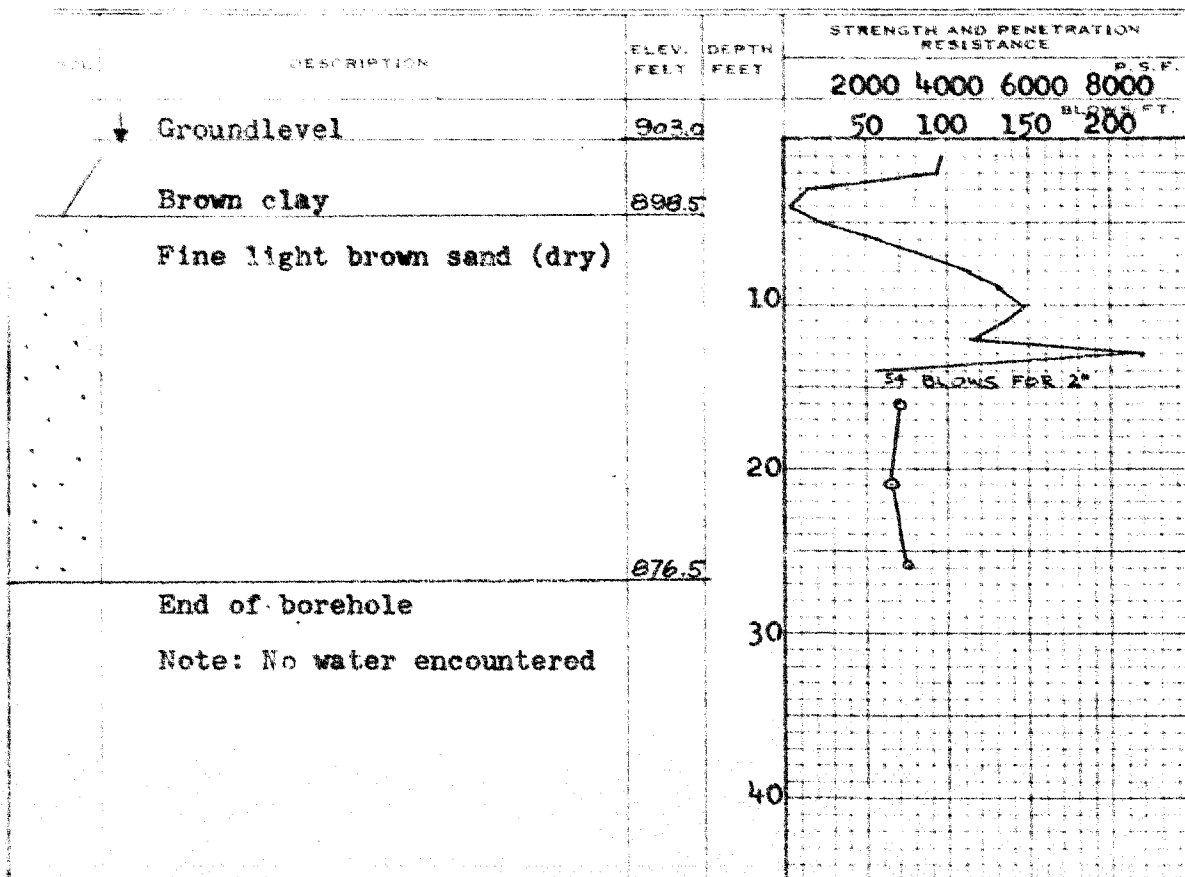
DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 31-55 BORE HOLE NO. 1
 JOB F-59-15 STATION 223+86 (70' R)
 DATUM Geodetic COMPILED BY B.K.
 BORING DATE Feb 25/59 CHECKED BY V.K.

2" DIA. SPLIT TUBE _____
 2" SHELBY TUBE _____
 2" SPLIT TUBE _____
 2" DIA. CONE _____
 2" SHELBY _____
 CASING _____

LEGEND

1/2 UNCONFINED COMPRESSION (Q_u) _____
 VANE TEST (C) AND SENSITIVITY (S) _____
 NATURAL MOISTURE AND LIQUIDITY INDEX _____
 LIQUID LIMIT _____
 PLASTIC LIMIT _____



CONSISTENCY	SAMPLE	NATURAL UNIT WT. (G.C.F.)
MOIST. CONTENT - % DRY WT.		
	TW1	
	SS2	
	SS3	
	SS4	
	SS5	
	SS6	

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. _____ BORE HOLE NO. 2

JOB F-59-15 STATION 224/18 (70' R.)

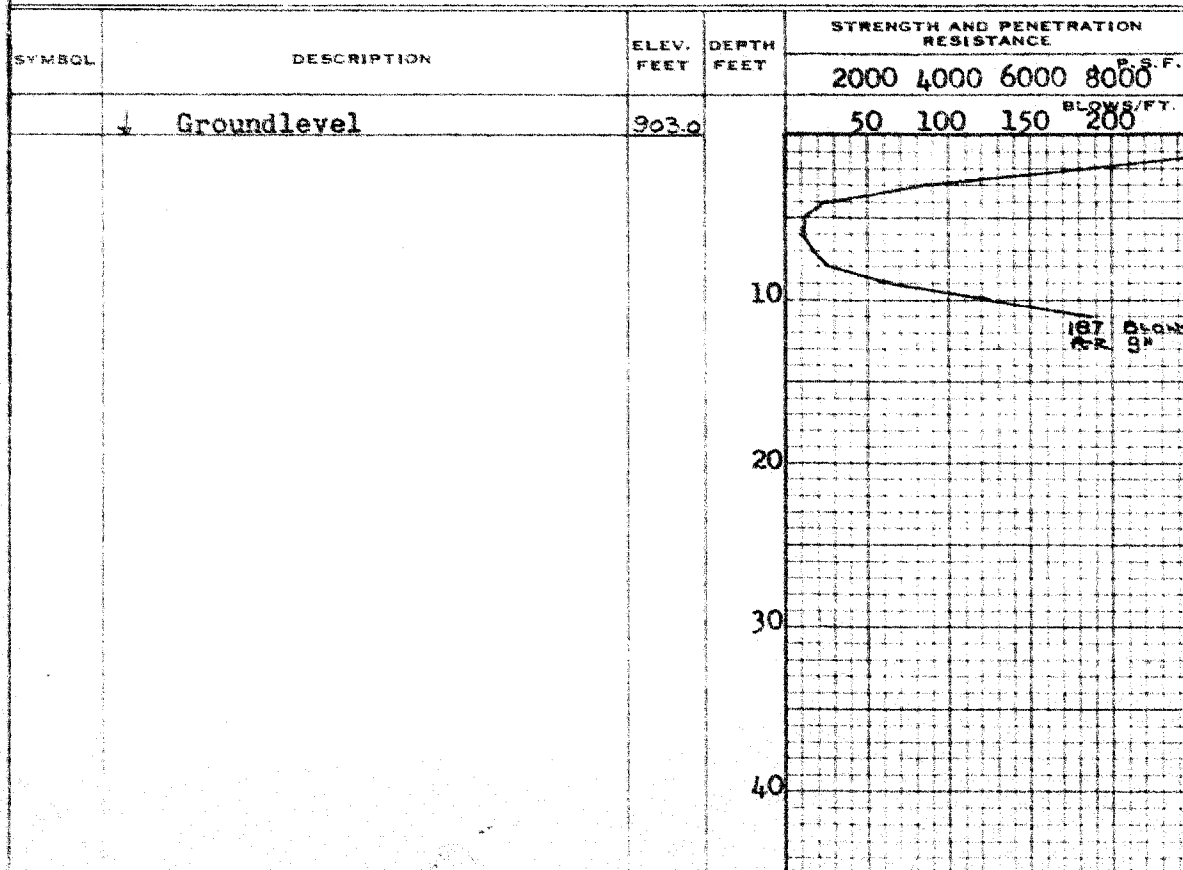
DATUM Geodetic COMPILED BY B.K.

BORING DATE Feb. 26/59. CHECKED BY V.K.

2" DIA. SPLIT TUBE -----
2" SHE. BY TUBE -----
2" SPLIT TUBE -----
2" DIA. CONE -----
2" SHELBY -----
CASING -----

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) _____ O
VANE TEST (C) AND SENSITIVITY (S) _____ + S
NATURAL MOISTURE AND _____ LI
LIQUIDITY INDEX _____ X
LIQUID LIMIT _____
PLASTIC LIMIT _____

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO

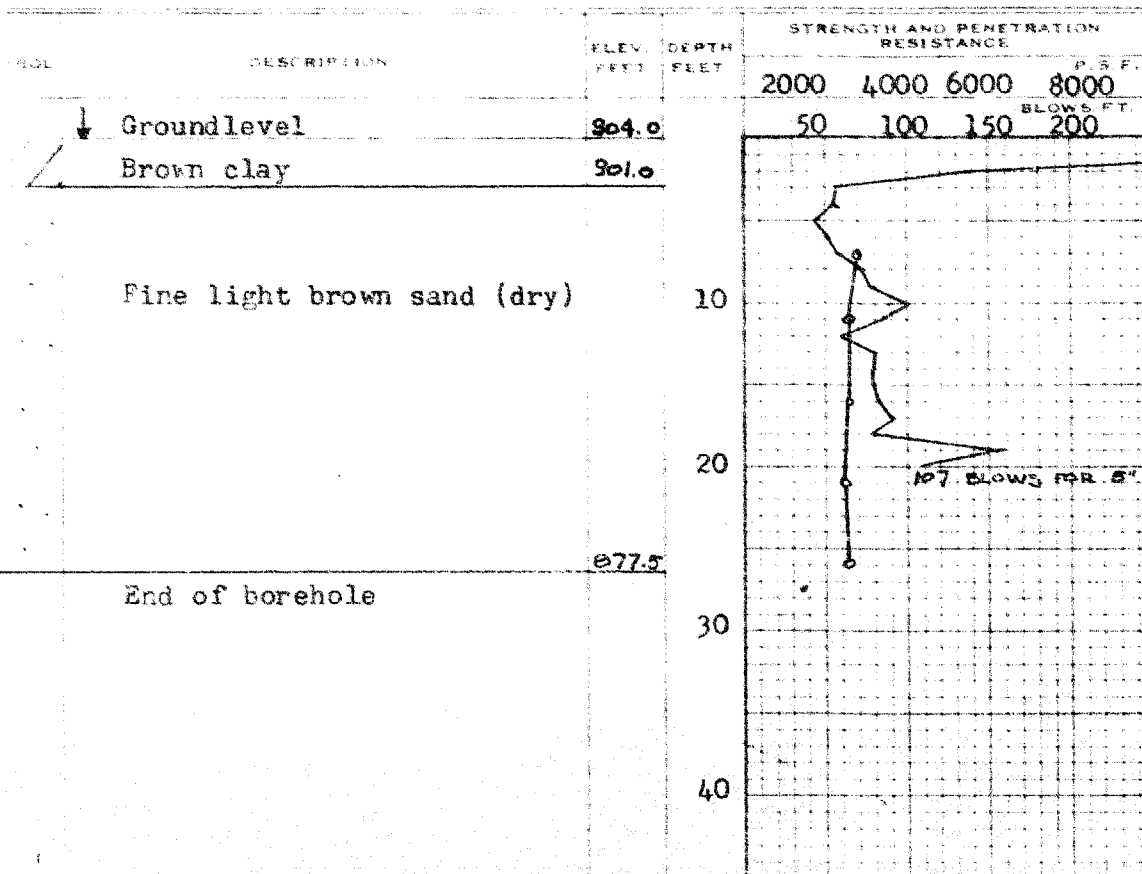
MATERIALS AND RESEARCH SECTION

W.P. - - - - - BORE HOLE NO. 3
 JOB F-59-15 STATION 223/63 (60' L)
 DATUM Geodetic COMPILED BY B.K.
 BORING DATE Feb. 26/59. CHECKED BY V.K.

2" DIA. SPLIT TUBE
 2" SHELBY TUBE
 2" SPLIT TUBE
 2" DIA. CONC.
 2" SHELBY
 CASING

LEGEND

1/2 UNCONFINED COMPRESSION (Q_u) O
 VANE TEST (C) AND SENSITIVITY (S) +
 NATURAL MOISTURE AND LIQUIDITY INDEX LI
 LIQUID LIMIT X
 PLASTIC LIMIT



CONSISTENCY			NATURAL
MOIST. CONTENT- % DRY WT.			SAMPLE UNIT WT. P.C.F.
10	20	30	
			SS1
			SS2
			SS3
			SS4
			SS5

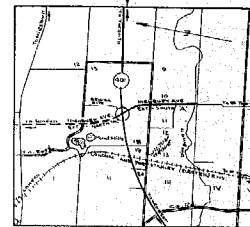
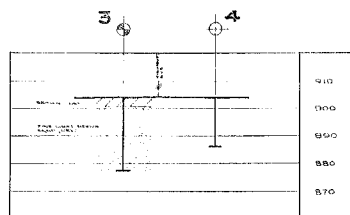
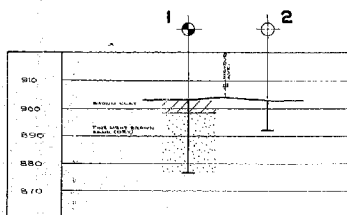
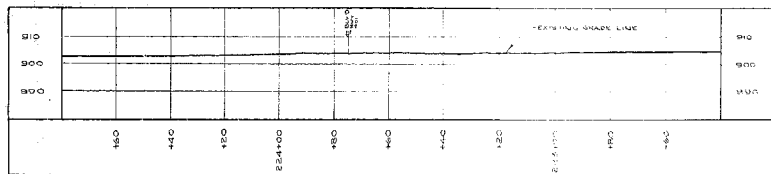
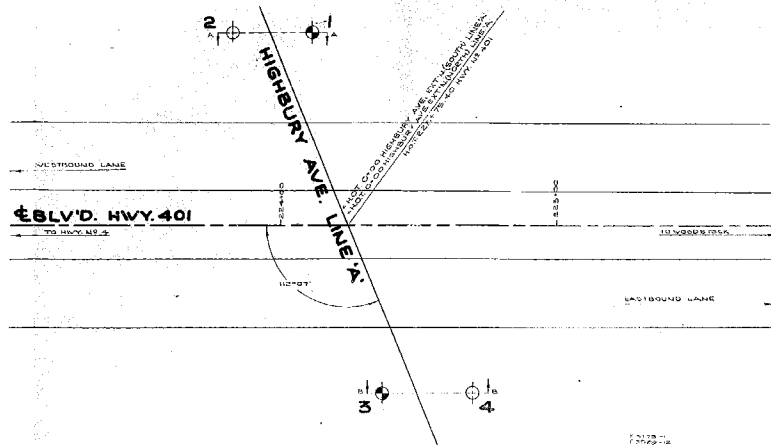
MATERIALS AND RESEARCH SECTION

2" DIA. SPLIT TUBE
2" SHELBY TUBE
2" SPLIT TUBE
2" DIA. CONE
2" SHELBY
CASING

1/2 UNCONFINED COMPRESSION (Qu) ---	O
VANE TEST (C) AND SENSITIVITY (S) ---	+S
NATURAL MOISTURE AND	
LIQUIDITY INDEX ---	L
LIQUID LIMIT ---	X
PLASTIC LIMIT ---	

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
					P.S.F.
	↓ Groundlevel	904.0			
				<p>BLOWS/FT.</p> <p>107 BLOWS PER FT.</p>	

[illegible]



SCALE: 1 in. = 1 mi.

LEGEND

- BORE HOLE
- PENETRATION HOLE
- BORE & PENETRATING HOLE

HOLE NO.	ELEVATION	STATION	DISTANCE FROM E.
1	902.0	223+18.4	75' RT.
2	905.0	224+10	70' RT.
3	904.0	223+63	60' LT.
4	904.0	223+70	60' LT.

NOTE
THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BORE HOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

**HIGHBURY AVE. EXT'N.
PROPOSED CROSSING**

SHOWING POSITIONS & ELEVATIONS OF HOLES

HWY. 401 DISTRICT 12 COUNTY MIDDLESEX
TOWNSHIP WESTANUSHTON LOT 16 CON. II
LOCATION APP. 5 MI. S. OF LONDON
ENGINEER: J. H. HARRISON | CHECKED BY: J. H. HARRISON
DATE: 5 JUNE 1959 | APPROVED BY: J. H. HARRISON
SCALE: 1 in. = 30 ft. DRAWING NO. **F59-15A**

Ministry of Transportation Ontario
G.W.P. 3032-11-00
Proposed Highway 401 Underpass at Highbury Avenue
Agreement # 3011-E-0019
MTO GEOCREs No. 40I14-148

12-1-IEG1-Highbury
Final Report
Appendix B
July 23, 2012

Appendix B

Explanation of Terms Used in Report

Record of Borehole Logs

Boreholes 1 and 2

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N}

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 1" SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T.W. ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T.W. ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_r	kPa	RESIDUAL SHEAR STRENGTH
τ_c	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_c}$

PHYSICAL PROPERTIES OF SOIL

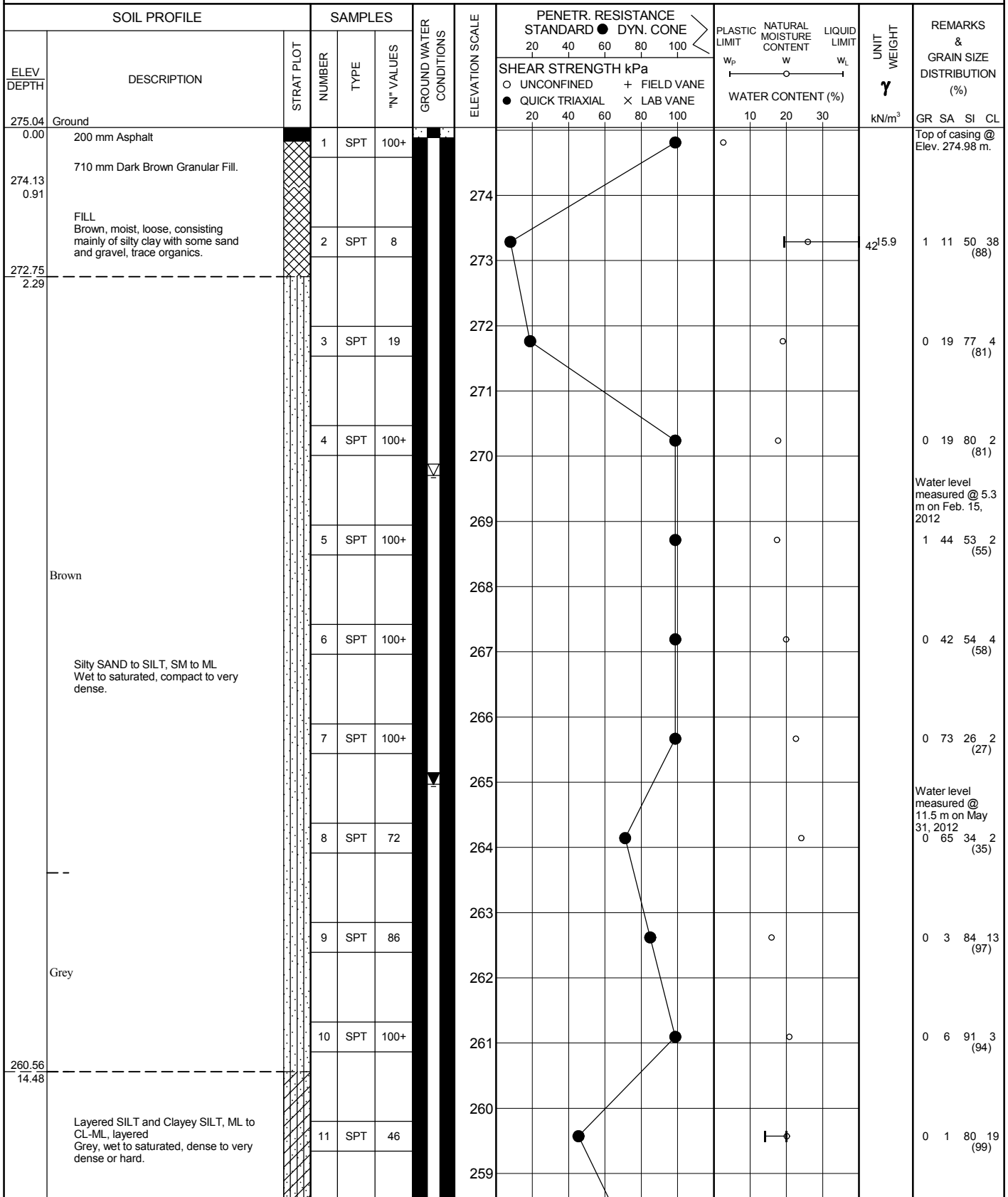
ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1. %	VOID RATIO	e_{min}	1. %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1. %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1. %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kn/m ³	UNIT WEIGHT OF DRY SOIL	i_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{i_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{i_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1. %	VOID RATIO IN LOOSEST STATE	j	kn/m ³	SEEPAGE FORCE
γ'	kn/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No 1

1 OF 2

METRIC

W.P. GWP 3032-11-00 LOCATION Highbury Ave. South Side Northing - 4755997, Easting - 412530 ORIGINATED BY JL
 DIST London HWY 401 BOREHOLE TYPE 110 mm H/S and Wash Boring COMPILED BY JL
 DATUM Geodetic DATE 14.2.12 - 16.2.12 CHECKED BY EC



Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity

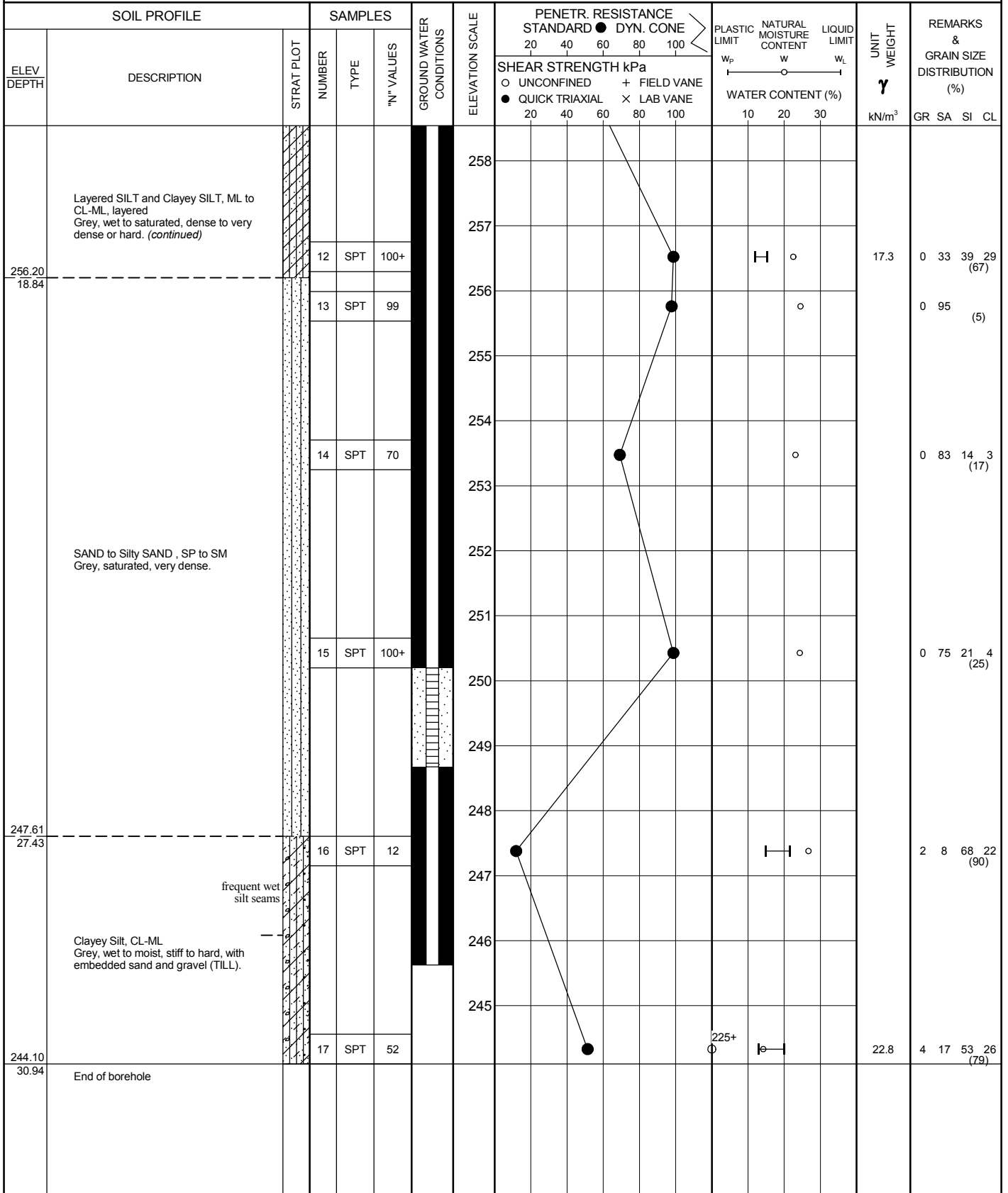
○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

RECORD OF BOREHOLE No 1

2 OF 2

METRIC

W.P. GWP 3032-11-00 LOCATION Highbury Ave. South Side Northing - 4755997, Easting - 412530 ORIGINATED BY JL
 DIST London HWY 401 BOREHOLE TYPE 110 mm H/S and Wash Boring COMPILED BY JL
 DATUM Geodetic DATE 14.2.12 - 16.2.12 CHECKED BY EC



JOE MTO 12-1-IEG1 Highbury Foundations.GPJ ONTARIO.MOT.GDT 23/7/12

+ 3, × 3: Numbers refer to Sensitivity

○ 150 UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

METRIC

[illegible]

○ ¹⁵⁰ UNCONFINED SHEAR STRENGTH INFERRED FROM POCKET PENETROMETER READINGS

Appendix C

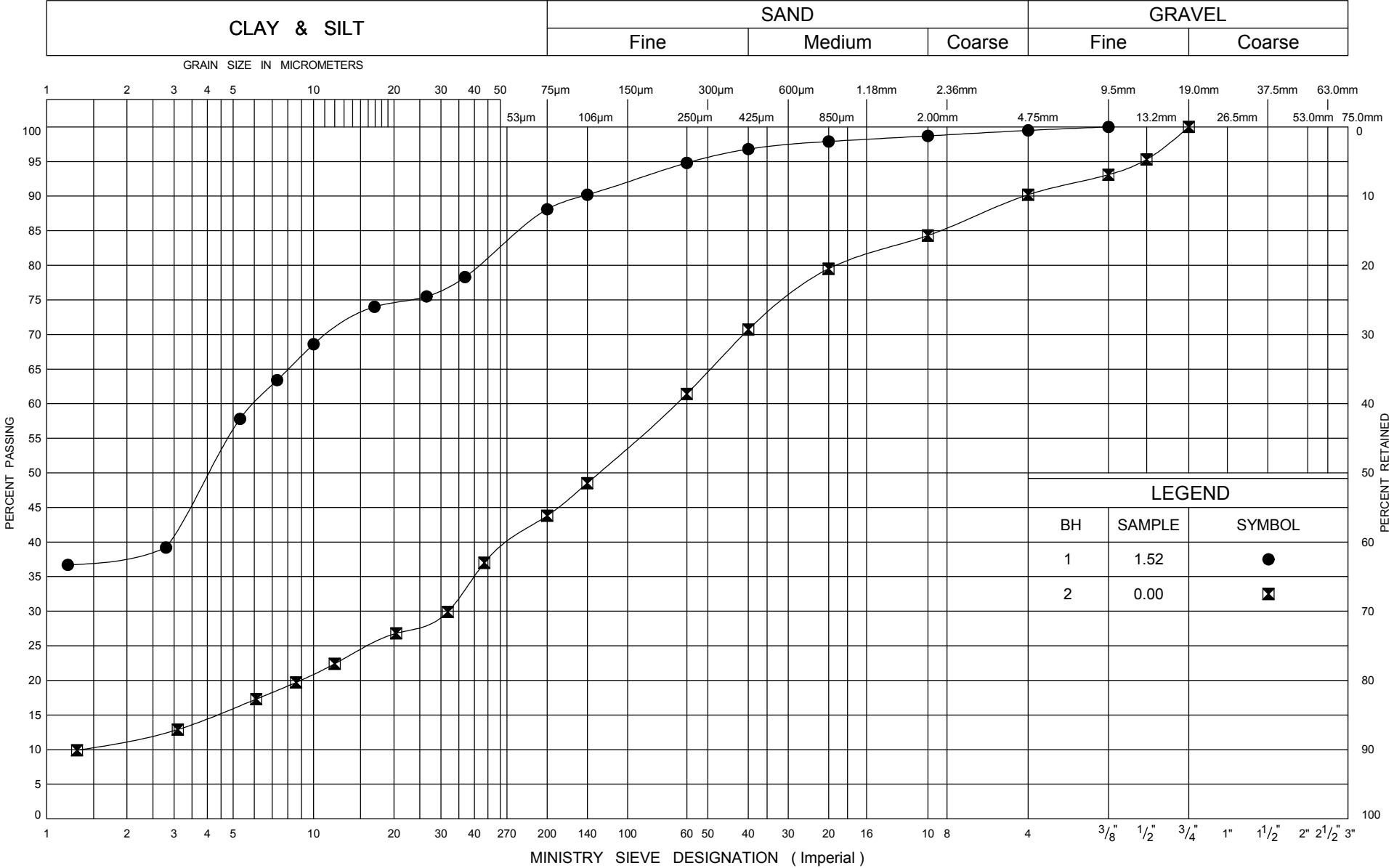
Grain Size Distribution

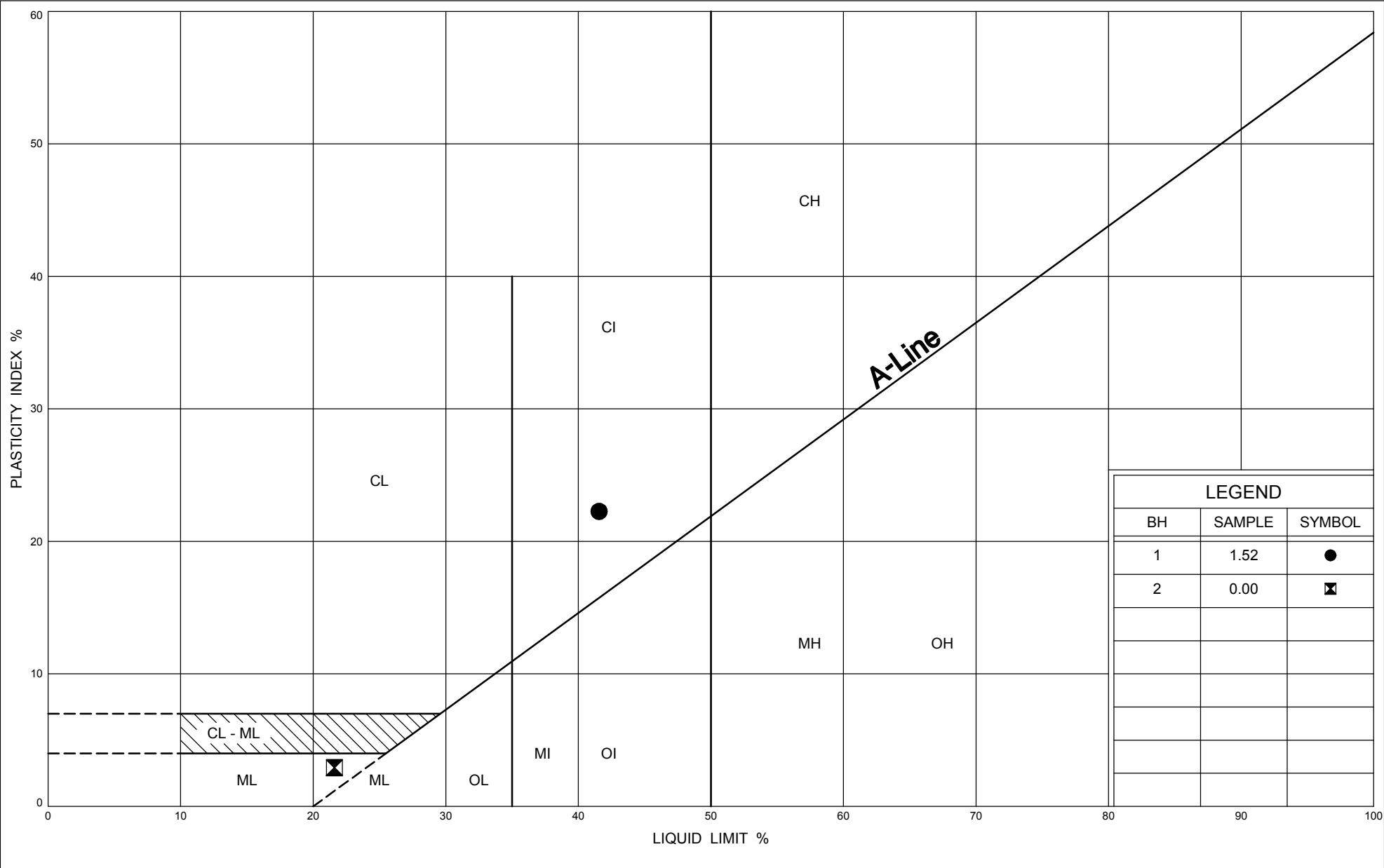
Figures 1, 3, 4, 5, 6, 8 & 9

Plasticity chart

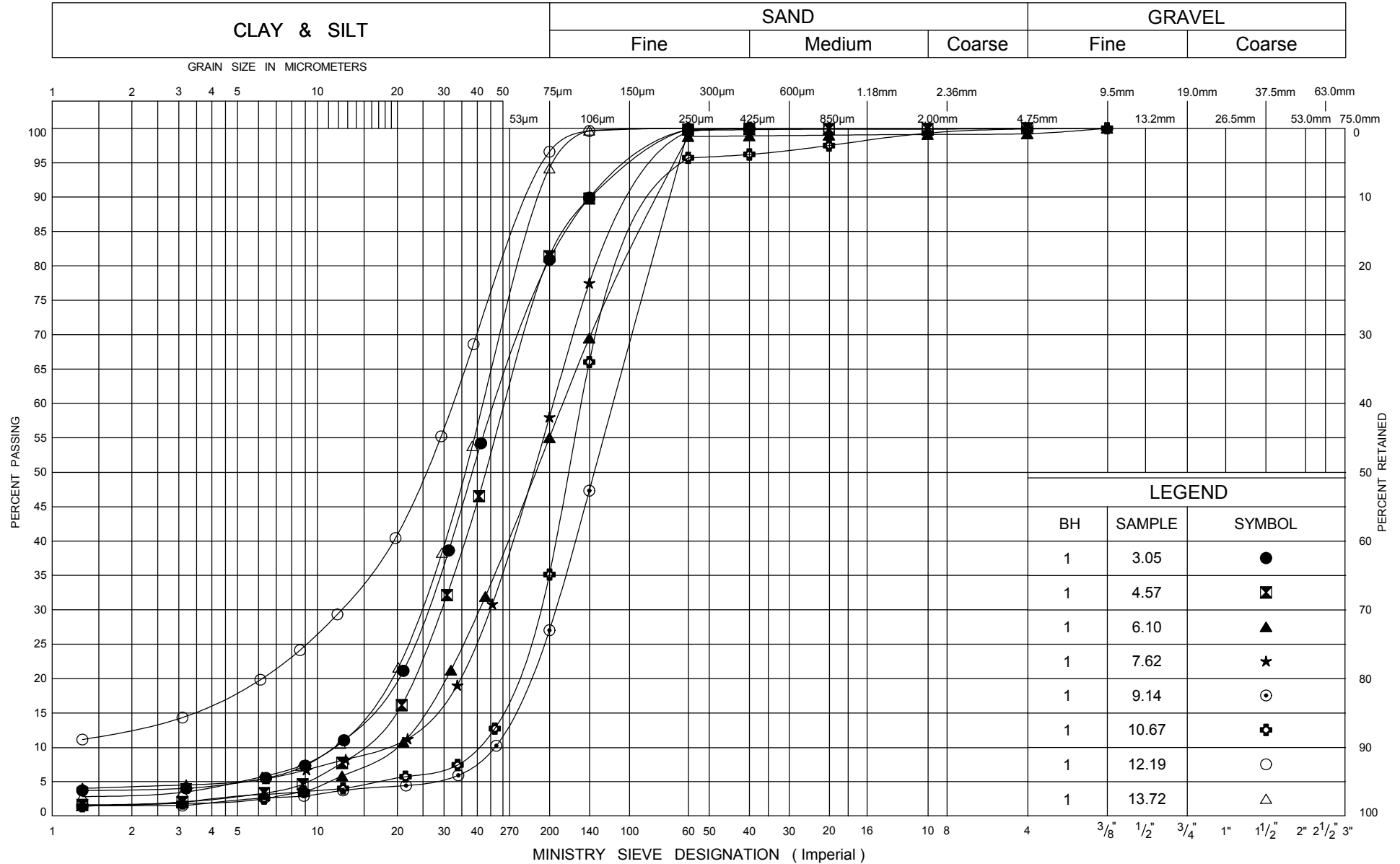
Figures 2, 7 and 10

UNIFIED SOIL CLASSIFICATION SYSTEM





UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

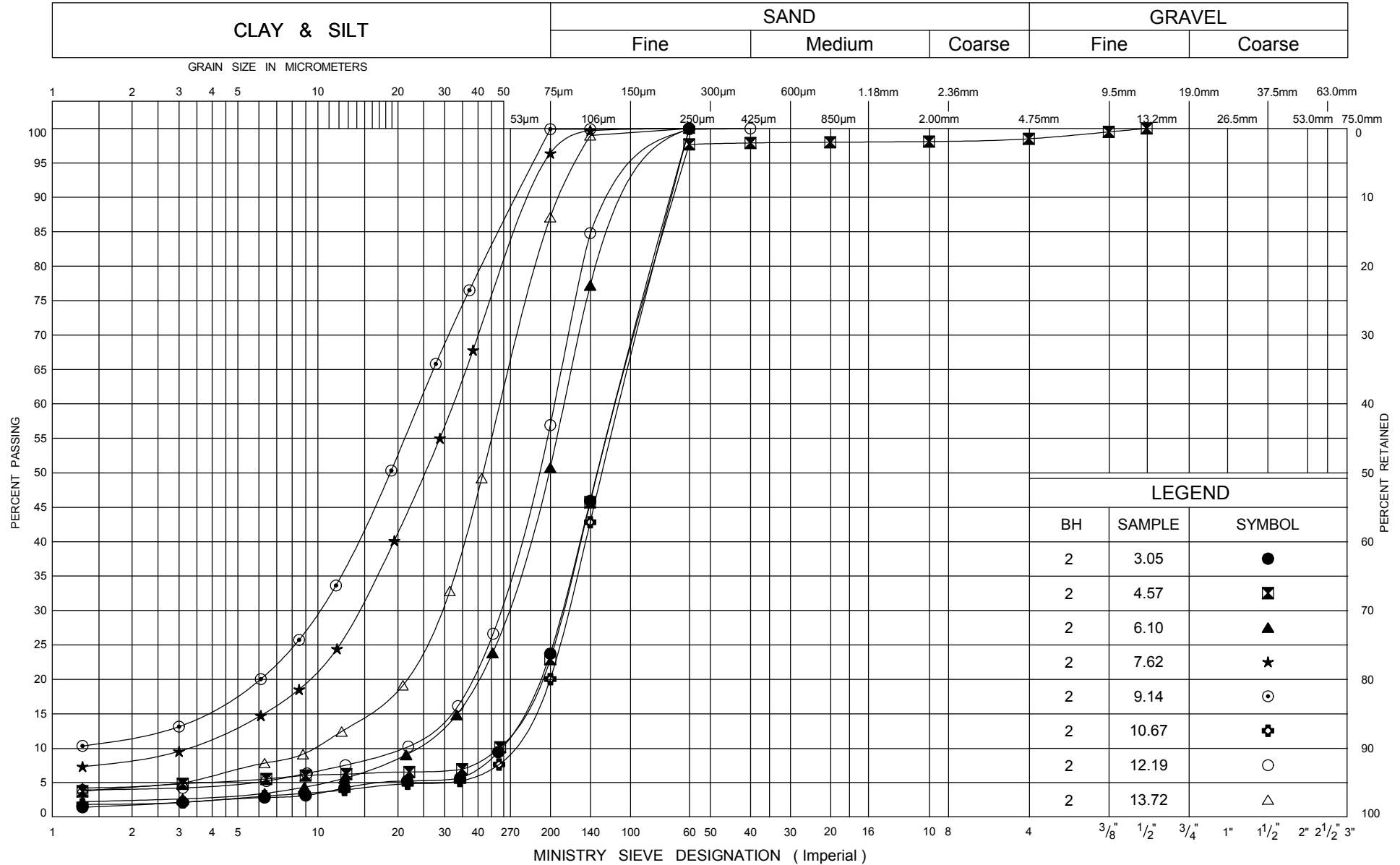
GRAIN SIZE DISTRIBUTION
Upper Silty Sand to Silt, SM to ML

FIG No 3

GWP 3032-11-00

Highbury Interchange Reconfiguration

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

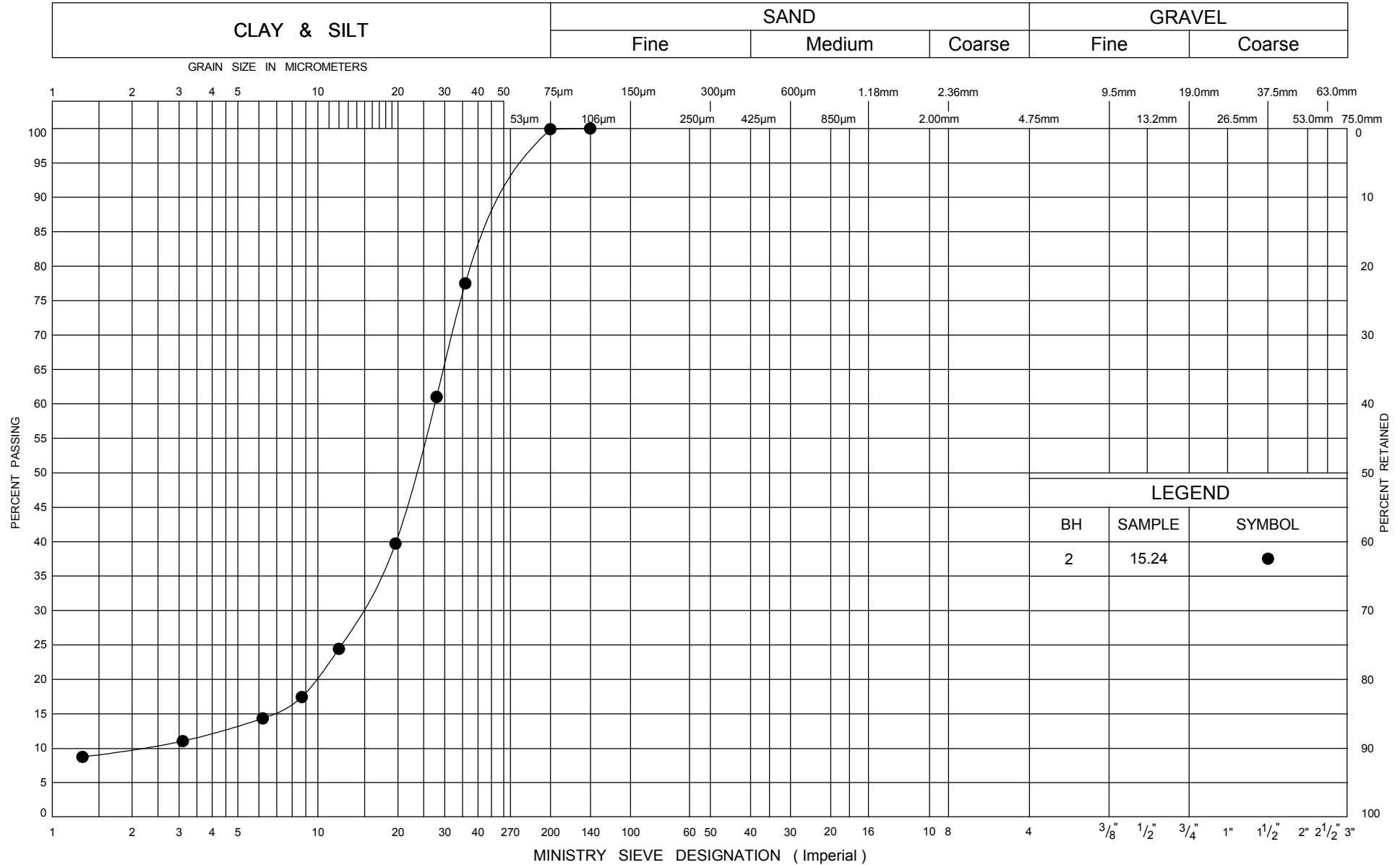
GRAIN SIZE DISTRIBUTION
Upper Silty Sand to Silt, SM to ML

FIG No 4

GWP 3032-11-00

Highbury Interchange Reconfiguration

UNIFIED SOIL CLASSIFICATION SYSTEM



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Transportation

GRAIN SIZE DISTRIBUTION

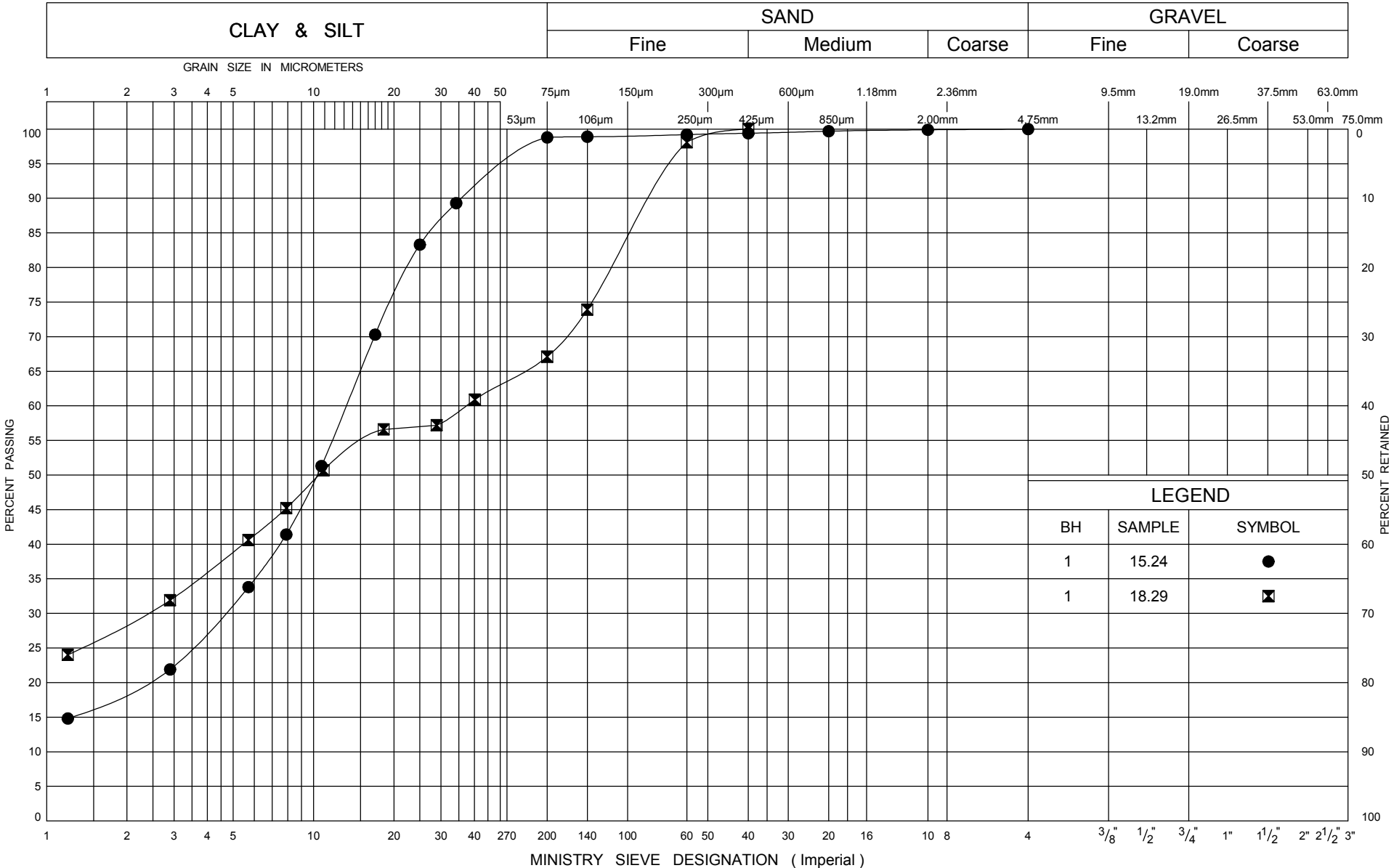
Upper Silty Sand to Silt, SM to ML

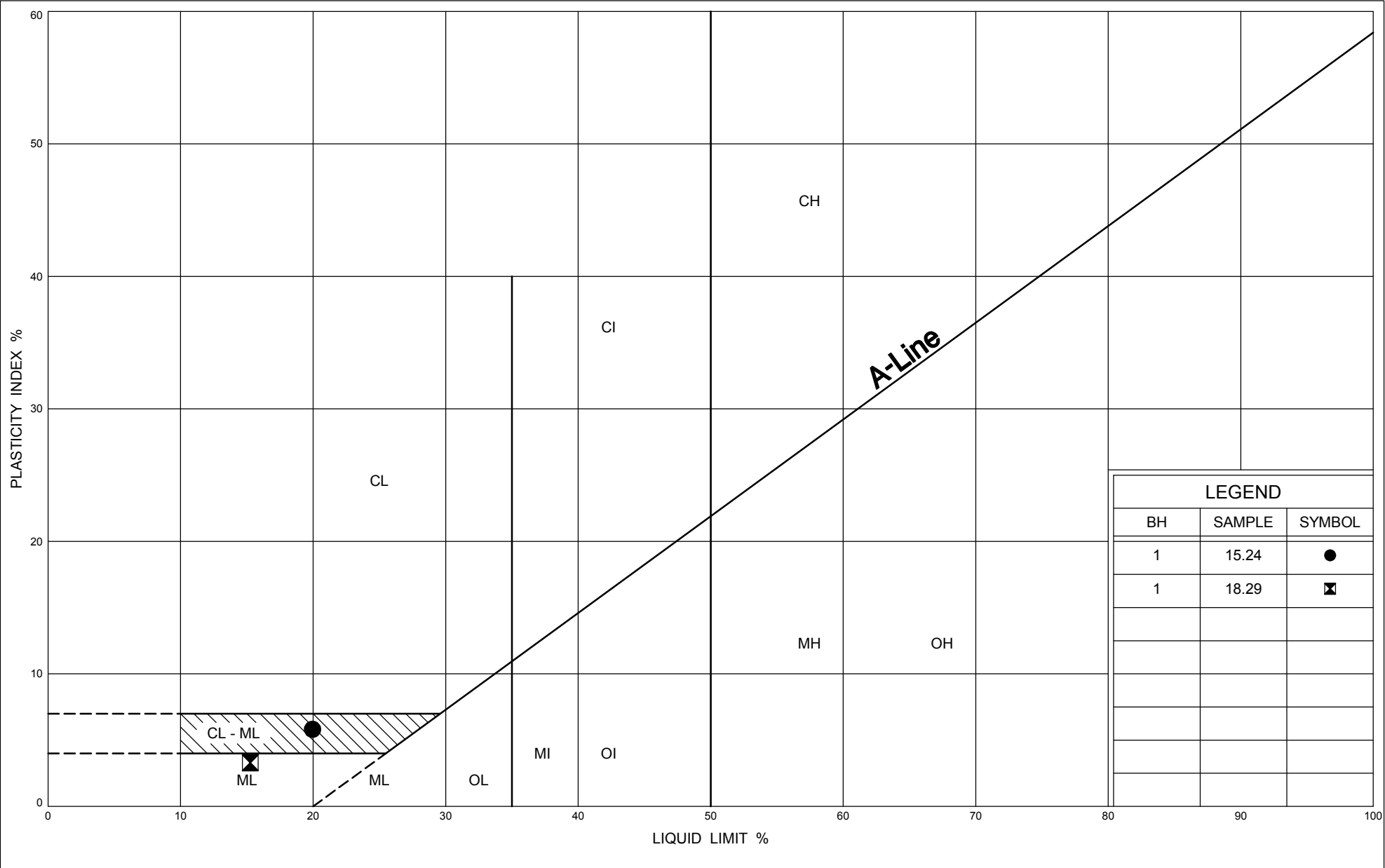
FIG No 5

GWP 3032-11-00

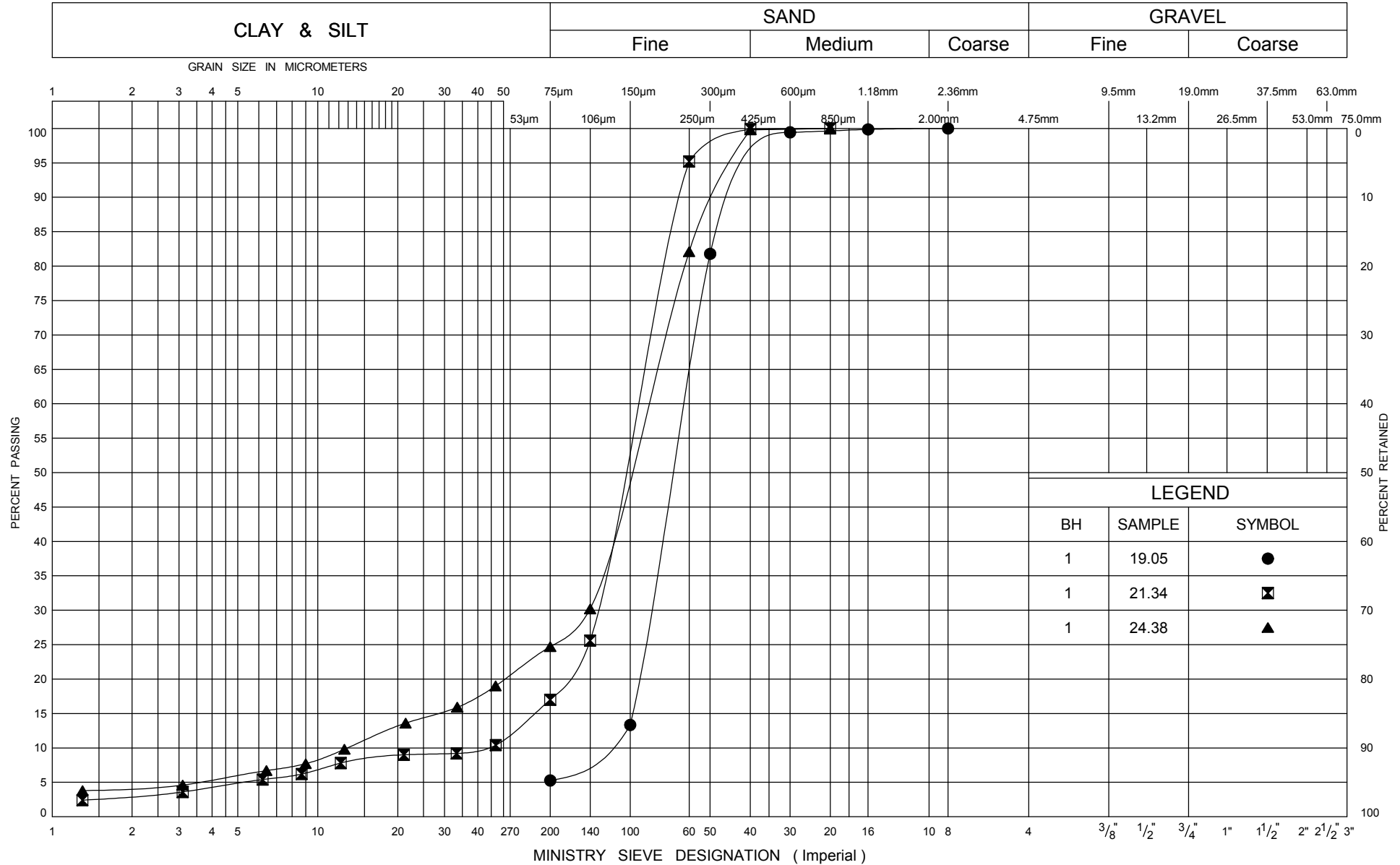
Highbury Interchange Reconfiguration

UNIFIED SOIL CLASSIFICATION SYSTEM





UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

GRAIN SIZE DISTRIBUTION

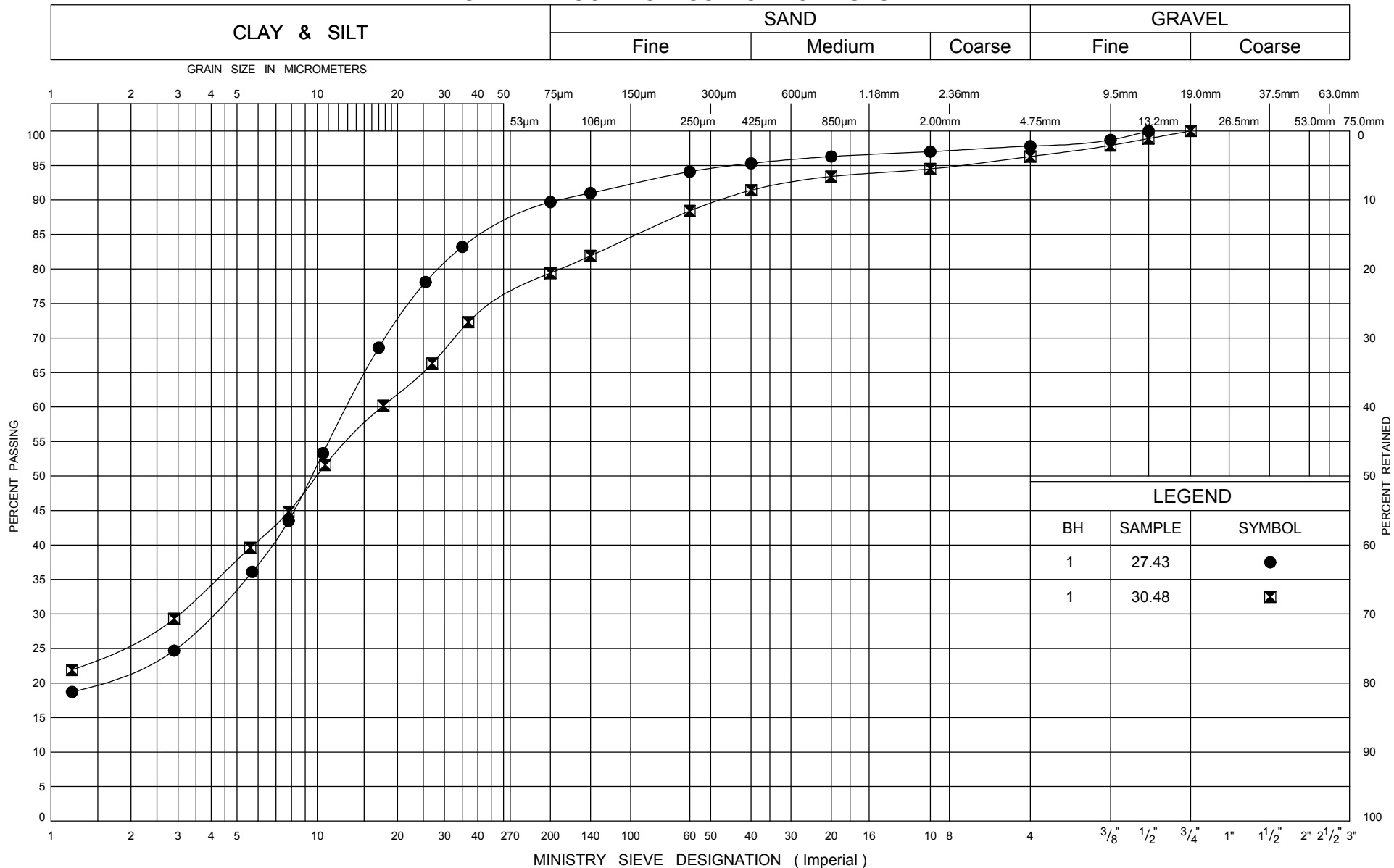
Lower Sand to Silty Sand, SP to SM

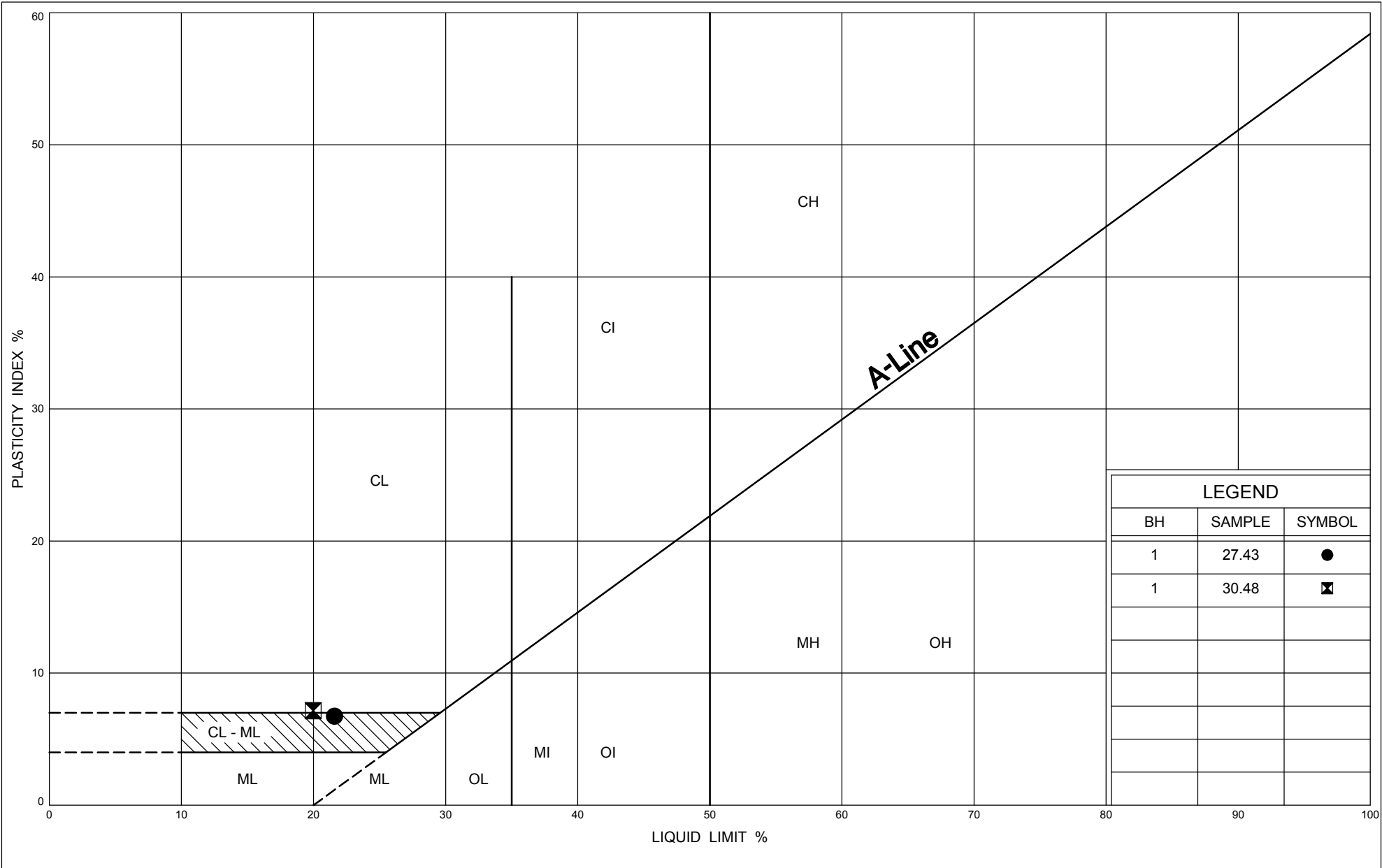
FIG No 8

GWP 3032-11-00

Highbury Interchange Reconfiguration

UNIFIED SOIL CLASSIFICATION SYSTEM





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Agreement # 3011-E-0019
MTO GEOCREs No. 40I14-148

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Appendix D

Limitations of Report

APPENDIX D

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Soils Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in the testholes.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusion as to how the subsurface conditions may affect their work.

The benchmark and elevations mentioned in this report were obtained strictly for use in the geotechnical design of the project and by this office only, and should not be used by any other parties for any other purposes.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Infrastructure Engineering Group Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

This report does not reflect the environmental issues or concerns unless otherwise stated in the report.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, IEG recommends that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

Ministry of Transportation Ontario
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Agreement # 3011-E-0019
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Appendix E
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Appendix E

Site Photographs



Aerial view of the existing Highway 401 Interchange @ Highbury Avenue



West Elevation of Highbury Avenue Structure



East Elevation of Highbury Avenue Structure



Highbury Avenue Looking South



Highbury Avenue Looking North