



FINAL REPORT

Foundation Investigation and Design Report

Slope Stability Assessment

Langstaff Road Underpass Southeast Embankment

Highway 400 Widening, Langstaff Road to Major Mackenzie Drive

Vaughan, Ontario

MTO GWP 2836-02-00

Submitted to:

Parsons Inc.

625 Cochrane Drive, Suite 300
Markham, Ontario L3R 9R9

Submitted by:

WSP Golder

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PART A

Foundation Investigation Report
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Highway 400 Widening
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1.0 INTRODUCTION

WSP Golder (formerly Golder Associates Ltd., now a member of WSP Canada Inc.) has been retained by Parsons Inc. (Parsons) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the Highway 400 widening and rehabilitation, extending from 1.3 km south of the Langstaff Road interchange to 1.5 km north of Major Mackenzie Drive (a length of approximately 7.3 km) in the City of Vaughan, Ontario. As part of the Highway widening and rehabilitation program, embankment slope remediation work will be undertaken in the southeast quadrant of the Highway 400/Langstaff Road interchange.

This report summarizes the factual results of field and laboratory work (including field investigation procedures, borehole stratigraphy, and geotechnical laboratory test results) and provides a description of interpreted soil and groundwater conditions in the vicinity of the approach embankment in the southeast quadrant of the Langstaff Road Underpass site.

2.0 SITE DESCRIPTION

The orientation (i.e., north, south, east, and west) stated in the text of this report is referenced to project north and therefore may differ from magnetic north shown on Drawing 1. For the purpose of this report, Langstaff Road is described as oriented in a west-east direction on a slight skew to Highway 400, which generally runs in a north-south direction.

In general, the topography surrounding the Langstaff Road interchange is relatively flat. The existing Langstaff Road embankment side slope and ground surface beyond the toe in the southeast quadrant of the Highway 400/Langstaff Road interchange is landscaped with grass cover and a few limited zones of tree cover. Land use surrounding the area is primarily commercial. There is an existing dry pond located just south of the embankment. A concrete box culvert runs through the embankment (i.e., perpendicular to Langstaff Road) to facilitate drainage along Black Creek. The inlet and outlet locations of the box culvert are near the toe of the slope on both sides of the north and south sides of the Langstaff Road embankment and retaining walls consisting of rows of gabions are present on each side of the culvert. There is also a culvert that runs beneath Highway 400 and connects to Black Creek immediately south of the embankment toe.

The Langstaff Road grade east of the underpass is at about Elevation 213 m to 214 m and the toe of the embankment in the southeast quadrant of the interchange is at about Elevation 205 m to 206 m (i.e., embankment height on the order of about 8 m). The southeast embankment has a side slope inclination of about 2 horizontal to 1 vertical (2H:1V).

The guardrail along the south side of Langstaff Road in this quadrant is located immediately adjacent to the crest of the embankment slope, and some of the guardrail posts are leaning southward (i.e., downslope). Some limited settlement of the south edge of the sidewalk was also observed. There was no other visual evidence of slope instability, gabion wall instability or loss of ground behind the gabion wall in this quadrant at the time of field investigation. The ground surface conditions in the vicinity of the southeast embankment are shown on Photographs 1 to 3 following the text of this report.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface exploration program consisted of three boreholes (designated LS-1, LS-2, and LS-3) advanced in the vicinity of the southeast embankment of the Langstaff Road Underpass. These boreholes were advanced between March 30, 2023, and July 17, 2023, at the approximate locations shown on Drawing 1.

Boreholes LS-1 and LS-2 were advanced through the existing embankment of Langstaff Road on the south side of the roadway (i.e., through the crest of the southeast embankment), and Borehole LS-3 was advanced at the toe of the southeast embankment near Highway 400 grade. The boreholes were advanced using a truck-mounted CME 75 drill rig supplied and operated by 3D Drilling of Whitchurch-Stouffville, Ontario. The boreholes were advanced through the overburden using 159 mm outside diameter hollow stem augers. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outside diameter split spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ATM D1586)¹. The split-spoon samplers used in the investigation limits the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions.

The groundwater conditions were noted in the boreholes during and upon completion of drilling and were backfilled in accordance with Ontario Regulation 903 (Wells, as amended), and the asphalt surface at the locations of Boreholes LS-1 and LS-2 was capped with tamped cold patch asphalt. At the location of Borehole LS-3, a standpipe piezometer was installed to allow monitoring of the groundwater level. The installed piezometer consists of a 50 mm diameter PVC pipe, with a 3.0 m long slotted screen within a filtered sand pack. The borehole and annulus surrounding the piezometer pipe above the filter sand pack was backfilled to near ground surface with bentonite pellets. The standpipe piezometer was left sticking up out of the ground and protected with a monument cover.

The field work was observed by members of WSP Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled, and transported to WSP Golder's Mississauga laboratory where the samples underwent further visual examination. Geotechnical laboratory testing (water content, grain size distribution, and Atterberg limits) was carried out on select soil samples, in accordance with MTO and / or ASTM Standards, as appropriate. In addition, select soil samples were submitted to Bureau Veritas Laboratories of Mississauga, Ontario for analysis of select parameters to assess for the potential corrosion of buried steel and deterioration of concrete.

The as-drilled borehole locations and elevations were surveyed by WSP Golder using a Trimble Geo 7x GPS unit. The locations are referenced to NAD 83(CSRS)v6 MTM Zone 10 coordinates and the ground surface elevations are referenced to CGVD28 Geodetic datum benchmark. The borehole locations, including geographic coordinates, ground surface elevations, and borehole depths are summarized below.

¹ ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Borehole No.	MTM NAD83 Northing (Latitude, °)	MTM NAD83 Easting (Longitude, °)	Ground Surface Elevation (m)	Borehole Depth (m)
LS-1	4,852,156.4 (43.809364)	301,336.7 (-79.543046)	213.9	12.8
LS-2	4,852,165.5 (43.809446)	301,368.7 (-79.542649)	213.3	9.8
LS-3	4,852,106.6 (43.808916)	301,348.8 (-79.542896)	205.4	8.2

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

As delineated in The Physiography of Southern Ontario (Chapman and Putnam, 1984)², this section of Highway 400 lies within the region known as the Peel Plain and consists of level to undulating tracts of clayey glacial till soils, which are presumed to have been derived from moraines, interspersed with non-cohesive silts and sands from interstadial stages of Wisconsinan glaciation.

Based on geological mapping by the Ministry of Northern Development and Mines (MNDM)³, the site is underlain by bedrock from the Upper Ordovician era consisting of shale, limestone, dolostone, and siltstone.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing from the investigation are shown on the borehole records presented in Appendix A. The detailed results of the geotechnical laboratory testing are presented in Appendix B. The results of the in situ field tests (i.e., SPT “N”-values) as presented on the borehole records and in Section 4.2 are uncorrected.

The stratigraphic boundaries shown in the borehole records are inferred from non-continuous sampling and, therefore, these boundaries represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the southeast embankment of the Langstaff Road Underpass consist of the existing Langstaff Road pavement structure underlain by cohesive embankment fill comprised of sandy clayey silt to silty clay having a stiff to hard consistency. The embankment fill is underlain by till or “till-like” soils comprised of clayey silt, sandy clayey silt-silt, silt and sand, and silt having a stiff to hard (but generally very stiff to hard) consistency. A more detailed description of the major stratigraphic units encountered in the boreholes is described in the sections below.

4.2.1 Asphalt

A layer of asphalt approximately 205 mm thick was encountered at ground surface in Boreholes LS-1 and LS-2, which were drilled through the Langstaff Road pavement.

² Chapman, L.J. and Putnam, D.F., 1984, The Physiography of Southern Ontario, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.)

³ Ministry of Northern Development of Mines. Bedrock Geology of Ontario – Southern Sheet, Ontario Geological Survey - Map 2544.

4.2.2 Topsoil

A layer of topsoil approximately 150 mm thick was encountered at the ground surface in Borehole LS-3, which was drilled at the toe of the embankment slope.

4.2.3 Granular Fill (Pavement Structure)

A layer of granular fill consisting of poorly graded gravelly sand was encountered underlying the asphalt in Boreholes LS-1 and LS-2, which were advanced through the crest of the Langstaff Road embankment on the south side of the roadway. The granular fill was encountered at a depth of approximately 0.2 m below ground surface (approximately Elevations 213.7 and 213.1 m) and was about 0.6 m to 1.3 m thick, extending to depths of 0.8 m to 1.5 m below ground surface (to approximately Elevation 212.4 m).

The SPT “N”-values measured within the granular fill range from 23 to 45 blows per 0.3 m penetration, indicating a compact to dense state of compactness. In one instance in the granular fill, the split-spoon sampler did not penetrate the entire SPT depth due to refusal conditions (100 blows for less than 0.3 m of penetration).

The water content measured on samples of the granular fill ranges from about 2% to 5%.

4.2.4 CLAYEY SILT (CL) to SILTY CLAY (CI) (FILL) – Embankment Fill

Cohesive embankment fill was encountered underlying the pavement structure in Boreholes LS-1 and LS-2 and underlying the topsoil in Borehole LS-3. The embankment fill was encountered at depths ranging from 0.8 m to 1.4 m below ground surface (approximately Elevations 212.4 m to 205.3 m) and extended to the termination depth of 9.8 m (approximately Elevation 203.5 m) in Borehole LS-2. In Boreholes LS-1 and LS-3, the embankment fill was about 2.2 m to 8.8 m thick, extending to Elevations 203.6 m to 203.2 m.

The SPT “N”-values measured within the cohesive fill range from 11 to 31 blows per 0.3 m of penetration, indicating a stiff to hard consistency. In one instance in the embankment fill, the split-spoon sampler did not penetrate the entire SPT depth due to refusal conditions (100 blows for less than 0.3 m of penetration).

Grain size distribution testing was carried out on five samples of the cohesive fill and the results are presented on Figure B1 in Appendix B. Atterberg limit testing was carried out on five samples of the cohesive fill and the results are presented on a plasticity chart in Figure B2 in Appendix B. The Atterberg limits tests measured liquid limits ranging from about 24% to 46%, plastic limits ranging from about 13% to 19%, and plasticity indices ranging from about 11% to 27%. The Atterberg limits tests generally indicate a clayey silt to silty clay of low to medium plasticity. The water content measured on samples of the cohesive fill ranges from about 11% to 22%, generally near the plastic limit of the material.

4.2.5 CLAYEY SILT (CL)

A cohesive deposit of clayey silt was encountered underlying the cohesive embankment fill in Borehole LS-3, which was advanced near the original ground surface at the toe of the Langstaff Road southeast embankment. The cohesive deposit was encountered at a depth of 2.2 m below ground surface (at approximately Elevation 203.2 m) and was about 1.5 m thick, extending to a depth of 3.7 m (approximately Elevation 201.7 m).

The SPT “N”-values measured within the cohesive deposit ranges from 12 to 24 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency.

Grain size distribution testing was carried out on a sample of the cohesive deposit and the results are presented on Figure B3 in Appendix B. Atterberg limit testing was carried out on a sample of the cohesive deposit and the results

are presented on Figure B4 in Appendix B. The Atterberg limits test measured a liquid limit of about 22%, a plastic limit of about 13% and a corresponding plasticity index of about 9%. The Atterberg limits test generally indicates a clayey silt of low plasticity.

4.2.6 SILT (ML) to Sandy CLAYEY SILT-SILT (CL-ML) (TILL)

A glacial till deposit varying in composition from silt to sandy clayey silt-silt was encountered underlying the cohesive embankment fill in Borehole LS-1 and underlying the clayey silt deposit in Borehole LS-3. The glacial till material was encountered at depths ranging from 3.7 m to 10.2 m below ground surface (approximately Elevations 203.6 m to 201.7 m); both Boreholes LS-1 and LS-3 were terminated in the till after penetrating 2.6 m to 4.5 m into the deposit (to Elevations 201.2 m to 197.2 m).

The SPT “N”-values measured within the till deposit ranges from 11 to 65 blows per 0.3 m of penetration, indicating a stiff to hard consistency.

Grain size distribution testing was carried out on three samples of the glacial till and the results are presented on Figure B5 in Appendix B. Atterberg limit testing was carried out on three samples of the glacial till and the results are presented on a plasticity chart in Figure B6 in Appendix B. The Atterberg limits tests measured liquid limits ranging from about 14% to 22%, plastic limits ranging from about 11% to 18%, and plasticity indices ranging from about 3% to 4%. The Atterberg limits tests generally indicate a silt to clayey silt-silt of slight to low plasticity. The water content measured on samples of the till ranges from about 7% to 13%, slightly below the plastic limit of the material.

4.3 Groundwater Conditions

The groundwater levels measured in the open boreholes at the time of the investigation are not considered representative of the stabilized hydrostatic groundwater levels at the site. All water levels recorded as part of this subsurface exploration program were taken shortly after drilling operations and therefore represent an unstabilized groundwater level.

A standpipe piezometer was installed in Borehole LS-3 to allow monitoring of the stabilized groundwater level at this site. The groundwater levels recorded during drilling (i.e., the unstabilized groundwater levels) and the groundwater level recorded in the standpipe piezometer (i.e., the stabilized groundwater level) are shown on the borehole records in Appendix A and are summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth (Elevation) of Screen Interval / Sand Pack (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
LS-1	213.9	N/A	10.7	203.2	Mar. 30, 2023	Open borehole / inside hollow stem augers
LS-2	213.3		Dry	-	Mar. 30, 2023	Open borehole / inside hollow stem augers
LS-3	206.4		8.1	197.3	Jul. 17, 2023	Open borehole / inside hollow stem augers
		5.2 m to 8.2 m (El. 194.2 to 197.2 m)	2.1	203.3	Oct. 31, 2023	Standpipe Piezometer

Groundwater levels are subject to seasonal fluctuations and precipitation events and should be expected to be higher during wet periods of the year.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Sunduss Asghar, EIT, and Mr. Mark Henderson, P.Eng., a Geotechnical Engineer with WSP Golder. Ms. Lisa Coyne, P.Eng., a Geotechnical Engineering Fellow and MTO Principal Foundations Contact for WSP Golder, conducted an independent technical and quality control review of this report.

Signature Page

WSP Golder



Sunduss Asghar
Geotechnical EIT



Mark Henderson, P.Eng.
Geotechnical Engineer



Lisa Coyne, P.Eng.
Fellow, MTO Principal Foundations Contact

MH/LCC/al

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PART B

Foundation Design Report
Slope Stability Assessment
Langstaff Road Underpass Southeast Embankment
Highway 400 Widening
Langstaff Road to Major Mackenzie Drive
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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical/foundation design recommendations for the proposed slope remediation works for the southeast embankment (i.e., the south side of the east approach embankment in the southeast quadrant of the Highway 400/Langstaff Road interchange), as part of the Highway 400 widening from south of Langstaff Road to Major Mackenzie Drive in the City of Vaughan, Ontario.

These recommendations are based on interpretation of the data obtained from the boreholes advanced during the current field investigations. The discussion and recommendations presented are intended to provide the designers with information to carry out the detail design of the Langstaff Road southeast embankment slope remediation works. The discussion and recommendations in this Foundation Design Report are intended for the use of MTO and its designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. Contractors must make their own interpretation based on the factual data presented in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction must make their own interpretation of the data provided as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

6.2 Project Understanding

Tilting of the existing guardrail posts and settlement along the concrete sidewalk has been identified at the Highway 400/Langstaff Road southeast embankment (see Photograph 2), triggering geotechnical investigation to assess the potential cause of this deformation/settlement and to provide foundation engineering recommendations for remedial works if and as applicable. Apart from the guardrail tilting and settlement of the sidewalk, the embankment side slopes are performing satisfactorily, with no visual signs of global instability (see Photograph 1).

At the Langstaff Road interchange, Highway 400 has been constructed near the original ground surface, with Langstaff Road and the associated interchange ramps constructed on cohesive embankment fill. The Langstaff Road grade near the southeast embankment is about Elevation 213 m to 214 m and the toe of the embankment is at about Elevation 205 m to 206 m (i.e., embankment height on the order of about 8 m). The southeast embankment has a side slope inclination of about 2H:1V, although according to survey information for this area the upper portion of the slope appears to be slightly steeper than 2H:1V. A concrete box culvert runs through the embankment to facilitate drainage along Black Creek. The inlet and outlet locations of the box culvert are near the toe of the slope on both sides of the embankment with retaining walls consisting of rows of gabions on each side of the culvert; based on our site observations, there is no visual evidence of distress (tilting or loss of ground) associated with the existing gabion wall at the south end of the culvert. There is also a culvert that runs beneath Highway 400 and connects to Black Creek immediately south of the embankment toe.

6.3 Assessment of Mechanism(s) for Observed Deformations

6.3.1 Global Stability of Embankment

Limit equilibrium slope stability analyses was carried out for the southeast embankment using the commercially available program Slide2 (Version 9.017) by Rocscience Inc., employing the Morgenstern-Price method of analysis. For the analyses, the Factor of Safety (FoS) of numerous potential surfaces was computed in order to establish the minimum FoS. The stability analyses were performed to check that the target minimum FoS was achieved for the

existing embankment height and geometry, which was generated from the Base Plan drawings provided by Parsons. In general, circular slip surfaces were analysed. The global stability analyses were carried out using borehole information from Boreholes LS-1 and LS-3, as shown in the soil strata section of Drawing 1.

The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the FoS is equal to the inverse of the product of the consequence factor, Ψ , and the geotechnical resistance factor, Φ_{gu} (i.e., $\text{FoS} = 1 / (\Psi * \Phi_{gu})$). Accordingly, a target minimum FoS of 1.33 has been used for comparison of the existing embankment configuration with temporary (short-term/undrained) total stress conditions, and a FoS of 1.54 has been used for comparison of the existing embankment configuration with permanent (long term/drained) effective stress conditions as per Table 6.2 of CHBDC (2019) and MERO (2020).

For the non-cohesive granular fill, effective stress parameters were employed in the analysis assuming drained conditions, and the strength parameters were estimated from empirical correlations based on the in-situ SPT 'N'-values. The correlations proposed by Terzaghi and Peck (1967) were employed, and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive embankment fill, native clayey silt, and borderline cohesive glacial till, total stress parameters were employed for the short-term, undrained conditions. The total stress parameters (i.e., average mobilized undrained shear strength - s_u) for these soils were estimated from correlations with the SPT 'N'-values. Effective friction angles have also been estimated for these soils for analysis of the factor of safety in the long-term, drained condition.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types encountered at the Langstaff Road Underpass southeast embankment. The groundwater level for the analyses was taken as Elevation 203 m.

Soil Deposit	Bulk Unit Weight (kN/m ³)	Short-Term Analysis		Long-Term Analysis
		Effective Friction Angle (°)	Undrained Shear Strength (kPa)	Effective Friction Angle (°)
Existing Compact Granular Fill <i>Gravelly Sand (SP) (FILL)</i>	19	30	-	30
Stiff to Hard Fill (Cohesive) <i>Clayey Silt-Silt (CL-ML) to Silty Clay (CI) (FILL)</i>	19	-	75	30
Stiff to Very Stiff Clayey Silt (CL)	19	-	100	30
Very Stiff to Hard Glacial Till <i>Silt (ML) to Clayey Silt-Silt (CL-ML) (TILL)</i>	20	-	200	32

The analyses yielded a FoS of about 3.3 in the short-term (undrained) condition and a FoS of about 1.55 in the long-term (drained) condition for the existing southeast embankment side slope. Based on the existing embankment geometry and encountered subsurface conditions, with consideration of the observed embankment performance, these calculated FoS values demonstrate that global instability is not an issue for this site and has not contributed to settlement of the sidewalk or tilting of the guardrail.

6.3.2 Composition of Embankment Fill

At the Langstaff Road interchange, Highway 400 has been constructed near the original ground surface, with Langstaff Road and the associated interchange ramps constructed on cohesive embankment fill as part of the 1990's construction contract for the Langstaff Road Underpass. Boreholes LS-1 and LS-2 were advanced on the south side of Langstaff Road, adjacent to the leaning guardrail and sidewalk (i.e., through the embankment crest). Based on the results of these boreholes, the cohesive embankment fill has a stiff to hard consistency and is relatively homogeneous (i.e., consistent plasticity ranging from low to intermediate and consistent grain size distribution), apart from a 0.3 m thick gravel pocket encountered around Elevation 210.5 m and a 1.6 m thick silty sand layer encountered between Elevation 206.7 m and 208.2 m in Borehole LS-1. These observations and test results do not correlate with poor embankment fill materials, and therefore it is considered unlikely that these materials are contributing to the localized settlement at the edges of the sidewalk and guardrail.

6.3.3 Settlement

Borehole LS-3 was advanced at the toe of the embankment to a depth of about 8.2 m below ground surface (Elevation 197.2 m, or some 5 to 7 m below the Langstaff Road embankment fill). No organic or excessively soft soils were encountered in Borehole LS-3. The subsurface conditions beneath the Langstaff Road embankment fill generally consist of borderline cohesive glacial till soils comprised silt, silt and sand, and sandy clayey silt-silt. These soils are overconsolidated and have a consistency ranging from stiff to hard (but generally hard, based on six SPTs obtained within the deposit). Based on the sand content and estimated coefficient of consolidation of this deposit, settlement would be expected to occur rapidly, i.e., immediately or shortly after the original embankment construction in the 1990s. An approximately 1.5 m thick cohesive deposit of stiff to very stiff clayey silt was encountered above the glacial till between Elevations 203.2 m and 201.7 m; although this deposit would be expected to undergo time-dependent consolidation settlement, total settlements due to an 8 m embankment load would be less than 25 mm, which does not correlate with the localized settlement at the edges of the sidewalk and guardrail.

6.3.4 Proximity of Guardrail and Sidewalk to Existing Slope Crest

Based on the slope stability and settlement assessment outlined above, the FoS for the existing embankment configuration meets the minimum target FoS for permanent conditions, the embankment consists of suitable cohesive fill materials, and there are no excessively soft or organic soils contributing to localized settlement at the toe of the embankment. It is therefore considered that the proximity of the guardrail and sidewalk to the crest of the embankment side slope has contributed to tilting of the guardrail, owing to reduced passive pressure on the downslope side of the guardrail posts. It is also possible that some surficial erosion or vegetation "creep" may have occurred on the embankment side slope, as the upper portion of the existing embankment side slope is slightly steeper than 2H:1V.

6.4 Slope Remediation Recommendations

It is recommended that the upper portion of the existing southeast side slope be regraded to an inclination of 2H:1V, new guardrail posts be installed, and the sidewalk be reconstructed. If it is possible to do so (given the proximity of the slope toe to the culvert and watercourse), consideration could also be given to more substantial regrading of the slope so that the guardrail posts are further away from the embankment crest. It is recommended that the fill material used for regrading consist of OPSS.PROV 1010 Granular A, Granular B Type II or SSM as such materials will have a higher effective friction angle than cohesive earth fill materials, and thus will promote surficial embankment stability. However, as the existing embankment fill is cohesive, fill for the regrading could also consist of cohesive earth fill excavated from elsewhere on the contract.

Regrading operations should be in accordance with the requirements of OPSS.PROV 206 (Construction Specifications for Grading), and the fill material should be benched into the existing side slope as per OPSD 208.010 (Benching of Earth Slopes). The new embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*).

To reduce surface water erosion on the cohesive embankment side slopes, topsoil and seeding as per OPSS.PROV 803 (*Vegetative Cover*) should be carried out as soon as possible after reconstruction of the embankment. Consideration could also be given to providing additional erosion protection measures to enhance stability and reduce the potential for surficial erosion or vegetation “creep”. Additional erosion protection measures could consist of rip-rap, rock protection, or granular sheeting, meeting the requirements of OPSS.PROV 1004 (Aggregates – Miscellaneous), placed/constructed in accordance with OPSS.PROV 511 (Rip-Rap, Rock Protection, and Granular Sheeting).

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Mark Henderson, P.Eng., a Geotechnical Engineer with WSP Golder. Ms. Lisa Coyne, P.Eng., a Geotechnical Engineering Fellow and MTO Principal Foundations Contact for WSP Golder, conducted an independent technical and quality control review of this report.

Signature Page

WSP Golder



Mark Henderson, P.Eng.
Geotechnical Engineer



Lisa Coyne, P.Eng.
Fellow, MTO Principal Foundations Contact

MH/LCC/al

REFERENCES

Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual, 4th Edition*

Canadian Standard Association (CSA) Group. *Canadian Highway Bridge Design Code (CHBDC (2019)) and Commentary on CAN/CSA-S6-14.*

Chapman, L.J. and Putnam, D.F. 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

Ministry of Northern Development of Mines. *Bedrock Geology of Ontario – Southern Sheet*, Ontario Geological Survey – Map 2544.

ASTM International

ASTM D1586 Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils

Ontario Provincial Standard Drawings (OPSD)

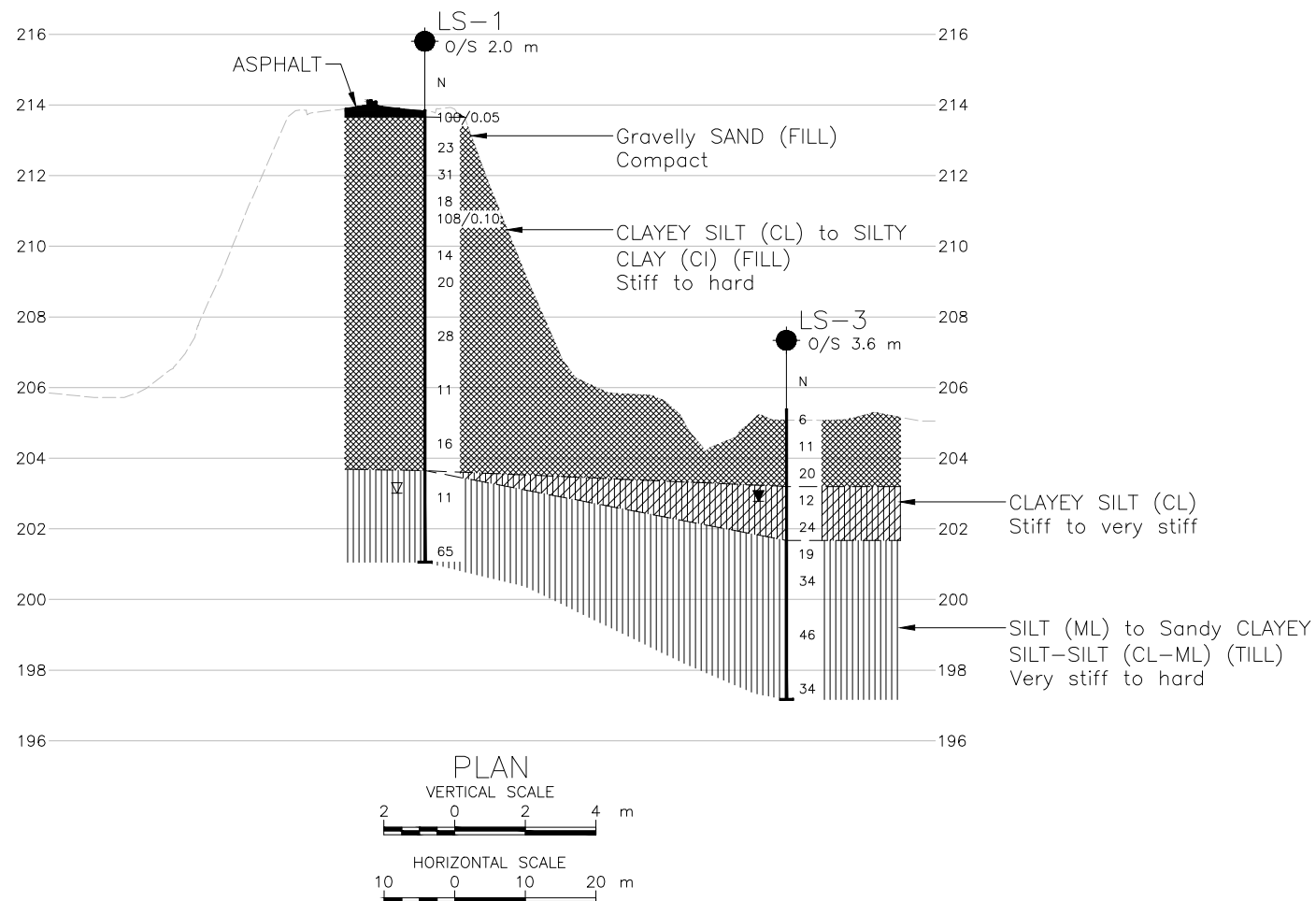
OPSD 208.010 Benching of Earth Slopes

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 803	Construction Specification for Vegetative Cover
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Water Resources Act

Ontario Regulation 903 Wells (as amended)



NO.		DATE		BY	
				REVISION	
Geocres No. 30M13-306					
HWY. 400			PROJECT NO. 21490972		DIST. .
SUBM'D. MH		CHKD. MH		DATE: 11/15/2023	SITE:
DRAWN: DD		CHKD. MH		APPD. LCC	DWG. 1



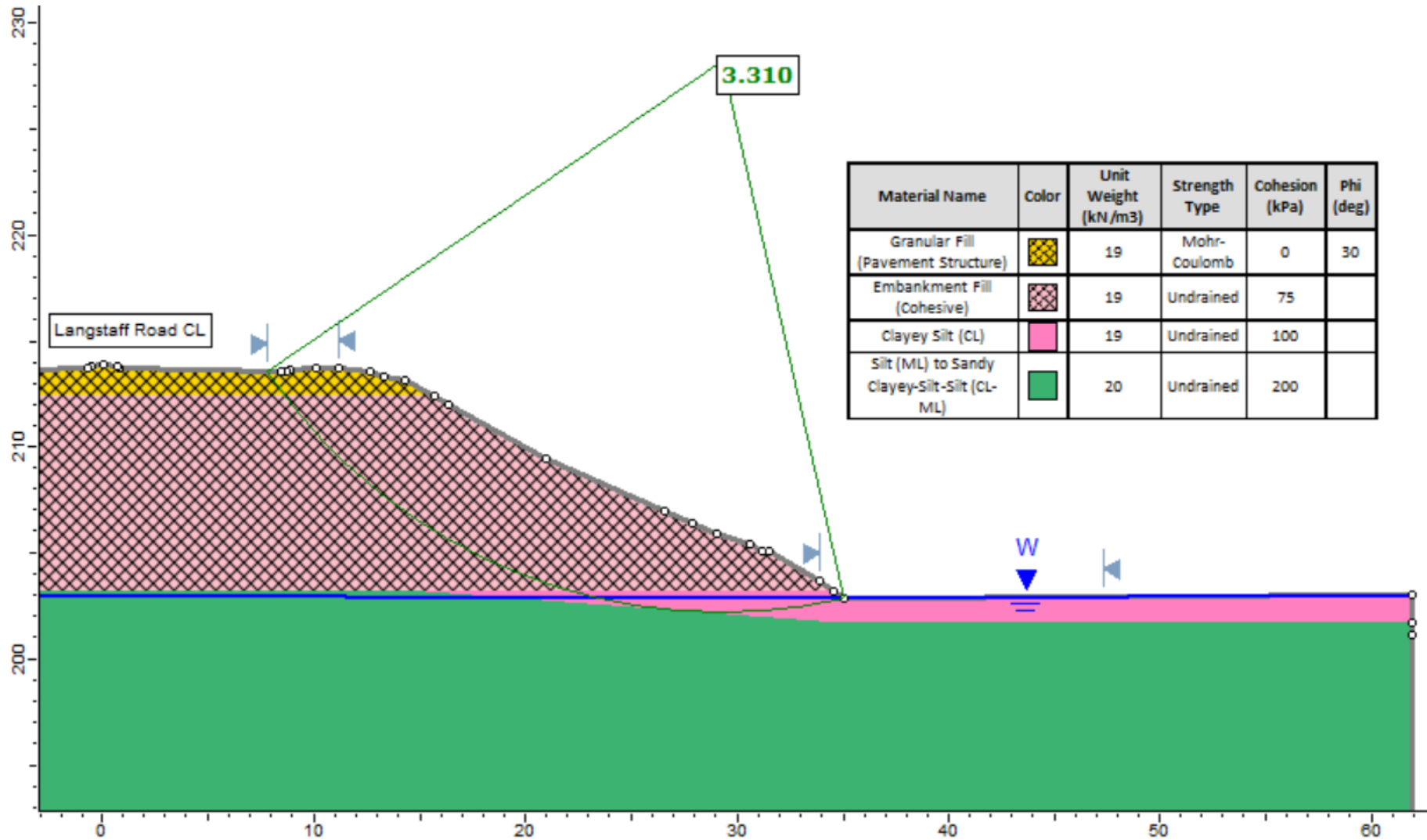
Photograph 1: Looking northwest at existing southeast embankment side slope; note the section of leaning guardrail in the background

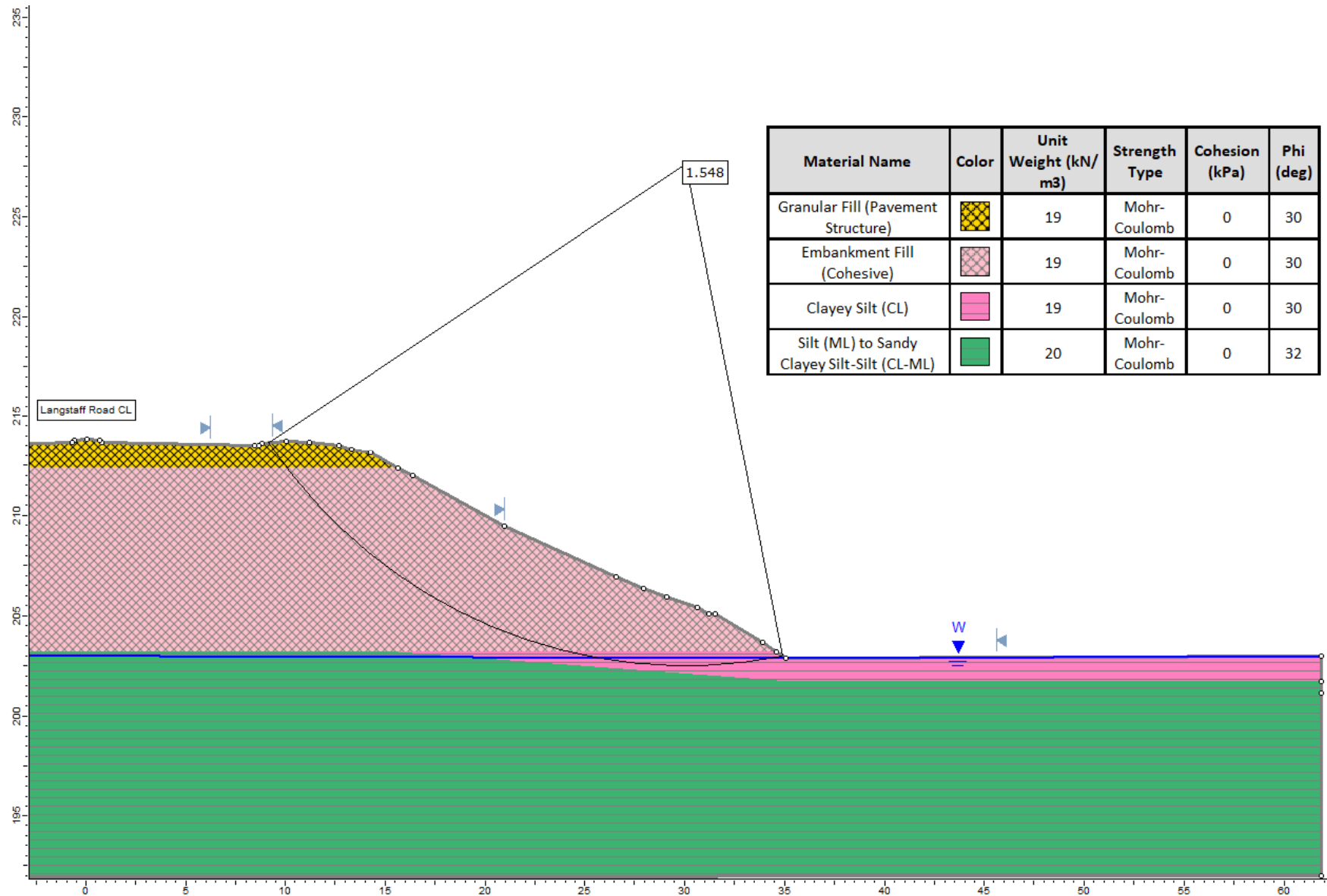


Photograph 2: Looking west at sidewalk along the crest of the southeast embankment; note the section of leaning guardrail in the background



Photograph 3: Looking east towards gabion walls at Black Creek Triple Cell Culvert outlet (photograph taken at the toe of the southeast embankment)





APPENDIX A

Borehole Records

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
		2.00 to 4.75	(10) to (4)
SAND	Coarse	0.425 to 2.00	(40) to (10)
	Medium	0.075 to 0.425	(200) to (40)
	Fine		
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (i.e., SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (i.e., some sand)
≤ 10	trace (i.e., trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

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.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An 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 (u) and sleeve friction (f_s) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); 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

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w_p	plastic limit
LL, 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
GS	specific gravity
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 (FV)	field vane (LV-laboratory vane test)
Y	unit weight

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

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

1. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

2. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.

2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

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

I. GENERAL

π	3.1416
$\ln x$	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

(a) Index Properties (continued)

w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index = $(w_L - w_P)$
NP	non-plastic
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
$C_{a(e)}$	secondary compression index
C_a	rate of secondary compression
$C_{a(e)}$	modified secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
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 or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (compressive strength)/2

PROJECT 21490972

RECORD OF BOREHOLE No. LS-1

Sheet 1 of 2

METRIC

G.W.P. 2836-02-00

LOCATION N 4852156.4; E 301336.7 NAD83 / MTM Zone 10 (LAT. 43.809364; LONG. -79.543046)

ORIGINATED BY M.L.

DIST CENTRAL HWY 400

BOREHOLE TYPE Power Auger; 159 mm O.D. Hollow Stem Augers

COMPILED BY M.L.

DATUM Geodetic Surface Elevation:213.9 m

DATE Mar 30, 2023

CHECKED BY M.H.

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y kN/m³	GR	SA	SI	CL	REMARKS
ELEV. ----- DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL	NMC	LL						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined	20	40	60	80	100	W _p	W						
0.0	ASPHALT (205 mm)																				
0.2	Gravelly SAND (SP), some silt (FILL) Compact Brown Moist		1	SS	100/0.05																
213.7																					
			2	SS	23																
212.4																					
1.4	Sandy CLAYEY SILT (CL), trace gravel (FILL) contains organics Stiff to hard Brown; becoming grey at about 4.6 m depth (Elev. 201.3 m) Moist - 3.1 to 3.4 m: Gravel pocket with black colouration (Elev. 210.7 m to Elev. 210.5 m) - 5.6 to 7.2 m: Silty sand layer (Elev. 208.2 m to Elev. 206.7 m)		3	SS	31																
			4	SS	18																
			5	SS	108/0.10																
			6	SS	14																
			7	SS	20																

Continued on Next Page

+³, x³ : Numbers refer to Sensitivity o³⁰% STRAIN AT FAILURE

METRIC

M.L.

M.L.

M.H.

+³, x³ : Numbers refer to Sensitivity o^{30%} STRAIN AT FAILURE

PROJECT	21490972	RECORD OF BOREHOLE	No. LS-2	Sheet 1 of 1	METRIC
G.W.P.	2836-02-00	LOCATION	N 4852165.5; E 301368.7 NAD83 / MTM Zone 10 (LAT. 43.809446; LONG. -79.542649)	ORIGINATED BY	M.L.
DIST	CENTRAL HWY 400	BOREHOLE TYPE	Power Auger; 159 mm O.D. Hollow Stem Augers	COMPILED BY	M.L.
DATUM	Geodetic Surface Elevation:213.3 m	DATE	Mar 30, 2023	CHECKED BY	M.H.

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					WATER CONTENT (%)			UNIT WEIGHT Y	GR	SA	SI	CL	REMARKS
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					PL W _p	NMC W	LL W _L						
								Field Vane Remoulded Pocket Pen Quick Triaxial Unconfined													
0.0	ASPHALT (205 mm)							20 40 60 80 100					20 40 60								
0.2 213.1	Gravelly SAND (SP), some silt (FILL) Dense Brown Moist		1	SS	45		213														
212.4	CLAYEY SILT (CL) and sand to sandy, trace gravel (FILL) Stiff to hard Brown; becoming grey at about 7.2 m depth (Elev. 206.1 m). Moist		2	SS	31		212														
			3	SS	17		211											3	37	42	18
			4	SS	18		210														
			5	SS	12		209														
			6	SS	25		208														
			7	SS	13		207														
							206														
							205														
204.6	SILTY CLAY (CI), some sand (FILL) Stiff Brown Moist		8	SS	14		204														
8.7			9	SS	11																
203.5			10	SS	14																
9.8	End of Borehole 1. Borehole open and dry upon completion of drilling.																				

+³, x³ : Numbers refer to Sensitivity o³⁰% STRAIN AT FAILURE

METRIC

T.T.

P.T.

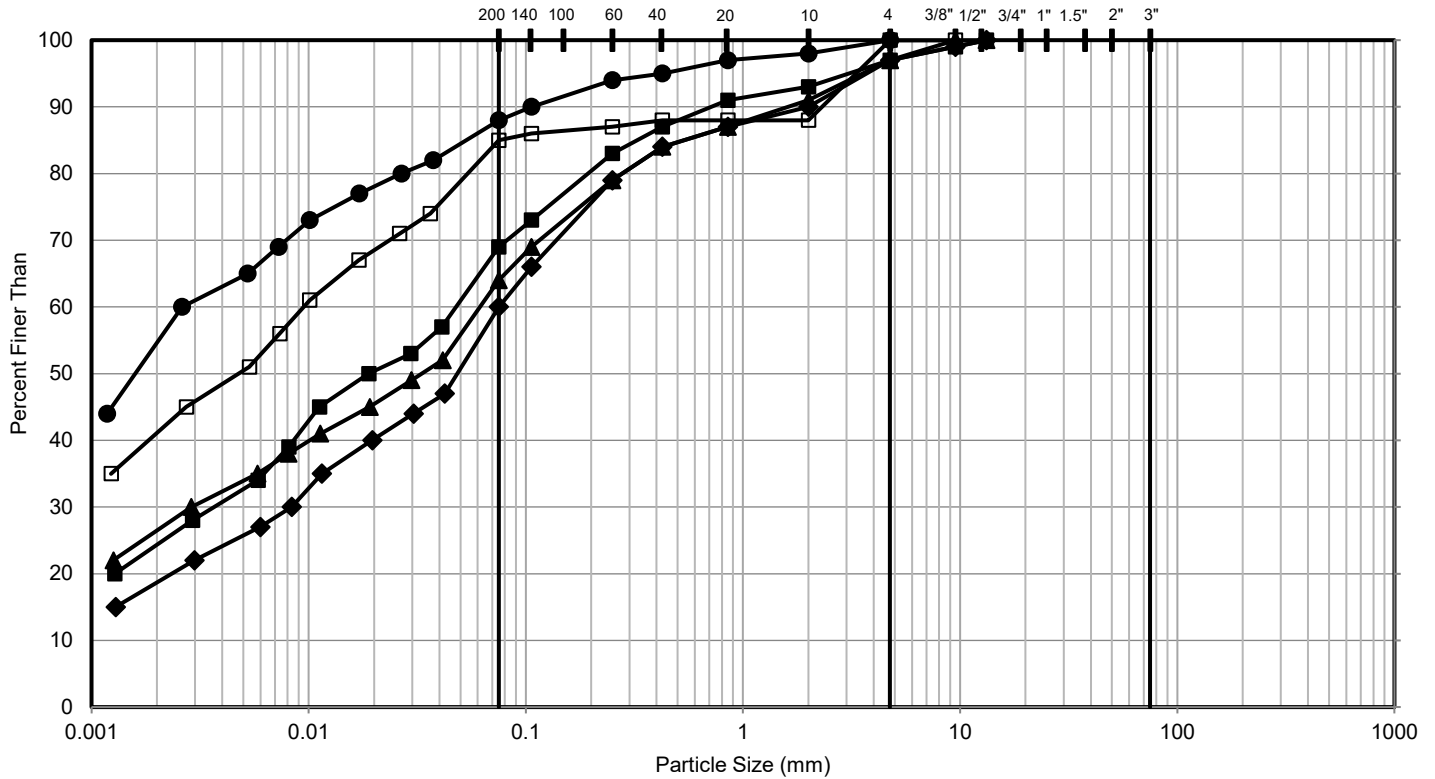
M.H.

+³, x³ : Numbers refer to Sensitivity o^{30%} STRAIN AT FAILURE

APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	LS-1	4	2.3 - 2.9	211.6 to 211.0
◆	LS-2	3	1.5 - 2.1	211.7 to 211.1
▲	LS-2	8	6.1 - 6.7	207.2 to 206.6
●	LS-2	10	9.1 - 9.8	204.1 to 203.5
□	LS-3	2	0.8 - 1.4	204.6 to 204.0

CLIENT

PARSONS / MTO

CONSULTANT



YYYY-MM-DD 2023-08-11

DESIGNED TT

PREPARED TT

REVIEWED MH

APPROVED LCC

PROJECT

LANGSTAFF ROAR UNDERPASS SOUTHEAST EMBANKMENT
HIGHWAY 400 WIDENING
GWP 2836-02-00

TITLE

GRAIN SIZE DISTRIBUTION
CLAYEY SILT (CL) to SILTY CLAY (CI) (FILL)

PROJECT NO.

21490972

CONTROL

0

REV.

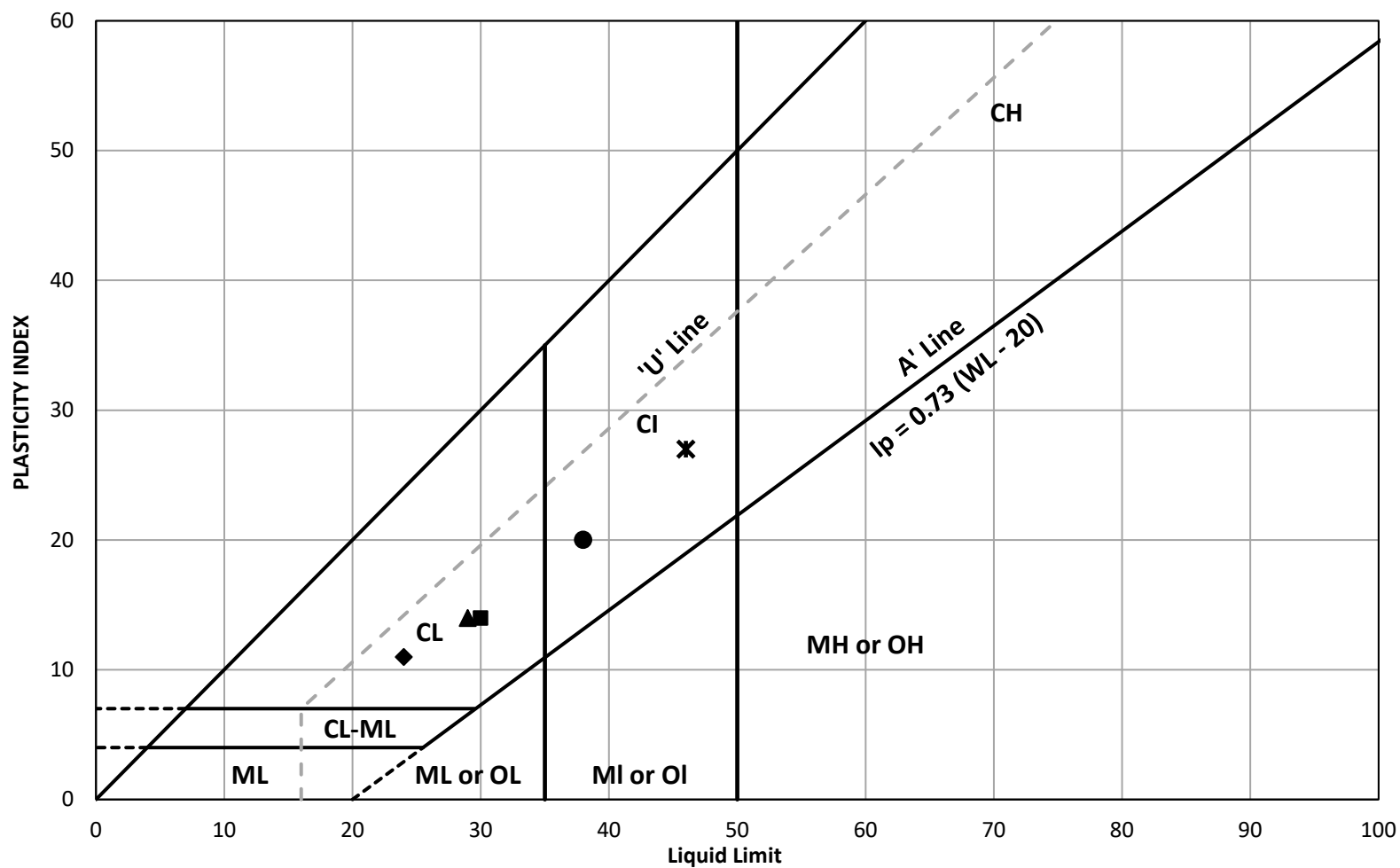
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FIGURE

B1

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PLASTICITY CHART

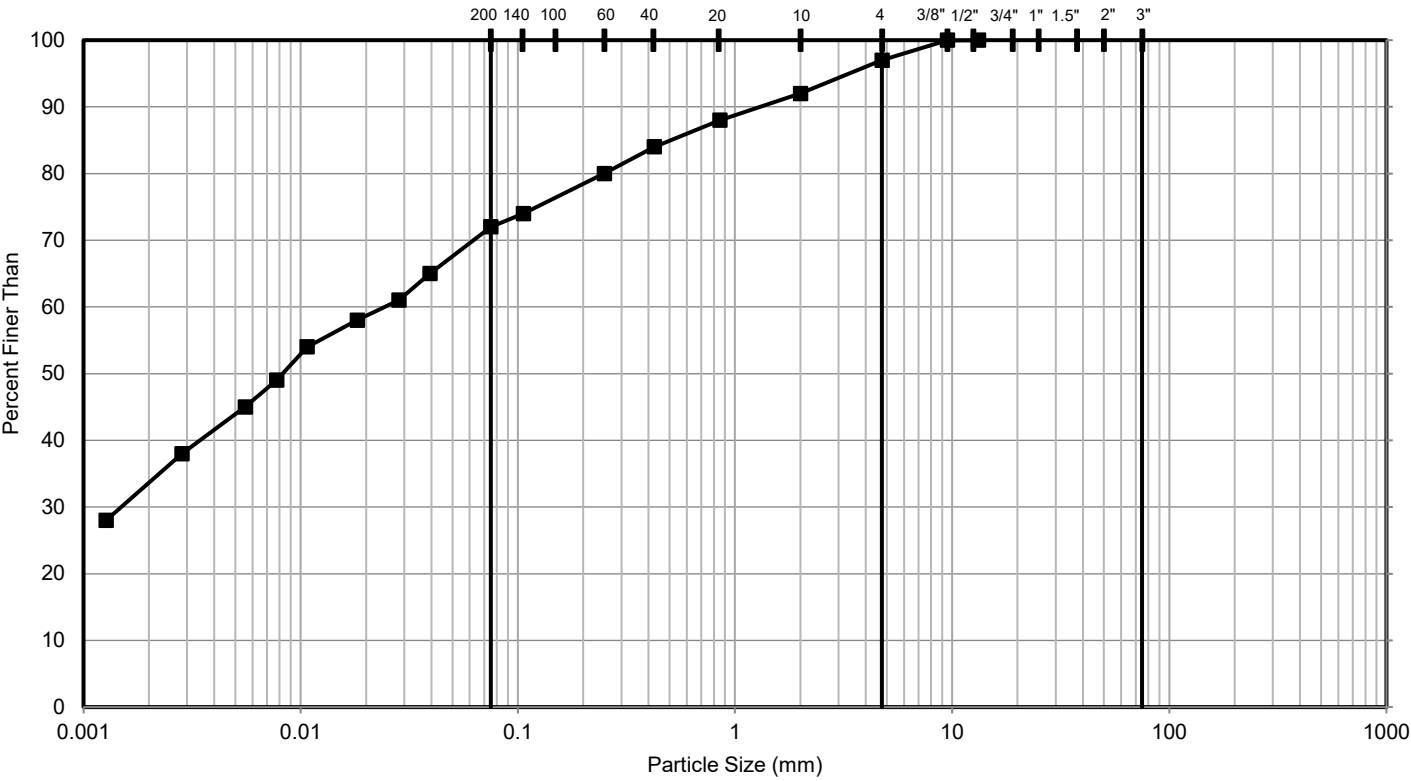


	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	
■	LS-1	4	2.3 - 2.9	18.3	30	16	14	
◆	LS-2	3	1.5 - 2.1	11.3	24	13	11	
▲	LS-2	8	6.1 - 6.7	16.1	29	15	14	
●	LS-2	10	9.1 - 9.8	21.6	38	18	20	
*	LS-3	2	0.8 - 1.4	-	46	19	27	

CLIENT		
PARSONS / MTO		
CONSULTANT	YYYY-MM-DD	2023-08-11
	DESIGNED	TT
	PREPARED	TT
	REVIEWED	MH
	APPROVED	MH

PROJECT			
LANGSTAFF ROAD SOUTHEAST EMBANKMENT HIGHWAY 400 WIDENING, GWP 2836-02-00			
TITLE			
PLASTICITY CHART CLAYEY SILT (CL) to SILT CLAY (CI) (FILL)			
PROJECT NO.	CONTROL	REV.	FIGURE
21490972	0	0	B2

GRAIN SIZE DISTRIBUTION



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	LS-3	4	2.3 - 2.9	203.1 to 202.5

CLIENT

PARSONS / MTO

CONSULTANT

 **GOLDER**

YYYY-MM-DD

2023-08-11

DESIGNED

TT

PREPARED

TT

REVIEWED

MH

APPROVED

LCC

PROJECT

LANGSTAFF ROAR UNDERPASS SOUTHEAST EMBANKMENT
HIGHWAY 400 WIDENING
GWP 2836-02-00

TITLE

GRAIN SIZE DISTRIBUTION
CLAYEY SILT (CL)

PROJECT NO.

CONTROL

REV.

FIGURE

21490972

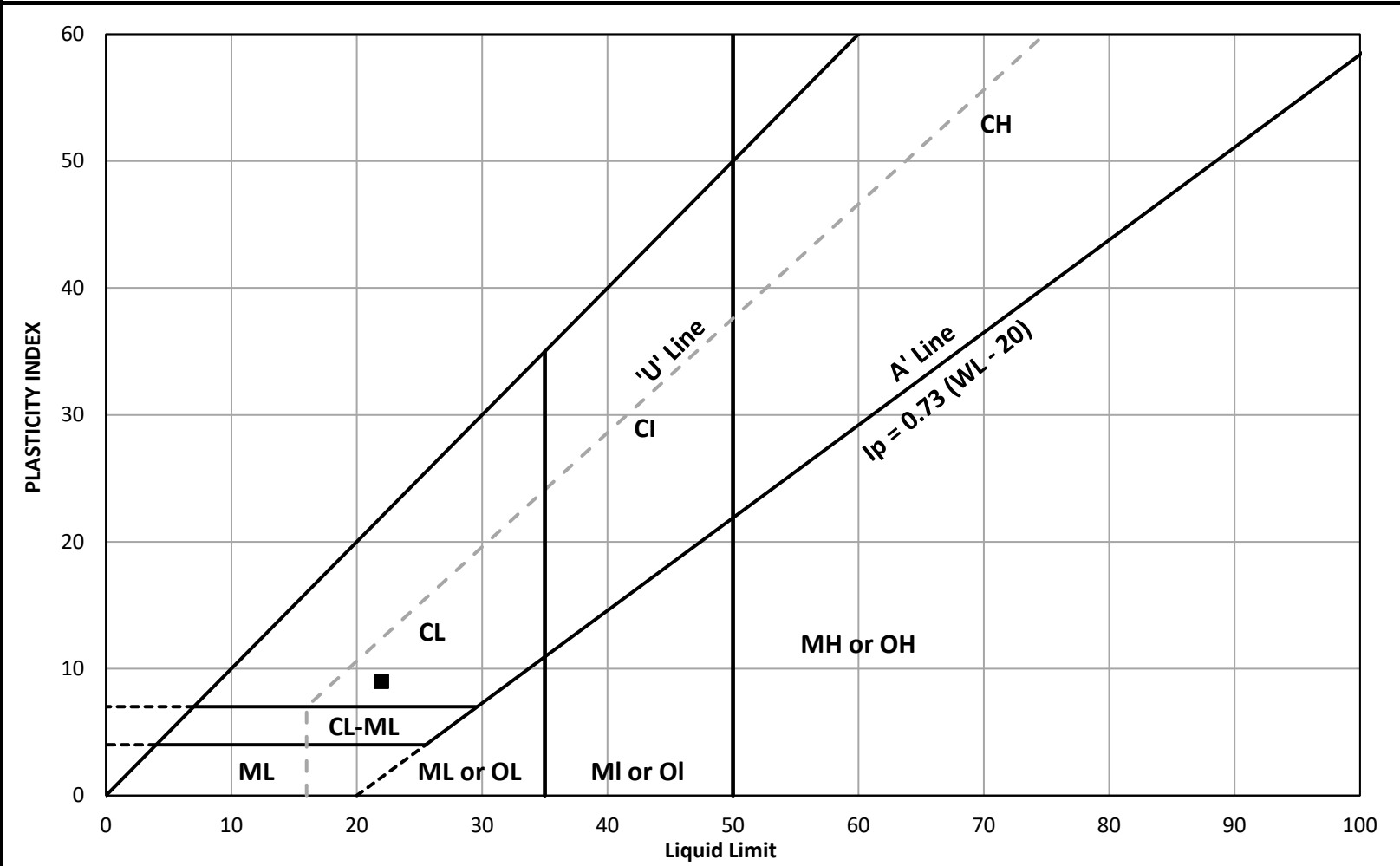
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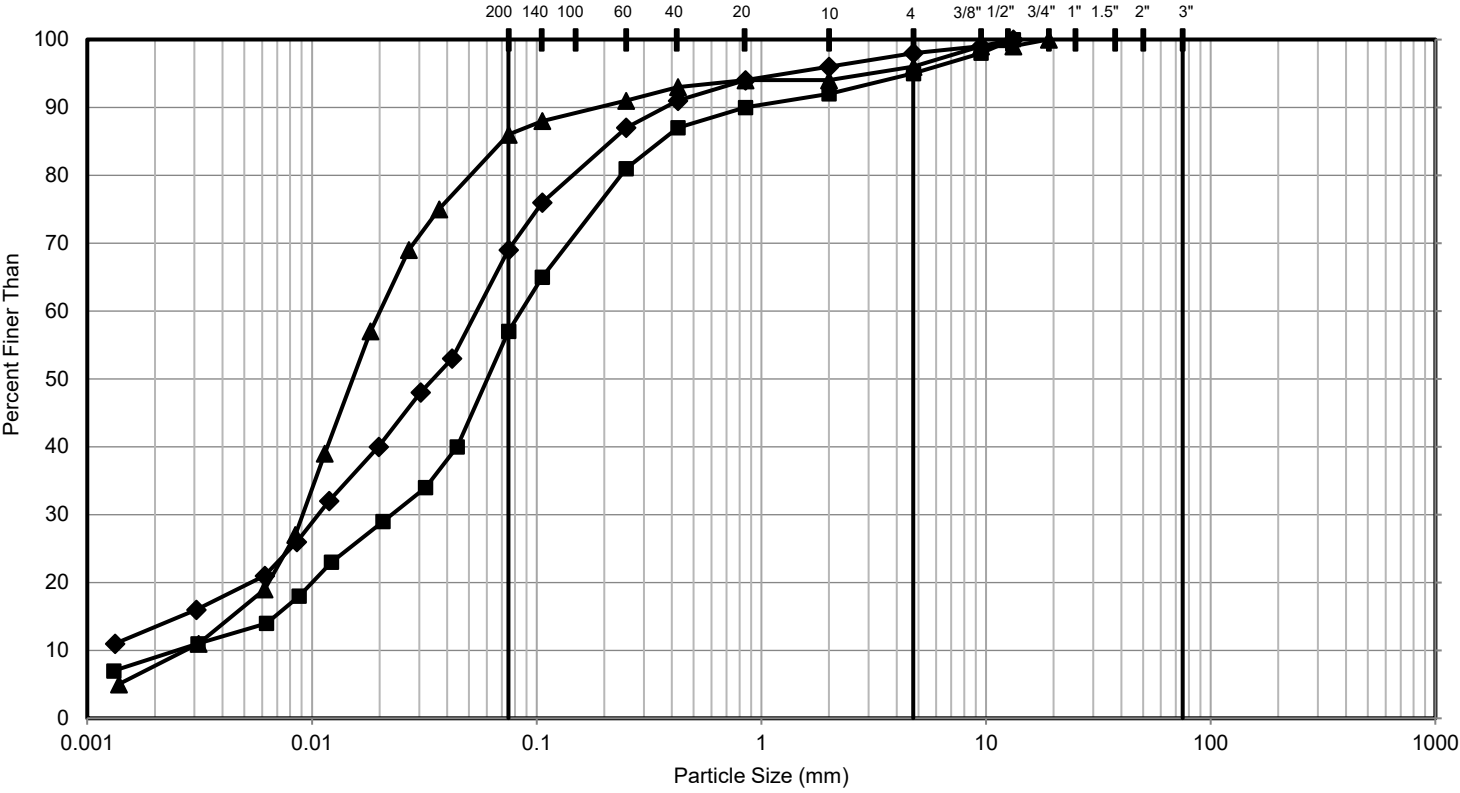


	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	
■	LS-3	4	2.3 - 2.9	-	22	13	9	

CLIENT		
PARSONS / MTO		
CONSULTANT	YYYY-MM-DD	2023-08-11
	DESIGNED	TT
	PREPARED	TT
	REVIEWED	MH
	APPROVED	LCC

PROJECT			
LANGSTAFF ROAD SOUTHEAST EMBANKMENT HIGHWAY 400 WIDENING, GWP 2836-02-00			
TITLE			
PLASTICITY CHART CLAYEY SILT (CL)			
PROJECT NO.	CONTROL	REV.	FIGURE
21490972	0	0	B4

GRAIN SIZE DISTRIBUTION



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	LS-1	11	10.7 - 11.3	203.2 to 202.6
◆	LS-1	12	12.2 - 12.8	201.7 to 201.1
▲	LS-3	8	6.1 - 6.7	199.3 to 198.7

CLIENT

PARSONS / MTO

CONSULTANT

 **GOLDER**

YYYY-MM-DD

2023-08-11

DESIGNED

TT

PREPARED

TT

REVIEWED

MH

APPROVED

LCC

PROJECT

LANGSTAFF ROAR UNDERPASS SOUTHEAST EMBANKMENT
HIGHWAY 400 WIDENING
GWP 2836-02-00

TITLE

GRAIN SIZE DISTRIBUTION
SILT (ML) to CLAYEY SILT-SILT (CL-ML)

PROJECT NO.

CONTROL

REV.

FIGURE

21490972

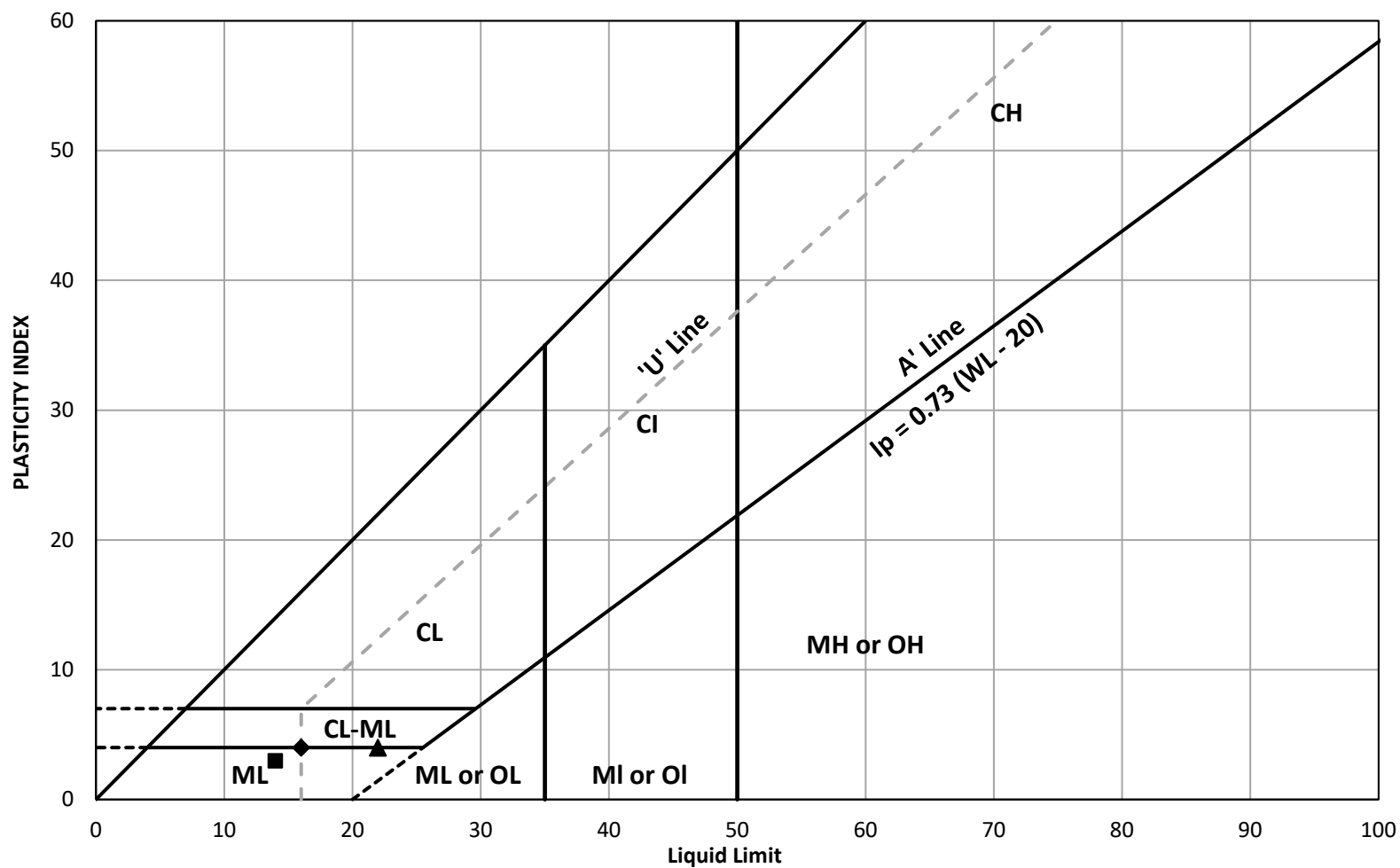
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PLASTICITY CHART



	Sample Location	Sample / Specimen Number	Depth (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	
■	LS-1	11	10.7 - 11.3	6.6	14	11	3	
◆	LS-1	12	12.2 - 12.8	9.7	16	12	4	
▲	LS-3	8	6.1 - 6.7	-	22	18	4	

CLIENT		
PARSONS / MTO		
CONSULTANT	YYYY-MM-DD	2023-08-11
	DESIGNED	TT
	PREPARED	TT
	REVIEWED	MH
	APPROVED	LCC

PROJECT			
LANGSTAFF ROAD SOUTHEAST EMBANKMENT HIGHWAY 400 WIDENING, GWP 2836-02-00			
TITLE			
PLASTICITY CHART SILT (ML) to CLAYEY SILT-SILT (CL-ML)			
PROJECT NO.	CONTROL	REV.	FIGURE
21490972	0	0	B6

