



September 22, 2017

FOUNDATION INVESTIGATION AND DESIGN REPORT

Westchester Avenue Underpass Structure Site 18-172 Highway 406 Structural Rehabilitation from Fourth Avenue to Westchester Avenue St. Catharines Ontario G.W.P. 2453-13-00

Submitted to:
AECOM
30 Leek Crescent
Richmond Hill, ON
L4B 4N4



REPORT

GEOCRES NO: 30M3-298

Report Number: 1541610-2

Distribution:

- 1 PDF+ Hard Copy - MTO Central Region
- 1 PDF+ Hard Copy - MTO Foundations Section
- 1 PDF+ Hard Copy- AECOM
- 1 Copy - Golder Associates Ltd.





Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION.....	1
2.0 SITE DESCRIPTION.....	1
3.0 PREVIOUS INVESTIGATION	2
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS	3
4.1 Regional Geology	3
4.2 Subsoil Conditions	3
4.2.1 Fill	4
4.2.2 Silty Clay to Clay	4
4.2.3 Sandy Clayey Silt Till	5
4.2.4 Groundwater	6
5.0 CLOSURE.....	7

PART B – FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS.....	8
6.1 General.....	8
6.2 Summary of Existing Foundations	8
6.3 Consequence and Site Understanding Classification	9
6.4 Assessment of Existing Foundations	9
6.5 Summary of Proposed Rehabilitation Options	10
6.6 Geotechnical Assessment of Proposed Rehabilitation Options	12
7.0 CLOSURE.....	15

DRAWINGS

Drawing 1 Borehole Locations and Soil Strata

APPENDICES

APPENDIX A

GEOCRE 30M3-42 Record of Borehole Sheets and Laboratory Data

APPENDIX B

GEOCRE 30M3-42 - Letter Dated November 29, 1965 and Drawing D-4965-1 General Plan, dated December 1961 and Drawing D-4965-2 Details of Abutment and Pier Footings



**FOUNDATION REPORT
WESTCHESTER AVENUE UNDERPASS
HWY 406, ST CATHARINES, ONTARIO G.W.P. 2453-13-00**

PART A

**FOUNDATION INVESTIGATION REPORT
WESTCHESTER AVENUE UNDERPASS
STRUCTURE SITE NO. 18-172
HIGHWAY 406 STRUCTURAL REHABILITATION
FOURTH AVENUE TO WESTCHESTER AVENUE
ST. CATHARINES, ONTARIO
G.W.P. 2453-13-00**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the future rehabilitation of nine structures along Highway 406 from Fourth Avenue to Westchester Avenue in the City of St. Catharines, Ontario.

This report presents the geotechnical site conditions at the site of the existing Westchester Avenue underpass structure at Highway 406. It was developed with information from a foundation investigation completed in 1961 at the Westchester Avenue underpass site, reported as follows:

- **MTO GEOCRETS No. 30M3-42:** *Foundation Investigation for Underpass for Hwy #58, (Line 'B') at Westchester Ave, near St. Catharines, W.J. 60-F-100, W.P. 47-61, District #4, dated October 3, 1961.*
- **MTO GEOCRETS No. 30M3-42:** *Westchester Avenue Underpass, Hwy #58, Line 'C', Dist. #4, W.P. 47-61, W.J. 60-F-100(B), dated January 8, 1962.*

The Terms of Reference and Scope of Work for the foundation engineering services are outlined in MTO's Request for Proposal, dated September 2015 that form part of the Consultant's Agreement (Number 2014-E-0075) for this project. The Scope of Work for the Westchester Avenue underpass is comprised of a Desktop Study as reported herein. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for this project, dated June 2016.

2.0 SITE DESCRIPTION

The Westchester Avenue underpass structure is located about 330 m east of the Geneva Street underpass, near the downtown core of St. Catharines (see Key Plan on Drawing 1). The existing underpass is a four-span structure with a total length of approximately 83 m and was constructed in about 1962.

The underpass structure is located on what was the left bank of the old second Welland Canal (now called Canal Valley; adjacent to Twelve Mile Creek). Westchester Avenue traverse the crest of the Canal Valley and grades gently downward to the east. The *General Plan* (Drawing D4565-1, dated 1961) shows topographic contour lines adjacent to the Westchester Avenue, suggesting that local gullies bounded the east abutment of the underpass. From interpretation of the General Plan 1961 borehole investigation results described in Section 4.2, it is inferred that Westchester Avenue was partially constructed across an infilled gully. Prior to construction of the underpass structure the ground surface in the vicinity of the west abutment was at approximately Elevation 104.8 m, descending to about Elevation 101 m at the east abutment. The current grade of Highway 406 is at about Elevation 100 m; therefore, Highway 406 was constructed in a 1 m to 4 m cut. The grade of the Westchester Avenue underpass is about Elevation 105 m suggesting an approximately 4 m high east approach embankment.

Drawing No. D4965-2 titled "Details of Abutments and Pier Footings" indicates that the west abutment is supported on a spread footing about 2.1 m wide founded at approximate Elevation 101.6 m, on native very stiff silty clay deposit, and the three piers are supported on 3.0 m wide spread footings founded at about Elevation 95.6 m. The east abutment is supported on a pile cap founded on 0.3 m diameter steel tube piles filled with concrete driven to Elevation 88.4 m in the transition between the very stiff silty clay to clay and the underlying firm to stiff clayey silt to silty clay.



Documents obtained from GEOCRESS indicate that in 1965 it was reported that there was observed distortion of the bearings and bending of the anchor bolts at the east abutment; however, there was no observed movement at the west abutment. It was concluded that the movement of the east abutment was due to differential settlement and rotation of the pile cap (footing), although it was supported on piles. In 1966 a berm was constructed by infilling a gully adjacent to the east abutment on the north side of the roadway.

Golder visited the site on December 2016, to perform a cursory review of the structure from a foundations perspective. There were no obvious visual signs of foundation-related issues in terms of foundation and embankment instability and settlement at that time.

3.0 PREVIOUS INVESTIGATION

The foundation investigations for the GEOCRESS reports referenced in Section 1.0 were conducted in January and October 1961. During those periods, a skid-mounted drill rig advanced a total of eight boreholes using wash-boring techniques to depths between about 9 m and 32.3 m below the ground surface. Borehole 1 is located east of the structure site and are not relevant to the present desktop study nor shown on the Borehole Locations and Soil Strata.

The foundation investigation report indicates that soil samples were obtained at 0.75 m to 4.5 m depth intervals using 50 mm outside diameter split-spoon samplers driven by manual hammers, in accordance with the Standard Penetration Test (SPT) procedure. In the firm to stiff cohesive deposit thin-walled Shelby tube samples were also taken and in situ field vane testing was conducted to measure the undrained shear strength in the cohesive deposits, Dynamic Cone Penetration Testing (DCPT) was conducted in Borehole 5 from the bottom of the sampled borehole at Elevation 91.2 m to refusal at Elevation 76.7 m, using a 50 mm diameter cone with an energy of 350 ft.lb (475 joules) per blow.

Observations of the water levels in the boreholes were recorded on some boreholes logs; however, piezometers were not installed in any of the boreholes.

Selected samples obtained in from the boreholes were subjected to classification testing, strength testing and consolidation testing and the results are shown on the borehole logs and/or on laboratory test figures attached to the report(s).

The boreholes locations as provided on the borehole record sheets in Station and Off-set were plotted on the General Arrangement drawing (Dwg R2-1, dated Nov. 2016) provided by AECOM on January 18, 2017 and the borehole coordinates were interpreted from the coordinate system superimposed on the plan. The borehole locations in MTM NAD 83 (Zone 10) coordinates, ground surface elevations referenced to Geodetic datum and the drilled depths are as follows:



FOUNDATION REPORT WESTCHESTER AVENUE UNDERPASS HWY 406, ST CATHARINES, ONTARIO G.W.P. 2453-13-00

Borehole No.	MTM NAD83 Northing (m)	MTM NAD 83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m) (DCPT Depth (m))
2	4,779,845.1	326,339.9	102.1	32.3
3	4,779,835.0	326,319.0	101.9	29.4
4	4,779,822.5	326,293.0	102.4	26.4
5	4,779,805.2	326,266.0	103.8	12.6 m (12.6 to 27.1)
6	4,779,793.1	326,263.0	104.2	15.7
7	4,779,800.3	326,293.1	104.1	15.7
8	4,779,814.8	326,321.2	101.1	9.0

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 406 is located within the Iroquois Plain physiographic region, as delineated in the *Physiography of Southern Ontario* (Chapman and Putnam, 1984)¹. The Iroquois Plain extends around the western shores of Lake Ontario and is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession. This site is bound to the north by shoreline beach deposits from Glacial lake Iroquois such as the Homer Bar on which downtown St Catharines is located, and the Niagara Escarpment located some 3 km to the south.

Surficial soil in this area of the Iroquois Plain is typically comprised of silty and clayey till of the Halton Till sheet according to the *Quaternary Geology of the Niagara-Welland Area* (Ontario Geological Survey Map 2496; Feenstra, 1984)². The Halton Till sheet is underlain by an older, red, sandy and silty till, possibly the Wentworth Till sheet (OGS Preliminary Map 764, Feenstra 1972)³. Shallow depressions on the surface of the clay plain upslope of the Homer Bar are infilled with bog sediments while fill materials comprised of earth and rock fill associated with the canal construction occur in the vicinity of the former Welland Canal (OGS Preliminary Map 764, Feenstra 1972).

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes in the vicinity of and pertinent to the Westchester Avenue underpass structure (i.e. Boreholes 2 to 8, inclusive) and the results of in situ and geotechnical laboratory testing, where available from the 1961 and 1962 reports, are given on the borehole records and laboratory test figures contained in Appendix A, following the text of this report. In the discussion below, the depth below ground surface is referenced to the ground surface on the borehole logs at the time of the investigation in 1961 and is not referenced to current ground surface.

¹ 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.

² Feenstra, B.H. 1984. Quaternary Geology of the Niagara-Welland Area. Ontario Geological Survey, Map 2496, Quaternary Geology Series. Scale 1:50,000

³ Feenstra, B.H. 1972. Quaternary Geology of the Niagara Area, Southern Ontario. Ontario Division of Mines, Preliminary Map P.764, Geological Survey. Scale 1:50,000



4.2.1 Fill

Fill was encountered in Boreholes 2, 3, 4 and 8 in the vicinity of the east abutment and approach, and east pier of the underpass at the ground surface (in 1961) extending to depths of between about 4.6 m to 8.5 m below ground surface, fill to between about Elevations 97.5 m and 96.5 m. Fill was also encountered in Borehole 5, advanced between the west and central piers, to Elevation 102.3 m (a depth of 1.5 m below ground surface).

The fill material is described as consisting of silty clay to clay and containing, sand, ashes, fine gravel and organic material.

Standard Penetration Test (SPT) “N”-values in the fill are variable, ranging between about 5 and 20 blows per 0.3 m of penetration, suggesting a firm to very stiff consistency. Undrained triaxial tests on the silty clay fill indicated undrained shear strengths ranging between about 40 kPa and 50 kPa.

Atterberg limits tests were carried out on three (3) samples of the cohesive fill material, and measured plastic limits between 24 per cent and 26 per cent, liquid limits between about 43 per cent and 57 per cent, and plasticity indices between about 17 per cent and 31 per cent. These test results indicates that the fill material is classified as silty clay of intermediate plasticity to clay of high plasticity.

The reported average bulk weight of the cohesive material(s) is about 18.5 kN/m³. The average water content of the fill ranges between about 27 per cent and 30 per cent.

4.2.2 Silty Clay to Clay

Underlying the fill, a deposit of silty clay to clay was encountered in Boreholes 2 to 8 between about Elevations 102.3 m and 93.4 m, and extends to between about Elevations 77.1 m and 74.6 m, where fully penetrated. The silty clay to clay contains pockets of fine sand and silt as well as traces of fine gravel. The authors of the 1961 report characterized this as a native deposit; however, comments in the “Summary of Field & Laboratory Tests” information sheets appended to the 1961 report indicated that glass was extracted from a Shelby tube sample obtained from Borehole 3 at a depth of about 9.1 m to 9.6 m below the ground surface (Elevation 92.8 m to 92.3 m), suggesting that a portion of the deposit may be fill. The deposit as shown in the Records of Borehole sheets is considered to be comprising of two layers / zones: an upper crust of higher consistency; and a lower layer of somewhat lower consistency, as described below.

Upper Weathered Crust – Cohesive Deposit

The cohesive deposit contains an upper weathered crust that extends to between about Elevations 90.8 m and 88.5 m, suggesting that the thickness of the upper crust is 2.6 m thick at Borehole 2 and about 8.5 m thick at Boreholes 3 and 4. The measured SPT “N”-values range from 7 to 40 blows per 0.3 m of penetration. Two in situ field vane tests carried out within the upper crust of the cohesive deposit measured undrained shear strength of 86 kPa and greater than 95 kPa with sensitivities ranging between 2.9 and 10.0. The field vane test results together with the SPT “N”-values indicates that the upper weathered crust of the cohesive deposit has a stiff to hard consistency.

Atterberg limits tests were carried out on twenty-eight (28) samples of the upper crust of the cohesive deposit, and measured plastic limits between about 20 per cent and 27 per cent, liquid limits between about 47 per cent and 63 per cent, and plasticity indices between about 25 per cent and 38 per cent. These test results indicates that this material is silty clay of intermediate plasticity to clay of high plasticity. One Atterberg limits test was carried out on a clayey silt portion of the deposit and measured a plastic limit of 12 percent, a liquid limit of 24 per cent and a



plasticity index of 12 per cent, suggesting that it was a clayey silt of low plasticity. The results of the Atterberg limits testing are presented on Figures A1 and A2 in Appendix A.

Firm to Stiff Cohesive Deposit

Underlying the upper weathered crust, the cohesive deposit is less plastic and was originally classified as a clayey silt to silty clay. In situ field vane shear tests carried out within the cohesive deposit measured undrained shear strengths ranging from about 43 kPa to 86 kPa and the calculated sensitivity varies from about 1.9 to 5.0. Based on the field vane tests the cohesive deposit has a firm to stiff consistency.

Atterberg limits tests were carried out on fifteen (15) samples of the cohesive deposit, and measured plastic limits between about 16 per cent and 23 per cent, liquid limits between about 31 per cent and 56 per cent, and plasticity indices between about 6 per cent and 32 per cent. These test results indicates that this material is a clayey silt of low plasticity to clay of high plasticity. The results of the Atterberg limits testing are presented on Figures A1 and A2 in Appendix A.

Figure A3 presents the envelope of the results of grain size distribution analysis carried out by the DHO.

Undrained triaxial tests were carried out on several samples of the cohesive material(s) from Boreholes 2 and 3; however, it is not clear what depths they were samples obtained from - the results are presented on Figure A4 in Appendix A and the 1961 report indicates that the shear strength varies from 47 kPa to 84 kPa. Laboratory Consolidation tests were conducted on three samples of the silty clay to clay deposit obtained from Boreholes 2 and 3 at Elevations 93.9 m, 88.1 m and 75.6 m and the results are presented on Figure A5 in Appendix A and pertinent parameters are summarized below.

BH/SA Elevation	σ_v' (kPa)	σ_p' (kPa)	OCR	e_o	c_c	c_r
Borehole 3 / SA5 El. 93.9 m	84	95	1.1	0.827	0.260	0.077
Borehole 2 / SA10 El. 75 m	158	316	2.0	0.785	0.365	0.058
Borehole 2 / SA15 El. 88.1	229	364	1.6	0.576	0.295	0.068

Figure A6 presents a summary of the Atterberg limits and accompanying water content test results, in situ vane shear strength test results and the calculated overburden pressure and preconsolidation pressure versus elevation.

The reported bulk weight of the clayey silt to clay deposit ranges from about 18.2 kN/m³ to 20.4 kN/m³. The water content of the clayey silt to clay deposit ranges from about 25 per cent to 41 per cent.

4.2.3 Sandy Clayey Silt Till

A till deposit consisting of sandy clayey silt was encountered underlying the clayey silt to clay deposit in Boreholes 2, 3 and 4, between Elevations 77.1 m and 75.2 m (between a depth of 25.2 m and 26.8 m below ground surface, at the time of the investigation). It is noted that on the borehole logs for Boreholes 2, 3 and 4 and in the "Summary of Field and Laboratory Tests" in Appendix A this material is described as a silty sand glacial till; however, based on the laboratory classification test results discussed below, the description of this deposit has been reinterpreted



as a sandy clayey silt till. Previous foundation investigations carried out by Golder in the vicinity of the Westchester underpass at Highway 406 indicate that a till deposit comprised of clayey silt is present at about Elevation 75 m, which is consistent with the presently reinterpreted classification of this deposit.

The measured SPT “N”-values within the sandy clayey silt till deposit range from about 36 to 95 blows per 0.3 m of penetration, suggesting a hard consistency. In Borehole 5 a DCPT was carried out from the bottom of the sampled borehole at Elevation 91.2 m to Elevation 76.7 m, where the DCPT value measured was 100 blows for 0.3m of penetration. The termination elevation of the DCPT is within the elevation where the sandy clayey silt till deposit was encountered and therefore it is likely that the DCPT terminated in the till deposit.

Grain size distribution analysis were carried out on samples of the sandy clayey silt till deposit and the envelope of the results is presented on Figure A7 in Appendix A.

Atterberg limits tests were carried out on four (4) samples of the cohesive till deposit, and measured plastic limits between about 12 per cent and 13 per cent, liquid limits between about 17 per cent and 20 per cent and plasticity indices between about 5 per cent and 7 per cent. These test results indicates that this material is a (sandy) clayey silt of low plasticity.

The reported bulk weight of four (4) samples of the sandy clayey silt till is about 22.0 kN/m³ and the water content of five samples of the deposit ranges between about 8 per cent and 11 per cent.

4.2.4 Groundwater

The 1961 foundation investigation indicates that groundwater was encountered in Borehole 2 and Borehole 3 at approximate Elevations 100.2 m and 99.9 m, respectively. There was no indication that a piezometer was installed in these boreholes. As such, the groundwater elevations noted in the 1961 reports are inferred to represent the water level immediately after drilling and therefore do not represent stabilized levels.

As discussed in Section 2.0, Highway 406 was constructed in a cut; the water level measured in the boreholes during the 1961 foundation investigation corresponded to an elevation at about the highway grade to about 1 m above the highway grade. At the time of our site visit (December 2016) there was no visible evidence of seepage at the east or west abutment slopes or at the highway level.

The groundwater level is expected to fluctuate seasonally and to be higher during wet periods of the year. The present water level is expected to be lower than at the time of the investigation in 1961.



5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Sandra McGaghran, M.Eng., P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M.A. Costa, P.Eng, a Designated MTO Foundations Contact and a Senior Consultant of Golder, conducted an independent technical and quality control review of this report.

GOLDER ASSOCIATES LTD.



Sandra McGaghran, M.Eng., P.Eng.
Senior Geotechnical Engineer, Associate



Jorge M.A. Costa, P.Eng.
Designated MTO Foundations Contact, Senior Consultant

SMM/JMAC/rb

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.

\\golder.gds\gal\whitby\active\2015\3 proj\1541610 aecom_hwy 406 structures 2014-e-0075\2000 foundation\5 - reports\2 - westchester underpass\final\1541610 fdr 17sept22
westchester.docx



**FOUNDATION REPORT
WESTCHESTER AVENUE UNDERPASS
HWY 406, ST CATHARINES, ONTARIO G.W.P. 2453-13-00**

PART B

**FOUNDATION DESIGN REPORT
WESTCHESTER AVENUE UNDERPASS
STRUCTURE SITE NO. 18-172
HIGHWAY 406 STRUCTURAL REHABILITATIONS
FOURTH AVENUE TO WESTCHESTER AVENUE
ST. CATHARINES, ONTARIO
G.W.P. 2453-13-00**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report presents an assessment of and discussion on foundation resistances and provides foundation engineering recommendations for the proposed rehabilitation of the existing Westchester Avenue underpass. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 1961 and 1962 investigations at the structure site. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the foundation rehabilitation alternatives and carry out the design of the structure foundations, as may be required. This Foundation Investigation and Design Report, discussions and recommendations are intended for the use of the Ministry of Transportation and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Summary of Existing Foundations

The existing bridge is a four-span structure with a total length of approximately 83 m. The October 1961 Foundation Report recommended that the all the foundation units for the structure be supported on large displacement piles driven to practical refusal on/into the dense to very dense silty sand deposit, reinterpreted as sandy clayey silt till – See Section 4.2.3 for discussion. Subsequent to this report, the alignment for Highway 406 was shifted to the west and a subsequent foundation investigation (Boreholes 6, 7 and 8) was completed in January 1962. The January 1962 follow-up letter report suggests that (the sic east abutment) west abutment and piers be supported by conventional concrete strip footings founded on the native stiff silty clay to clay stratum and the east abutment be founded on a pile cap supported on 12 ¾ inch (0.324 m) diameter steel tube piles driven into the stiff clay crust to Elevation 88.4 m (Elev. 290 ft) and recommended a “design load” for each pile of 220 kN (25 tons).

As-built drawings or pile driving notes during construction were not included in any of the documentation obtained from GEOCRE, hence neither the pile capacity nor the pile tip elevation is known. However, the original General Plan (Drawing D4965-1, dated December 1961 – See Appendix B) and the Details of Abutment and Pier Footings (Drawing D-4965-2, dated January 1962 – See Appendix B), were included in the GEOCRE documents and the foundation details shown on the drawings are consistent with the recommendations in the January 1962 follow-up letter. The Elevations and dimensions of the existing strip and spread footings as shown on the General Plan are summarized below.

Footing Location	Top of Footing Elevation	Footing Thickness	Underside of Footing Elevation	Length	Width
West Abutment	101.6 m	0.9 m	100.7 m	22.2 m	2.21 m
Piers	96.6 m	1.1 m	95.5 m	22.6 m	3.05 m



The estimated founding elevations noted above are generally consistent with the recommendations of the January 1962 follow-up letter to the Foundation Investigation Report, to found the footing for each of the piers at Elevation 95.7 m or lower and the (sic East Abutment) West Abutment at Elevation 101.2 m or lower. The foundation design bearing capacities are not indicated on the original drawings, however, the Foundation Investigation Report (GEOCRE 30M3-42) suggests a “design load” of 4,000 psf (about 190 kPa) may be used for design of footing founded in the stiff silty clay crust.

As discussed in Section 2.0, documents obtained from GEOCRE indicate that in 1965 it was reported that there was observed distortion of the bearings and bending of the anchor bolts at the east abutment; however, there was no observed movement at the west abutment. It was concluded that the movement of the east abutment was due to differential settlement and rotation of the pile cap (footing), although it was supported on piles. In 1966 a berm was constructed by infilling a gully adjacent to the east abutment on the north side of the roadway, presumably to provide additional lateral support to the structure.

6.3 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC (2014)) and its Commentary, the proposed underpass structure and foundation system may be classified as having large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a “typical consequence level” associated with exceeding limits states design. In addition, given the desktop nature of the assessment based on the limited level of foundation investigation completed to date (i.e. number of boreholes, borehole depth, laboratory testing that consisted of triaxial tests, consolidations tests, Atterberg limits and grain size distribution), as presented in Sections 3.0 and 4.0 and that some of the boreholes are not positioned on/at the foundations footprints in comparison to the degree of site understanding in Section 6.5 of CHBDC (2014), the level of confidence for design is considered to be between a “low” and “typical” degree of site and prediction model understanding. Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC (2014) have been used for design in Section 6.4 for both a “low” and “typical” degree of site and prediction model understanding for comparison.

6.4 Assessment of Existing Foundations

The geotechnical resistance values are sensitive to the position of the deposits, general level of the groundwater table and to the value of the friction angle used in effective stress analysis or undrained shear strength of the cohesive deposits. The silty clay to clay foundation soils have a high plasticity and are stiff to very stiff in consistency, consequently, the friction angle of the cohesive deposit used in the analyses was estimated with consideration also to our past experience with similar soils at nearby sites. Further, as no piezometers were installed during the 1961 foundation investigation to provide an indication of the long-term water level, we have assumed a water level at Elevation 98.0 m at the west abutment and Elevation 96.5 m at the piers, which corresponds to an inferred water level about 1 m above the piers footing level.

Based on our interpretation of the available information in the GEOCRE reports, and applying the applicable resistance factors from Tables 6.1 and 6.2 of the CHBDC (2014), the factored ultimate geotechnical resistance at Ultimate Limit States (ULS) and the factored serviceability geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement for the west abutment and piers, the footings founded on the stiff silty clay at the approximate elevations given in Section 6.2 are provided below. In accordance with the interpretation of Section



**FOUNDATION REPORT
WESTCHESTER AVENUE UNDERPASS
HWY 406, ST CATHARINES, ONTARIO G.W.P. 2453-13-00**

6.5 of the CHBDC (2014), “low” understanding of the site is defined by “limited representative information (e.g. previous experience, extrapolation from nearby and/or similar sites) combined with conventional prediction models to achieve a lower level of confidence with performance predictions”. Whereas, a “typical” understanding of the site is defined as “typical project-specific investigation procedures and/or knowledge are combined with conventional prediction models to achieve a typical level of confidence with performance predictions”. There is generally one borehole in the vicinity of or on/near the footprint at some of the foundation units, with the exception of the west abutment where no boreholes were previously advanced. During the previous investigation there were in situ vane shear tests carried out in the cohesive deposits and the laboratory analysis and consisting of water contents, grain size distribution analysis, Atterberg limits, triaxial and consolidations tests meets the MTO requirements. The available geotechnical resistances for the “low” and “typical” degree of site understanding are presented below:

Location	West Abutment Footing		Pier Footings		East Abutment (Steel Tube Piles)	
Degree of Site Understanding CHBDC (2014)	Factored Ultimate Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance at SLS (kPa)	Factored Ultimate Geotechnical Resistance at ULS (kPa)	Factored Serviceability Geotechnical Resistance at SLS (kPa)	Factored Ultimate Geotechnical Resistance at ULS (kN)	Factored Serviceability Geotechnical Resistance at SLS (10 mm) (kN)
“Low”	330	210	225	150	250	-- ¹
“Typical”	370	240	250	170	290	--

Note:

1. The factored serviceability geotechnical resistance at SLS (for 10 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS will govern the design.

The geotechnical resistance values provided above are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.4 and C6.7.4 in the CHBDC (2014).

6.5 Summary of Proposed Rehabilitation Options

We understand that geotechnical resistances for the existing foundations (for the strip footings and steel tube piles) at this structure are required for consideration of an alternative rehabilitation design to proceed. We further understand that the rehabilitation works proposed do not envisage the need for new foundations or widening of existing footings to accommodate the anticipated additional design loads, provided that the foundation subsoils have sufficient geotechnical resistances to accommodate the expected load increases from the bridge rehabilitation works.

We understand that the proposed rehabilitation of the existing underpass structure is currently considering five options:

- **Option 1** - holding strategy for 10 to 15 years and includes replacement of expansion joint seals and patch repairs to the existing bridge deck and substructure.
- **Option 2** - considers partial bridge deck replacement in which the existing steel girders will remain in place.



FOUNDATION REPORT

WESTCHESTER AVENUE UNDERPASS

HWY 406, ST CATHARINES, ONTARIO G.W.P. 2453-13-00

- **Option 3** - superstructure replacement with steel girders and concrete deck.
- **Option 4** – superstructure replacement with concrete precast girders and concrete deck.
- **Option 5** - replacement of expansion joints seals, patch repairs to the existing bridge deck and substructure, and replacement of the existing backfill behind the east abutment with Expanded Polystyrene (EPS) backfill.

Based on the updated loading evaluations carried out by AECOM (provided to Golder on July 24, 2017), the estimated foundation resistances for the existing and proposed future bridge loads (and per cent difference) are provided below.

Foundation Unit	Factored Loading / Resistance ¹	Rehabilitation Option				
		1	2	3	4	5
West Abutment Footings	Existing F-ULS ² (kPa)	183				
	Proposed – F-ULS (% increase) (kPa)	183 (0%)	195 (7%)	195 (7%)	231 (26%)	183 (0%)
	Available F-ULS – “Low” ³ / “Typical” ⁴ (kPa)	330 / 370				
	Existing F-SLS ⁵ (kPa)	137				
	Proposed – F-SLS (% increase) (kPa)	137 (0%)	147 (7%)	147 (7%)	177 (29%)	137 (0%)
	Available F-SLS ⁴ “Low” / “Typical” (kPa)	210 / 240				
Pier Footings	Existing F-ULS (kPa)	181				
	Proposed – F-ULS (% increase) (kPa)	181 (0%)	187 (3%)	187 (3%)	227 (25%)	181 (0%)
	Available F-ULS – “Low” / “Typical” (kPa)	225 / 250				
	Existing F-SLS (kPa)	136				
	Proposed – F-SLS (% increase) (kPa)	136 (0%)	141 (4%)	141 (4%)	175 (29%)	136 (0%)
	Available F-SLS “Low” / “Typical” (kPa)	150 / 170				
East Abutment Back Vertical Tube Piles	Existing F-ULS (kN)	292				
	Proposed – F-ULS (% increase) (kN)	292 (0%)	304 (4%)	304 (4%)	338 (16%)	285 (-13%)
	Available F-ULS – “Low” / “Typical” (kN)	250 / 290				
	Existing F-SLS (kN)	217				
	Proposed – F-SLS (% increase) (kN)	217 (0%)	226 (4%)	226 (4%)	255 (18%)	209 (-4%)
	Available F-SLS “Low” / “Typical” (kN)	--/-- ⁶				
East Abutment Front Battered Tube Piles	Existing F-ULS (kN)	328				
	Proposed – F-ULS (% increase) (kN)	328 (0%)	346 (5%)	346 (5%)	403 (23%)	286 (-13%)
	Available F-ULS – “Low” / “Typical” (kN)	250 / 290				
	Existing F-SLS (kN)	245				
	Proposed – F-SLS (% increase) (kN)	245 (0%)	260 (6%)	260 (6%)	307 (25%)	213 (-13%)
	Available F-SLS “Low” / “Typical” (kN)	--/-- ⁶				

Notes:

1. Existing and proposed factored loadings/resistances provided by AECOM
2. F-ULS = Factored Ultimate Geotechnical Resistance at ULS.
3. “Low” indicates Low Degree of Site Understanding as described in Section 6.5 of CHBDC (2014).
4. “Typical” indicates Typical Degree of Site Understanding as described in Section 6.5 of CHBDC (2014).
5. F-SLS = Factored Serviceability Geotechnical Resistance at SLS.
6. The factored serviceability geotechnical resistance at SLS (for 10 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS will govern the design.



As discussed in Section 6.2, the 12 ¾ inch steel tube piles were designed for a load of 220 kN for each pile. Based on the information provided by AECOM the current loading on the east abutment front battered piles exceeds the original “design load” and is at about 80 per cent of the estimated ultimate capacity and therefore there is the potential for undesirable movement. It must be noted that if there is a grade raise to the underpass structure, a similar grade raise will be required at the approach embankments, which will result in additional loadings being imposed on the native subgrade/subsurface soil strata. Such additional loadings could induce differential settlements to occur, especially at the east abutment, potentially leading to rotation of the abutment footing and/or wing walls/stem wall, as reportedly occurred up to 1965 (refer to DOH letter dated November 29, 1965, in Appendix B).

6.6 Geotechnical Assessment of Proposed Rehabilitation Options

A discussion and recommendations on the comparison of the required resistances for the five rehabilitation options proposed by AECOM and the assessed factored ultimate geotechnical resistance at ULS and the factored serviceability geotechnical resistance at SLS, for a “low” and “typical” degree of site understanding, CHBDC (2014) are provided below. As discussed in Section 6.3, taking into account that at some foundation units only one borehole was advanced and also considering the extent of previous geotechnical site investigation and laboratory testing carried out, the site is classified as having a “low” to “typical” degree of site understanding. The per cent increase of the proposed geotechnical resistances for both degrees of site understanding compared to the available geotechnical resistance is provided in the tables below for the various rehabilitation options.

Option 1: It is understood that Option 1 includes replacement of expansion joint seals and patch repairs to the existing bridge deck and substructure resulting in no increase in load to the structure. Detailed below is a comparison of the per cent increase for the required resistance compared to the available factored ultimate geotechnical resistance at ULS and the available factored serviceability geotechnical resistance at SLS, in accordance with CHBDC (2014) for a “low” and “typical” degree of site understanding.

Foundation Unit	Degree of Site Understanding	Option 1: Per Cent Increase in Proposed Loads Compared to Available Resistance CHBDC (2014)	
		Factored Ultimate Geotechnical Resistance at ULS ¹	Factored Serviceability Geotechnical Resistance at SLS
West Abutment	Low	- 45 (no increase)	- 35 (no increase)
	Typical	- 51 (no increase)	- 43 (no increase)
Piers	Low	- 20 (no increase)	- 9 (no increase)
	Typical	- 28 (no increase)	- 20 (no increase)
East Abutment Back Vertical Piles	Low	17	-- ²
	Typical	1	-- ²
East Abutment Front Battered Piles	Low	31	-- ²
	Typical	13	-- ²

Note:

1. Negative values indicate that the proposed (required) resistance is less than the available resistance.
2. The factored serviceability geotechnical resistance at SLS (for 10 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS will govern the design.



FOUNDATION REPORT
WESTCHESTER AVENUE UNDERPASS
HWY 406, ST CATHARINES, ONTARIO G.W.P. 2453-13-00

For this option the proposed loading on the shallow foundations are less than the available geotechnical resistance ULS and SLS values.

The proposed loadings on the back vertical piles at the east abutment exceed the available factored ultimate geotechnical resistances at ULS by 17 per cent for a “low” degree of site understanding and are less than the available geotechnical resistances at SLS. The proposed loadings on the back vertical piles at the east abutment are about the same as the available factored ultimate geotechnical resistance at ULS for a “typical” degree of site understanding and are less than the available geotechnical resistances at SLS. The proposed loadings on the front battered piles at the east abutment exceed the available factored ultimate geotechnical resistances at ULS by about 31 per cent for a “low” degree of site understanding and by about 13 per cent for a “typical” degree of site understanding and both are less than the available geotechnical resistances at SLS. Therefore, the design would have to be based on the factored ultimate geotechnical resistances at ULS.

The existing structure does not show any obvious visual signs of foundation-related issues in terms of foundation and embankment instability and settlement, and although as discussed above the proposed loads at the front battered piles at the east abutment are greater than to the available factored ultimate resistances at ULS for a “low” and “typical” degree of site understanding, it is considered that Option 1 may proceed; provided that the rigidity and area of the existing east abutment foundations or the available resistance are augmented (i.e. increased) by an alternative support / foundation system such as underpinning by the use of micropiles. As an alternative, the existing earth fill behind the east abutment wall could be replaced with light-weight fill to reduce the loadings on the foundations.

Options 2 and 3: The per cent increase for the proposed load compared to the existing loading for rehabilitation Options 2 and 3 is in the order of between 3 per cent and 7 per cent depending on the foundation element. Detailed below is a comparison of the per cent increase for the required resistance compared to the available factored ultimate geotechnical resistance at ULS and the available factored serviceability geotechnical resistance at SLS, in accordance with CHBDC (2014) for a “low” and “typical” degree of site understanding.

Foundation Unit	Degree of Site Understanding	Options 2 and 3: Per Cent Increase in Proposed Loads Compared to Available Resistance CHBDC (2014)	
		Factored Ultimate Geotechnical Resistance at ULS ¹	Factored Serviceability Geotechnical Resistance at SLS
West Abutment	Low	-41 (no increase)	-30 (no increase)
	Typical	-47 (no increase)	-39 (no increase)
Piers	Low	-17 (no increase)	-6 (no increase)
	Typical	-25 (no increase)	-17 (no increase)
East Abutment Back Vertical Piles	Low	22	-- ²
	Typical	5	-- ²
East Abutment Front Battered Piles	Low	38	-- ²
	Typical	19	-- ²

Note:

1. Negative values indicate that the proposed (required) resistance is less than the available resistance
2. The factored serviceability geotechnical resistance at SLS (for 10 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS will govern the design.



FOUNDATION REPORT
WESTCHESTER AVENUE UNDERPASS
HWY 406, ST CATHARINES, ONTARIO G.W.P. 2453-13-00

For this option the proposed loading on the shallow foundations are less than the available geotechnical resistances ULS and SLS values.

The proposed loadings on the back vertical piles at the east abutment exceed the available factored ultimate geotechnical resistances at ULS by 22 per cent and 5 per cent for a “low” and “typical” degree of site understanding, respectively, and are less than the available geotechnical resistances at SLS. Similarly, the proposed loadings on the front battered piles at the east abutment also exceed the available factored ultimate geotechnical resistances at ULS by 38 per cent and 19 per cent for a “low” and “typical” degree of site understanding, respectively, but similarly are less than the available geotechnical resistances at SLS, therefore the design would have to be based on the factored ultimate geotechnical resistances at ULS.

If consideration is given to adopting Option 2 or 3 an additional foundation investigation of sufficient scope could be carried out to obtain a “high” degree of site understanding as per Table 6.2 of CHBDC (2014). However, the increase in factored ultimate geotechnical resistance obtained from a “high” degree of site understanding will not be sufficient enough to increase the geotechnical resistance to the proposed loadings and therefore consideration should instead be given to increasing the rigidity and area of the existing east abutment foundations or augmenting the available resistances by an alternative support / foundation system such as underpinning by the use of micropiles. As an alternative, the existing earth fill behind the east abutment wall could be replaced with light-weight fill to reduce the loadings on the foundations.

Option 4: As noted in Section 6.5, the per cent increase in the proposed load compared to the existing loading for rehabilitation Options 4 is in the order of between 16 per cent and 29 per cent. Detailed below is a comparison of the per cent increase in the proposed factored geotechnical resistances compared to the available factored ultimate geotechnical resistance at ULS and the factored serviceability geotechnical resistance at SLS for a “low” and “typical” degree of site understanding.

Foundation Unit	Degree of Site Understanding	Option 4: Per Cent Increase in Proposed Loads Compared to Available Resistance CHBDC (2014)	
		Factored Ultimate Geotechnical Resistance at ULS ¹	Factored Serviceability Geotechnical Resistance at SLS
West Abutment	Low	-30 (no increase)	-16 (no increase)
	Typical	-38 (no increase)	-26 (no increase)
Piers	Low	1 (slight increase)	17
	Typical	-9 (no increase)	3
East Abutment Back Vertical Piles	Low	35	-- ²
	Typical	17	-- ²
East Abutment Front Battered Piles	Low	61	-- ²
	Typical	39	-- ²

Note:

1. Negative values indicate that the proposed (required) resistance is less than the available resistance
2. The factored serviceability geotechnical resistance at SLS (for 10 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS will govern the design.

For this option the proposed loading on the shallow foundations are less than the available factored ultimate geotechnical resistances ULS at the west abutment and the pier, and are less than the available geotechnical



FOUNDATION REPORT
WESTCHESTER AVENUE UNDERPASS
HWY 406, ST CATHARINES, ONTARIO G.W.P. 2453-13-00

resistances at SLS at the west abutment. At the pier the proposed loading exceeds the available geotechnical resistances at SLS by 17 per cent and 3 per cent at SLS for a “low” and “typical” degree of site understanding, respectively.

The proposed loadings on the back vertical piles at the east abutment are 35 per cent and 17 per cent higher (for a “low” and “typical” degree of site understanding), respectively, than the available factored ultimate geotechnical resistances at ULS and are less than the available geotechnical resistances at SLS. However, the proposed loadings on the front battered piles at the east abutment greatly exceed the available factored ultimate geotechnical resistances at ULS by 61 per cent and 39 per cent for a “low” and “typical” degree of site understanding, respectively, but are less than the available geotechnical resistances at SLS. Therefore the design would have to be based on the factored ultimate resistances at ULS in either case of “low” or “typical” degree of site understanding.

Considering that the per cent increase in loadings for Option 4 vary from 16 per cent to 29 per cent, the anticipated increase in available geotechnical resistance from carrying out an additional foundation investigation of sufficient scope to obtain a “high” degree of site understanding, as per Table 6.2 of CHBDC (2014), will likely indicate that the available factored ultimate resistances at ULS would still be much less than the proposed loadings. Therefore, consideration should instead be given to increasing the rigidity and area of the existing foundations (shallow and deep) or augmenting the available resistances by an alternative support / foundation system such as underpinning by the use of micropiles. As an alternative, the existing earth fill behind the east abutment wall could be replaced with light-weight fill to reduce the loadings on the foundations.

Option 5: This option is similar to Option 1, but this option considers the removal of the existing fill material behind the east abutment and replacement with EPS in order to reduce the loading on the front battered piles. Option 5 results in no increase in load to the structure at the west abutment and pier foundation units. At the east abutment Option 5 results in a reduction of the proposed loading from between 4 per cent and 13 per cent. Detailed below is a comparison of the per cent increase for the proposed factored geotechnical resistances compared to the available factored ultimate geotechnical resistance at ULS and the factored serviceability geotechnical resistance at SLS for a “low” and “typical” degree of site understanding.

Foundation Unit	Degree of Site Understanding	Option 5: Per Cent Increase in Proposed Loads Compared to Available Resistance CHBDC (2014)	
		Factored Ultimate Geotechnical Resistance at ULS ¹	Factored Serviceability Geotechnical Resistance at SLS
West Abutment	Low	-45 (no increase)	-35 (no increase)
	Typical	-51 (no increase)	-43 (no increase)
Piers	Low	-20 (no increase)	-9 (no increase)
	Typical	-28 (no increase)	-20 (no increase)
East Abutment Back Vertical Piles	Low	14	-- ²
	Typical	-2 (no increase)	-- ²
East Abutment Front Battered Piles	Low	14	-- ²
	Typical	-1 (no increase)	-- ²

Note:

1. Negative values indicate that the proposed (required) resistance is less than the available resistance
2. The factored serviceability geotechnical resistance at SLS (for 10 mm of settlement) is greater than the factored ultimate geotechnical resistance at ULS therefore the ULS will govern the design.



**FOUNDATION REPORT
WESTCHESTER AVENUE UNDERPASS
HWY 406, ST CATHARINES, ONTARIO G.W.P. 2453-13-00**

For this option the proposed loading on the shallow foundations (west abutments and piers) are less than the available geotechnical resistance ULS and SLS values.

The proposed loadings on the back vertical piles and front battered at the east abutment are 14 per cent higher than the available factored ultimate geotechnical resistances at ULS for a “low” degree of site understanding. For a “typical” degree of site understanding, the proposed loadings are about the same as the available factored ultimate geotechnical resistance at ULS. The proposed loadings on the back vertical piles and front battered are less than the available geotechnical resistances at SLS for a “low” and “typical” degree of site understanding, therefore the design would have to be based on the factored ultimate resistances at ULS.

If Option 5 is adopted, it is recommended that the existing backfill behind the east abutment be excavated and replaced with EPS light-weight fill placed over the full width of the pile cap behind the abutment wall stem. The existing fill behind the lightweight fill zone should be sloped up and back from the back edge of the pile cap at 1.5 horizontal to 1 vertical (1.5H:1V), to minimize the load transmitted through the light weight fill onto the abutment pile cap and wall.

The EPS fill should be installed in accordance with the manufacturer’s requirements and EPS should be keyed/stepped into the cut slope behind the abutment. It is recommended that a minimum of 300 mm thick levelling pad/drainage layer, comprised of 200 mm of Granular ‘B’ Type II and 100 mm of sand, be placed on the stepped cut slope prior to the installation of the EPS. Some form of positive drainage behind and below the EPS fill mass (and behind the abutment wall) must be provided to drain any perched water that infiltrates through the roadway surface. The following design parameters may be used in assessing the loadings on the abutment piles:

Fill Type	Unit Weight (kN/m ³)	At Rest Earth Pressure Coefficient
EPS	0.5	0.20

The EPS is susceptible to degradation from exposure to sunlight and contact with hydrocarbon (i.e. fuel spills), therefore it is recommended that the EPS be protected by wrapping it in a 10 mil (0.25 mm) polyethylene sheet with a minimum of 300 mm overlap at the joints.

A reinforced concrete slab to be designed by the structural engineer will be required on top of the full zone of the EPS for distribution and protection from traffic loads.

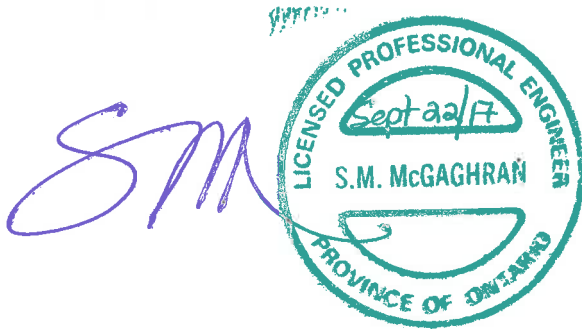


**FOUNDATION REPORT
WESTCHESTER AVENUE UNDERPASS
HWY 406, ST CATHARINES, ONTARIO G.W.P. 2453-13-00**

7.0 CLOSURE

This report was prepared by Sandra McGaghran, M.Eng., P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Jorge M.A. Costa, P.Eng., a Designated MTO Foundations Contact and Senior Consultant of Golder, conducted an independent technical and quality control review of this report. We trust the above information meets with your current requirements; should you have any questions, please do not hesitate to contact us.

GOLDER ASSOCIATES LTD.



Sandra McGaghran, M.Eng., P.Eng.
Senior Geotechnical Engineer, Associate

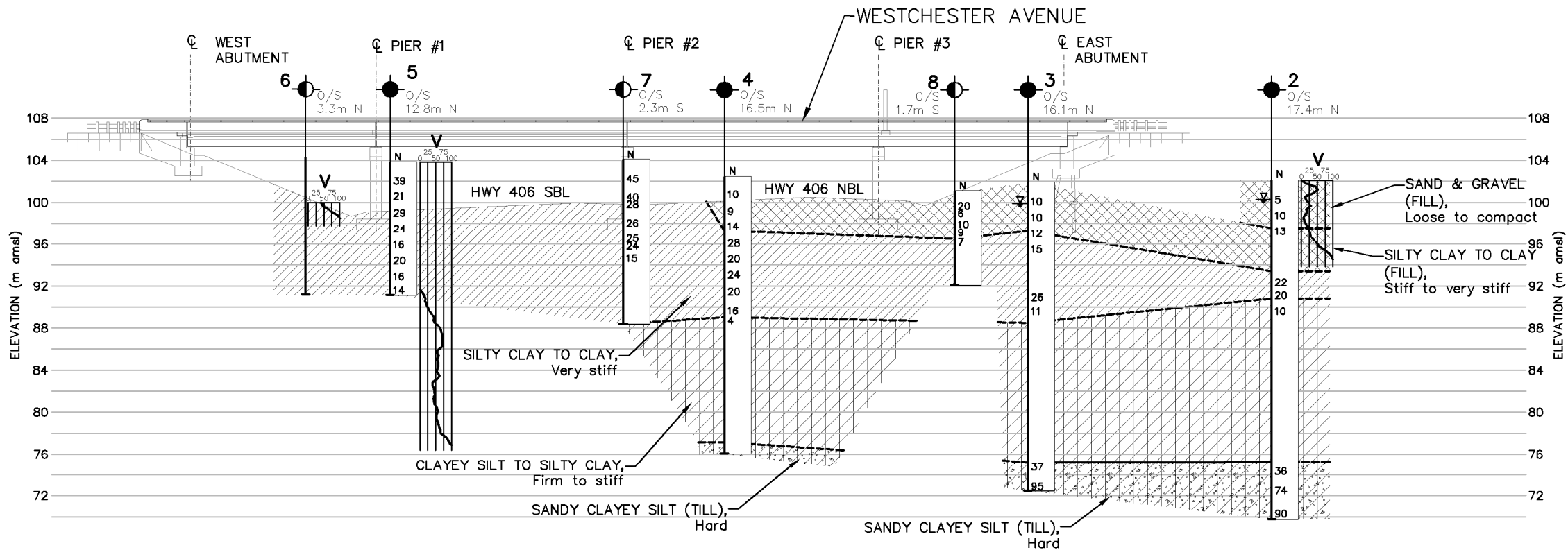
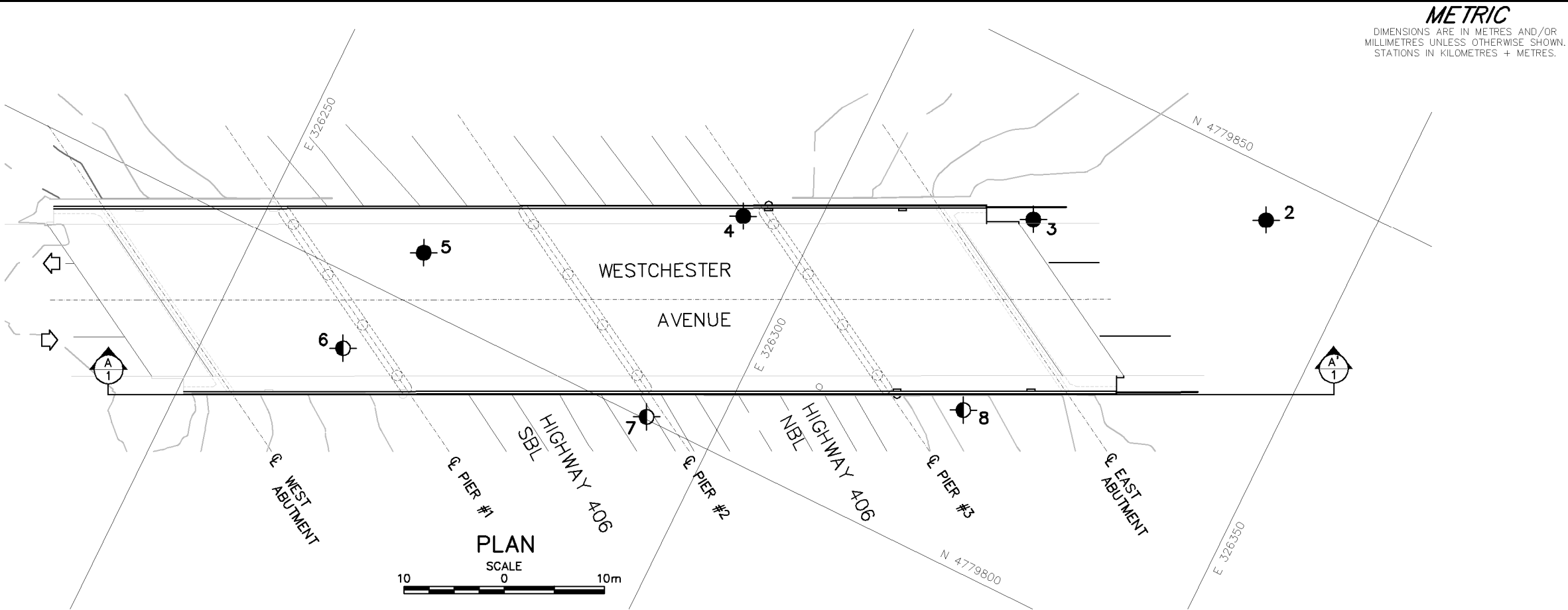


Jorge M.A. Costa, P.Eng.
Designated MTO Foundations Contact, Senior Consultant

SMM/JMAC/rb

Golder, Golder Associates and the GA globe design are trademarks of Golder Associates Corporation.

\\golder.gds\gal\whitby\active\2015\3 proj\1541610 aecom_hwy 406 structures 2014-e-0075\2000 foundation\5 - reports\2 - westchester underpass\final\1541610 fdr 17sept22
westchester.docx



PROFILE ALONG SOUTH EDGE OF WESTCHESTER AVENUE BRIDGE

HORIZONTAL & VERTICAL SCALE
10 0 10m



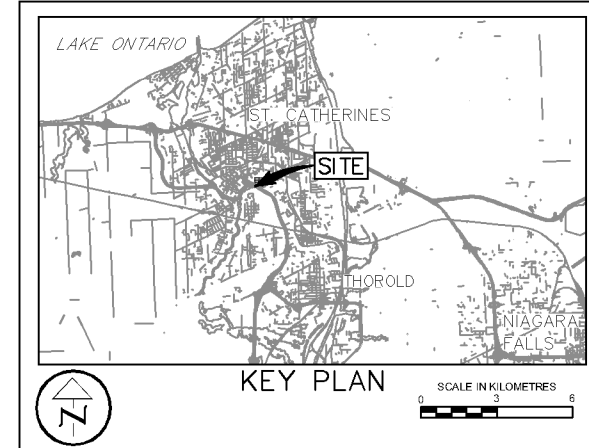
CONT No.
GWP No. 2453-13-00



HIGHWAY 406
WESTCHESTER AVENUE UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA



Golder Associates Ltd.
LONDON, ONTARIO, CANADA



LEGEND

- Borehole GEOCRE 30M3-42 (October 1961)
- Borehole GEOCRE 30M3-42 (January 1962)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- V Insitu Vane Test
- Water Level on completion or during drilling

No.	ELEVATION	BOREHOLE CO-ORDINATES (MTM ZONE 10)	
		NORTHING	EASTING
2	102.1	4 779 845.1	326 339.9
3	101.9	4 779 835.0	326 319.0
4	102.4	4 779 822.5	326 293.0
5	103.8	4 779 805.2	326 266.0
6	104.2	4 779 793.1	326 263.0
7	104.1	4 779 800.3	326 293.1
8	101.1	4 779 814.8	326 321.2

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

REFERENCE

The borehole locations as provided on the borehole record sheets in Station and Off-set were plotted on the General Arrangement drawing (Dwg R2-1, dated Nov. 2016) provided by AECOM on January 18, 2017 and the borehole coordinates were interpreted from the coordinate system superimposed on the plan. The borehole locations are approximate. AECOM, Drawing file No. R2-01_WestChester Ave. Underpass_GA.dwg, received January 18/17;

D.H.O Foundation Investigation Report (W.J. 60-F-100, W.P. 47-61, Site No. 18-172, dated October 3/61 (GEOCRE 30M03-042); and

D.H.O Foundation Investigation Report (W.J. 60-F-100(B), W.P. 47-61, Site No. 18-172, dated January 8/62 (GEOCRE 30M03-042).

NO.	DATE	BY	REVISION
Geocres No. 30M3-298			
HWY.	HIGHWAY 406	PROJECT NO.	1541610
SUBM'D.	SMM	CHKD.	SMM
DRAWN:	DCH	APPD.	JMAC
DATE: Sept. 20/17		SITE: 18-172	
DIST. Central		DWG. 1	



APPENDIX A

GEOCRE 30M3-42 Record of Borehole Sheets and Laboratory Data

OFFICE REPORT ON SOIL EXPLORATION

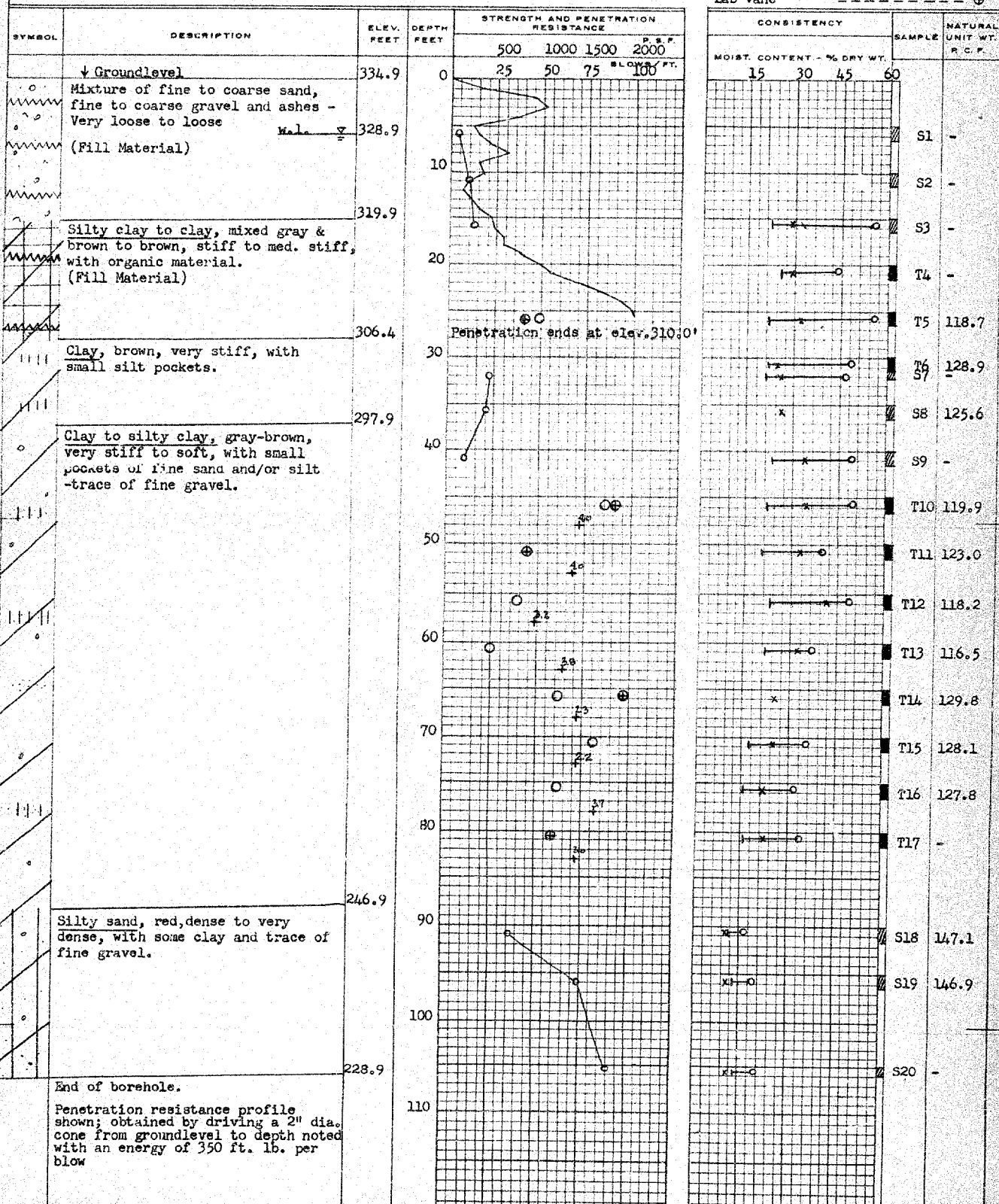
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 47-61 BORE HOLE NO. 2
 JOB 60-F-100 STATION 254+64 - 98' Rt.
 DATUM 334.9' COMPILED BY B.K.
 BORING DATE Dec. 15/60 CHECKED BY G.G.C.

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

LEGEND

Triaxial compression - - - - -
 1/2 UNCONFINED COMPRESSION (Qu) - - - - -
 VANE TEST (C) AND SENSITIVITY (S) - - - - -
 NATURAL MOISTURE AND LIQUIDITY INDEX - - - - -
 LIQUID LIMIT - - - - -
 PLASTIC LIMIT - - - - -
 Lab vane - - - - -



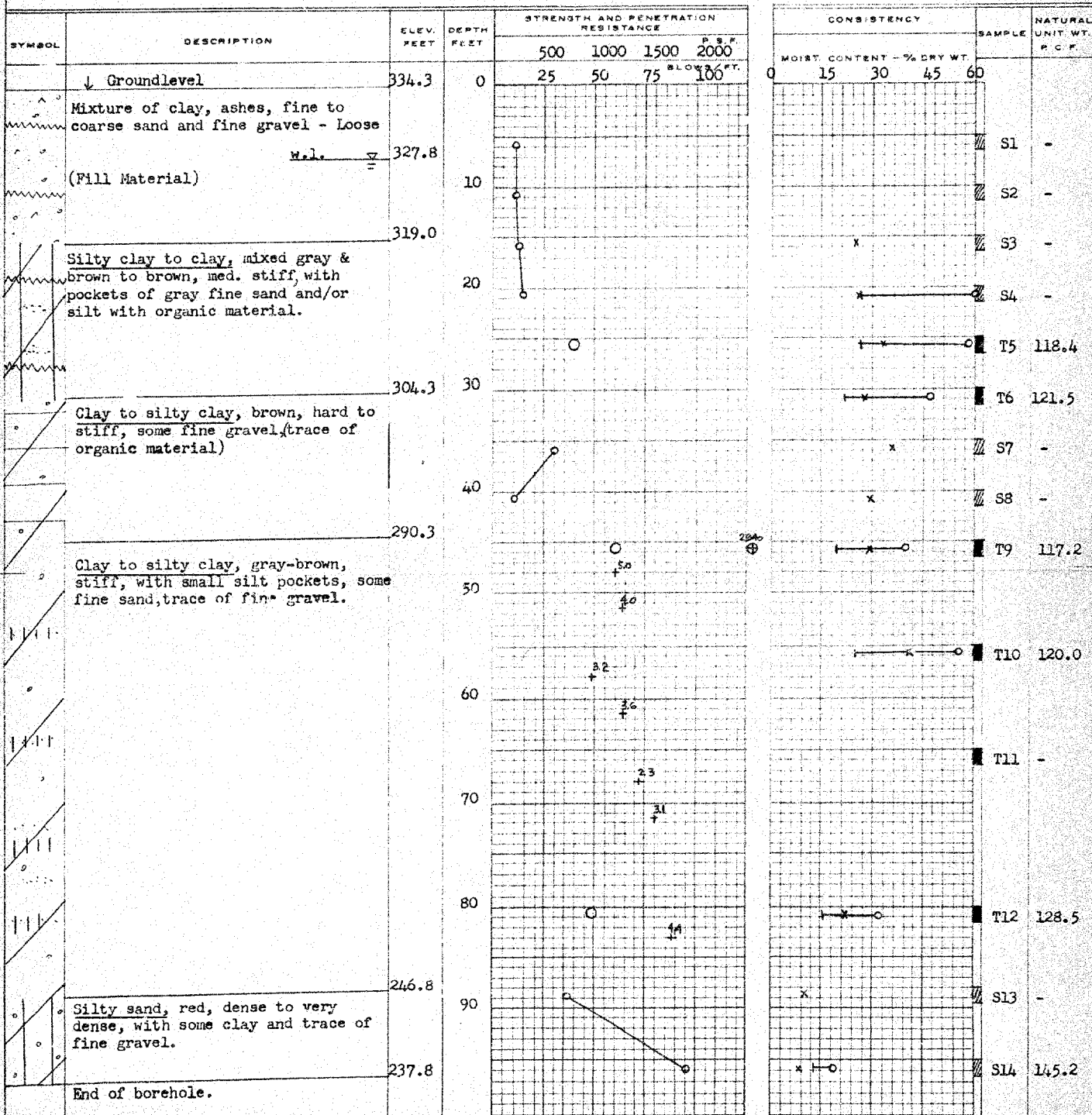
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 47-61 _____ BORE HOLE NO. 3
 JOB 60-F-100 _____ STATION (OnTangent)
 DATUM 334.3' _____ 274+83 24' Rt.
 BORING DATE Dec. 21/60 _____ COMPILED BY B.K.
 _____ CHECKED BY G.G.C.

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

LEGEND

2	Triaxial compression - - - -	0
1/2	Uniaxial compression (Qu) - - - -	5
	VANE TEST (C) AND SENSITIVITY (S) - - - -	+5
	NATURAL MOISTURE - - - -	10
	LIQUIDITY INDEX - - - -	X
	LIQUID LIMIT - - - -	15
	PLASTIC LIMIT - - - -	20
	Lab vane - - - -	25



W.P. 47-61 BORE HOLE NO. 4
(OnTangent)
JOB 60-F-100 STATION 255+07 - 70' Lt.
DATUM 336.1' COMPILED BY B.K.
BORING DATE Jan. 3/61 CHECKED BY G.G.C.

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

Lab vane - - - - -
1/2 UNCONFINED COMPRESSION (QU) - - - - -
VANE TEST (C) AND SENSITIVITY (S) - - - - -
NATURAL MOISTURE AND
LIQUIDITY INDEX - - - - -
LIQUID LIMIT - - - - -
PLASTIC LIMIT - - - - -
Triaxial compr - - - - -

[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 47-61

BORE HOLE NO. 5

JOB 60-F-100

(On Tangent)
STATION 255+24 175' Lt.

DATUM 340.6'

COMPILED BY B.K.

BORING DATE Jan. 6/61

CHECKED BY G.G.C.

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

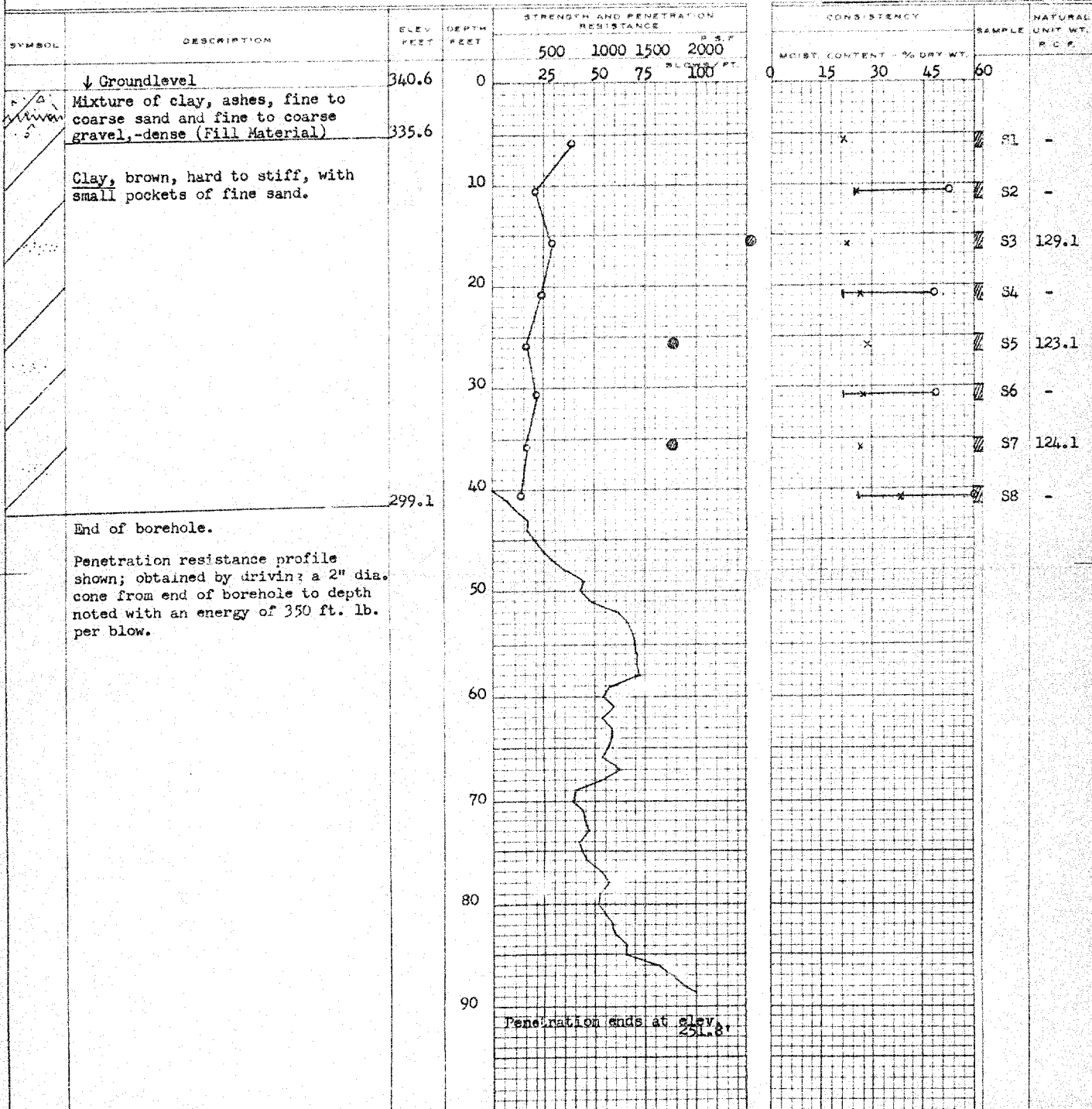
2" DIA. CONE

2" SHELBY

CASING

LEGEND

1/2 UNCONFINED COMPRESSION (QU) \odot
 VANE TEST (C) AND SENSITIVITY (S) \oplus
 NATURAL MOISTURE AND LIQUIDITY INDEX \times
 LIQUID LIMIT \bigcirc
 PLASTIC LIMIT \square
 1/2 Triaxial compression - - - \bigcirc



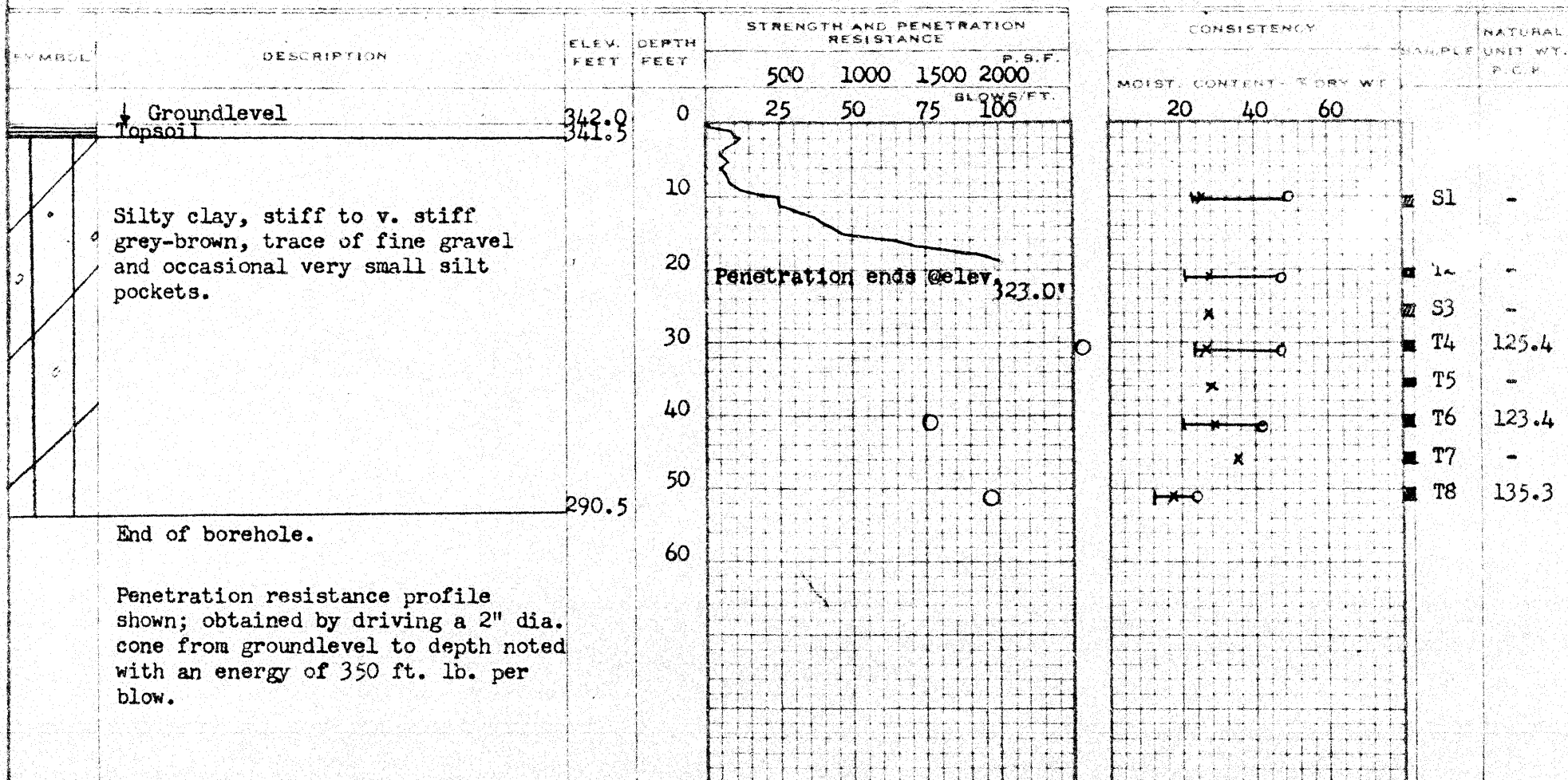
DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 47-61 BORE HOLE NO. 6
 JOB 60-F-100B STATION 12+89 (16' Rt.)
 DATUM 342.0' COMPILED BY B.K.
 BORING DATE Oct. 26/61. CHECKED BY

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



DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 47-61

BORE HOLE NO. 7

JOB 60-F-100B

STATION 13+90 (39' Rt.)

DATUM 341.5'

COMPILED BY B.K.

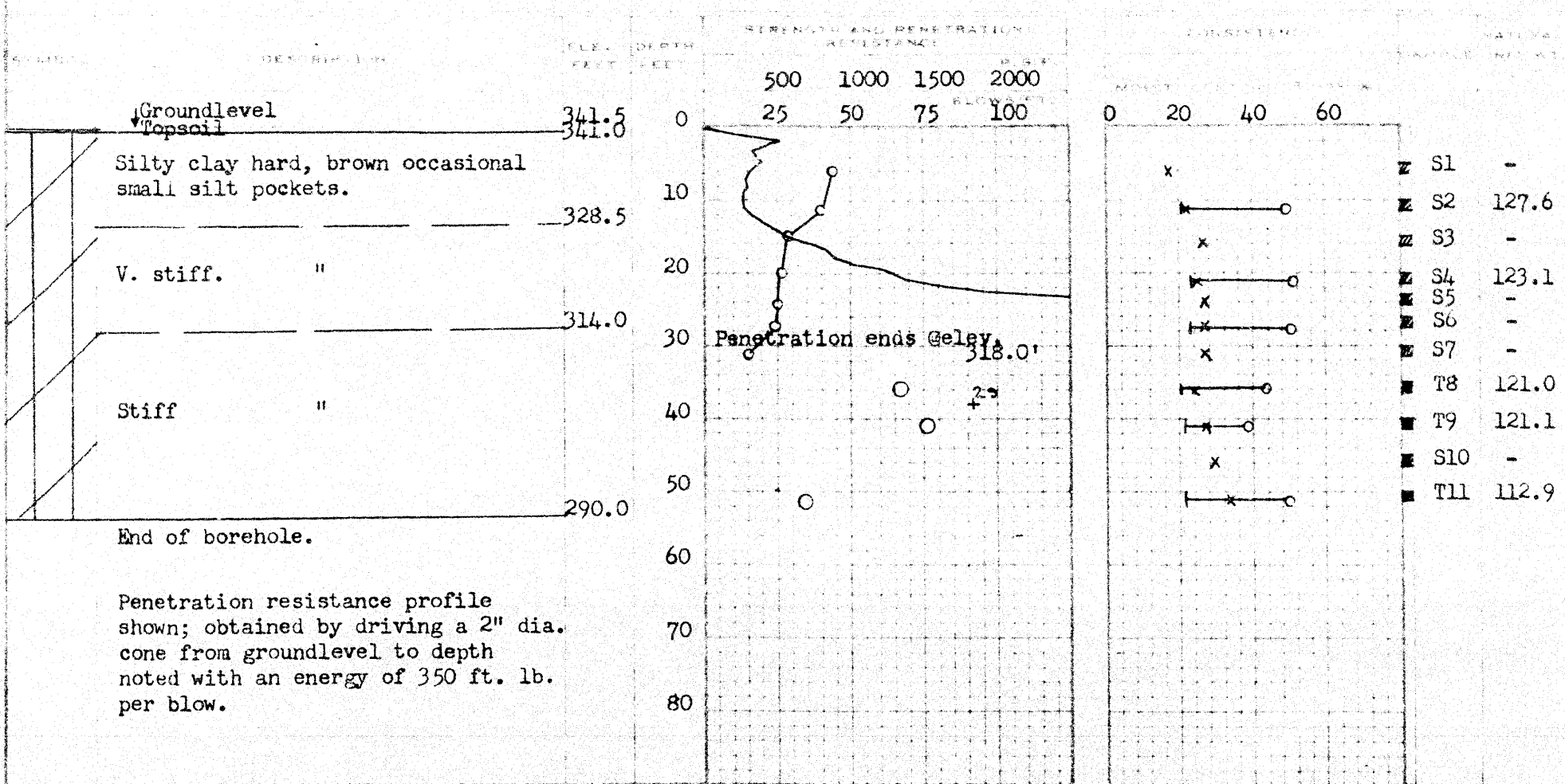
BORING DATE Oct. 27/61.

CHECKED BY

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

LEGEND

UNCONFINED COMPRESSION (QU) ○
 VANE TEST (C AND SENSITIVITY) x
 NATURAL MOISTURE AND LIQUIDITY INDEX □
 LIQUID LIMIT —
 PLASTIC LIMIT —



DEPARTMENT OF HIGHWAYS - ONTARIO MATERIALS AND RESEARCH SECTION

W.P. 47-61

BORE HOLE NO. 8

JOB 60-F-100B

STATION 14+95 (37' Rt.)

DATUM 331.6'

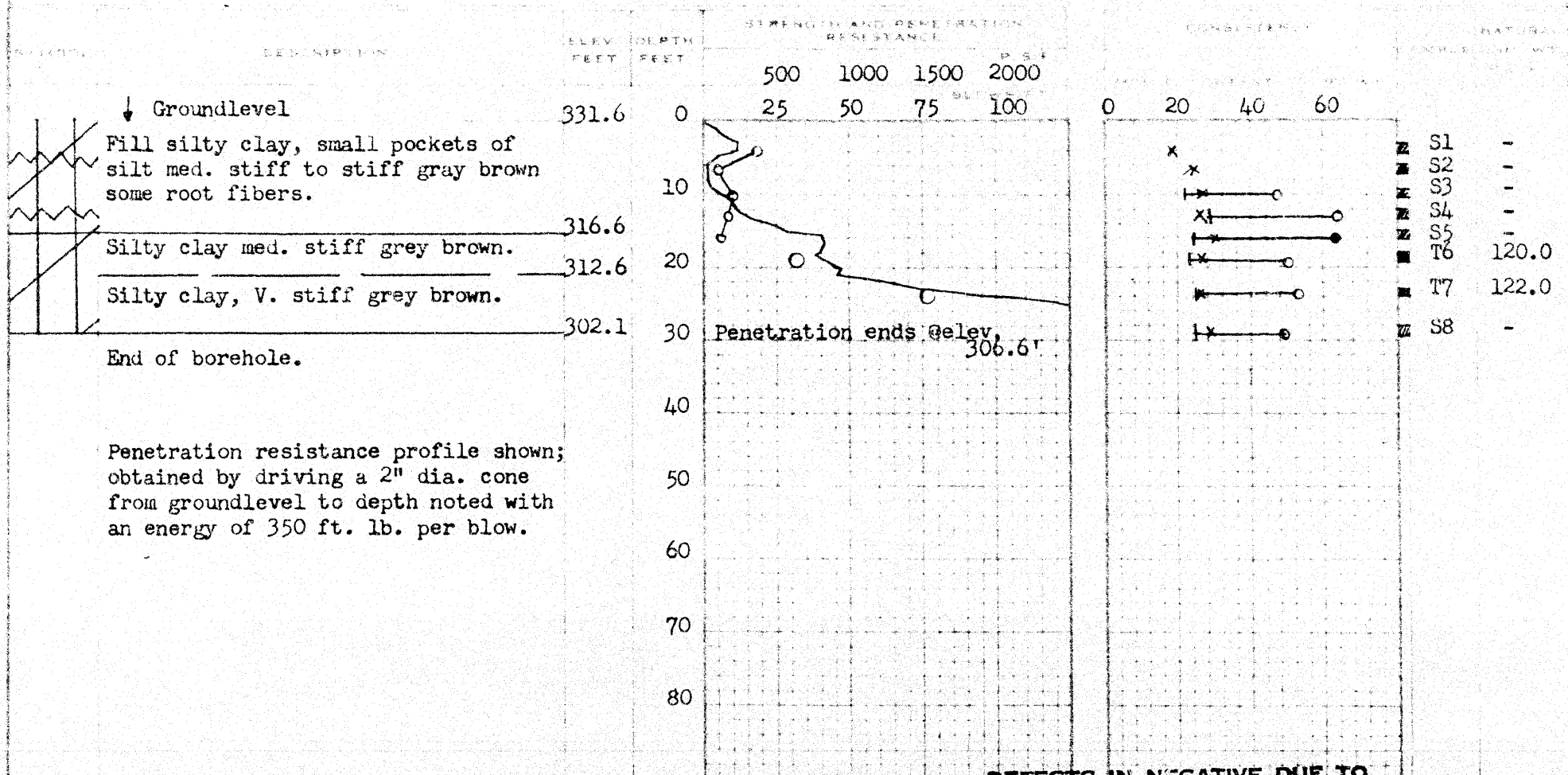
COMPILED BY B.K.

BORING DATE Oct. 23/61. CHECKED BY

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

LEGEND

UNCONSOLIDATED COMPRESSION (QU)
 VANE TEST (C) AND SENSITIVITY (S)
 NATURAL MOISTURE AND
 LIQUID LIMIT
 PLASTIC LIMIT



SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-100

W.P. 47-61

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
1	S1	5'-6.5'	Loose mixture of clay, ashes, sand, and fine to coarse gravel.	13	-	-	-	-	-	
	S2	10'-11.0' 11'-11.5'	As above. Clay, brown, very stiff, flaky.	24	25.6	-	-	-	-	
	S3	11.5'-13'	Clay, brown, hard, flaky.	34	-	-	-	-	-	
2	S1	5'-6.5'	Very loose mixture of ashes, sand and fine to coarse gravel.	4	-	-	-	-	-	
	S2	10'-11.5'	Loose mixture of fine to coarse sand and fine gravel.	9	-	-	-	-	-	
	S3	15'-16.5'	Silty clay, mixed gray & brown, stiff, with trace of fine gravel.	12	28.8	23.6	55.5	-	-	
	T4	20'-21.5'	Silty clay, brown, stiff with some fine sand and organic material.	Levered	29.7	25.8	43.4	-	-	
	T5	25'-26.5'	Clay, brown, med. stiff, with seams of silty clay and some organic material.	Levered	31.7	22.2	55.2	TR=975	118.7	
	VANE	28'	As above.	-	-	-	-	V=852 >2000	-	Sens: >10.0
	T6	30'-30.8'	Clay, brown, very stiff, flaky.	Levered 7 1/2" S/1 1/2"	24.5	22.2	48.3	-	128.9	Fine roots present in sample.
	S7	30.8'-32.3'	As above.	25	25.7	21.0	46.5	-	-	
	S8	35'-36.5'	Clay, brown, very stiff, flaky, with small silt pockets.	20	25.7	-	-	-	125.6	

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-100

W.P. 47-61

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
2	S9	40'-41.5'	Clay, gray-brown, very stiff, with small silt pockets.	8	33.5	23.2	48.6	-	-	
	VANE	43.5'	Clay gray-brown, very stiff, flaky.	-	-	-	-	>2000	-	Sens: >5.0
	T10	45'-46.5'	Clay gray-brown, stiff, flaky with seams of clay silt.	Levered	33.8	22.0	49.6	TR=1700 V=1810	119.9	
	VANE	48'	Clay, gray-brown, stiff.	-	-	-	-	1440	-	Sens: 4.0
	T11	50'-51.5'	Clay, gray-brown, med. stiff, with silt pockets.	Levered	32.7	20.5	39.7	V=866	123.0	
	VANE	53'	Clay, gray-brown, stiff.	-	-	-	-	1360	-	Sens: 4.0
	T12	55'-56.5'	Clay, gray-brown, med. stiff, flaky, with small silt pockets.	Levered	41.0	23.1	48.3	TR=770	118.2	
	VANE	58'	Clay, gray-brown, med. stiff.	-	-	-	-	960	-	Sens: 3.2
	T13	60'-61.5'	Silty clay, gray-brown, soft, with silt pockets.	Pushed	32.2	22.0	37.2	TR=480	116.5	
	VANE	63'	Clay, gray-brown, stiff, with layers of very silty clay.	-	-	-	-	1280	-	Sens: 3.8
	T14	65'-66.5'	Silty clay, gray-brown, stiff, with pockets of fine sand and trace of fine gravel.	Pushed	25.2	-	-	TR=1230 V=1935	129.8	Fragments of red shale in sample.

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-100

W.P. 47-61

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH P.S.F.	UNIT WEIGHT P.C.F.	REMARKS
2	VANE	68'	Silty clay, gray-brown, stiff, with trace of fine gravel and fragments of red shale.	-	-	-	-	1440	-	Sens: 2.3 Fragments of red shale in sample.
	T15	70'-71.5'	Silty-clay, gray-brown, stiff, with trace of fine sand & fine gravel-fragments of red shale.	Levered	25.0	17.8	35.8	TR=1620	128.1	
	VANE	73'	As above.	-	-	-	-	1440	-	Sens: 2.2
	T16	75'-76.5'	" "	Levered	22.2	15.6	30.9	TR=1230	127.8	
	VANE	78'	" "	-	-	-	-	1640	-	Sens: 3.7
	T17	80'-81.5'	" "	Levered	22.7	16.5	33.6	V=1168	-	
	VANE	83'	" "	-	-	-	-	1440	-	Sens: 3.0
	S18	90'-91.5'	Silty sand, red-brown, dense with some clay and trace of fine gravel.	36	10.8	12.2	17.0	-	147.1	Glacial Till.
	S19	95'-96.5'	Silty sand, red-brown, very dense with some clay and trace of fine gravel.	74	10.2	12.3	18.9	-	146.9	Glacial Till.
	S20	105'-106'	As above.	90	10.6	13.1	20.1	-	-	Glacial Till.

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-100

W.P. 47-61

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH PSI	UNIT WEIGHT PCF	REMARKS
3	S1	5'-6.5'	Loose mixture of clay, sand, ashes, and some fine gravel.	11	-	-	-	-	-	
	S2	10'-11.5'	As above.	11	-	-	-	-	-	
	S3	15'-15.25 15.25-16.5'	As above. Clay, mixed gray & brown, with fine sand and fine gravel. (trace of organic material)	12	25.5	-	-	-	-	
	S4	20'-21.5'	Silty clay, brown, med. stiff, with trace of fine sand & fine gravel.	14	26.2	27.1	60.1	-	-	Fine roots present in sample.
	T5	25'-26.5'	Clay, brown, med. stiff, with pockets of gray fine sand & silt. (trace of organic material)	Levered	33.0	26.1	58.0	TR=790	118.4	
	T6	30'-31.25'	Clay, brown, hard, with fine gravel.	Levered	27.2	21.3	46.8	-	121.5	Pieces of broken glass in sample.
	S7	35'-36.5'	Clay, brown, hard, mixed with decayed wood fibers.	30	35.4	-	-	-	-	
	S8	40'-41.5'	Silty clay, brown, stiff.	10	29.8	-	-	-	-	Fine roots present in sample.
	T9	45'-46.5'	Clay, gray-brown, stiff, with small silt pockets.	Levered	29.6	19.8	39.2	TR=1200 V=2940	117.2	
	VANE	48'	As above.	-	-	-	-	1200	-	Sens: 5.0
	VANE	51.5'	As above.	-	-	-	-	1280	-	Sens: 4.0

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-100

W.P. 47-61

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
3	T10	55'-56.5'	Clay, gray-brown, stiff, with small silt pockets.	Pushed	41.1	24.5	55.8	-	120.0	
	VANE	58'	As above.	-	-	-	-	960	-	Sens: 3.2
	VANE	61.5'	As above.	-	-	-	-	1280	-	Sens: 3.6
	T11	65'-66.5'	Clay, gray-brown, stiff, with some silt and fine sand. - trace of fine gravel.	Levered	-	-	-	-	-	
	VANE	68'	As above.	-	-	-	-	1440	-	Sens: 2.3
	VANE	71.5'	As above.	-	-	-	-	1600	-	Sens: 3.1
	T12	80'-81.5'	Silty clay, gray-brown, stiff, with some fine sand.	Levered	22.2	16.2	32.1	TR=995	128.5	
	VANE	83'	As above.	-	-	-	-	1760	-	Sens: 4.4
	S13	88'-89.5'	Silty sand, red-brown, dense, with some clay.	37	9.6	-	-	-	-	Glacial Till.
	S14	95'-96.5'	Silty sand, red-brown, very dense, with some clay.	95	8.6	12.1	19.6	-	145.2	Glacial Till.

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-100

W.P. 47-61

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
4	S1	5'-6.5'	Loose mixture of clay, sand, ashes & organic material.	10	27.4	-	-	-	-	
	S2	10'-11.5'	Clay, mixed gray & brown, med. stiff containing organic material.	9	30.0	25.9	56.8	-	-	
	S3	15'-16.5'	As above.	14	-	-	-	-	-	
	S4	20'-21.5'	Clay, brown, very stiff, flaky, with small pockets of sand & silt.	29	25.0	23.1	50.3	U=2210	128.1	
	S5	25'-26.5'	Clay, brown, very stiff, flaky, with small pockets of fine sand.	20	25.0	-	-	-	-	
	S6	30'-31.5'	Clay, brown, very stiff, flaky, with trace of fine sand & fine gravel.	23	26.0	21.6	47.3	-	-	
	S7	35'-36.5'		20	-	-	-	-	-	Sample Lost.
	S8	40'-41.5'	Clay, brown, stiff, flaky.	15	25.0	22.4	50.5	-	129.7	
	T9	45'-46.5'	Clay, gray-brown, med. stiff, flaky.	Levered	33.3	21.0	45.3	TR=900	119.3	
	VANE	48'	Clay, gray-brown, stiff, flaky.	-	-	-	-	V=906 1200	-	Sens: 3.8

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-100

W.P. 47-61

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
4	T10	55'-56.5'	Clay, gray-brown, stiff, with small silt pockets.	Levered	40.6 37.0	22.4	49.8	TR=1755 V=886	115.7	
	VANE	58'	As above.	-	-	-	-	1600	-	Sens: 3.8
	T11	65'-66.5'	Silty clay, gray-brown, stiff, with seams of clay silt.	Levered	30.7	20.6	26.2	-	120.0	
	VANE	68'	As above.	-	-	-	-	1520	-	Sens: 2.1
	T12	75'-76.5'	Silty clay, gray-brown, stiff, with some fine sand.	Levered	24.8	16.6	34.5	V=1020	124.0	
	VANE	78'	As above.	-	-	-	-	1840	-	Sens: 1.9
	S13	85'-86.5'	Sandy silt, red-brown, dense, with some clay and trace of fine gravel.	44	10.4	-	-	-	145.5	Glacial Till.
5	S1	5'-5.5'	Dense mixture of clay, ashes, fine to coarse sand and fine to coarse gravel.							
		5.5'-6.5'	Clay, brown, hard, flaky.	38	21.6	-	-	-	-	
	S2	10'-11.5'	Clay, brown, very stiff, flaky.	21	25.0	24.1	53.5	-	-	
	S3	15'-16.5'	Clay, brown, very stiff, flaky, with trace of fine sand.	28	23.3	-	-	U=2680	129.1	
	S4	20'-21.5'	Clay, brown, very stiff, flaky, with pocket of fine sand & trace of fine gravel.	24	26.0	22.4	48.1	-	-	

JOB 60- F-100

W.P. 47-61

HOLE NO.	SAMP NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS FT	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
5	S5	25'-26.5'	Clay, brown, stiff, flaky.	17	28.8	-	-	U=1755	123.1	
	S6	30'-31.5'	Clay, brown, stiff, flaky, with small pockets of red fine sand.	21	27.4	21.2	48.9	-	-	
	S7	35'-36.5'	Clay, brown, stiff, flaky, with trace of fine sand.	17	26.8	-	-	U=1760	124.1	
	S8	40'-41.5'	Clay, brown, stiff, flaky, with small pockets of fine gray sand.	13	38.4	25.0	63.0	-	-	
			S denotes split spoon sample							
			T " Shelby tube "							
			U " unconfirmed compression							
			V " lab vane							
			TR " triaxial compression							

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-100BW.P. 47-61

HOLE NO.	SAMP. NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
6	S1	10'-11.5'	Silty clay, v. stiff grey-brown.	24	25.2	23.4	49.3	-	-	
	T2	20'-21'	"	Levered	27.2	20.6	47.2	-	-	
	S3	25'-26.5'	"	20	27.1	-	-	-	-	
	T4	30'-31'	"	Levered	26.9	23.4	47.7	2665	125.4	
	T5	35'-36'	"	Levered	28.0	-	-	-	-	
	T6	40'-41.5'	Silty clay, stiff grey-brown.	Levered	29.0	20.9	42.7	1535	123.4	
	T7	45'-46.5'	"	Levered	35.9	-	-	-	-	
	T8	50'-51.5'	"	Levered	17.7	13.8	23.9	1930	135.3	
7	S1	5'-6.5'	Silty clay, hard brown occasional small silt pockets.	44	17.4	-	-	-	-	
	S2	10'-11.5'	"	39	22.3	21.2	49.4	-	127.6	
	S3	15'-16.5'	Silty clay with traces of fine gravel and occasional silt pockets - v. stiff-grey brown.	28	26.1	-	-	-	-	
	S4	20'-21.5'		27	24.8	23.4	51.1	-	123.1	
	S5	23'-24.5'	"	25	26.7	-	-	-	-	
	S6	26'-27.5'	"	24	26.8	23.0	50.1	-	-	
	S7	30'-31.5'	Silty clay, stiff, grey-brown.	15	26.5	-	-	-	-	

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-100BW.P. 47-61

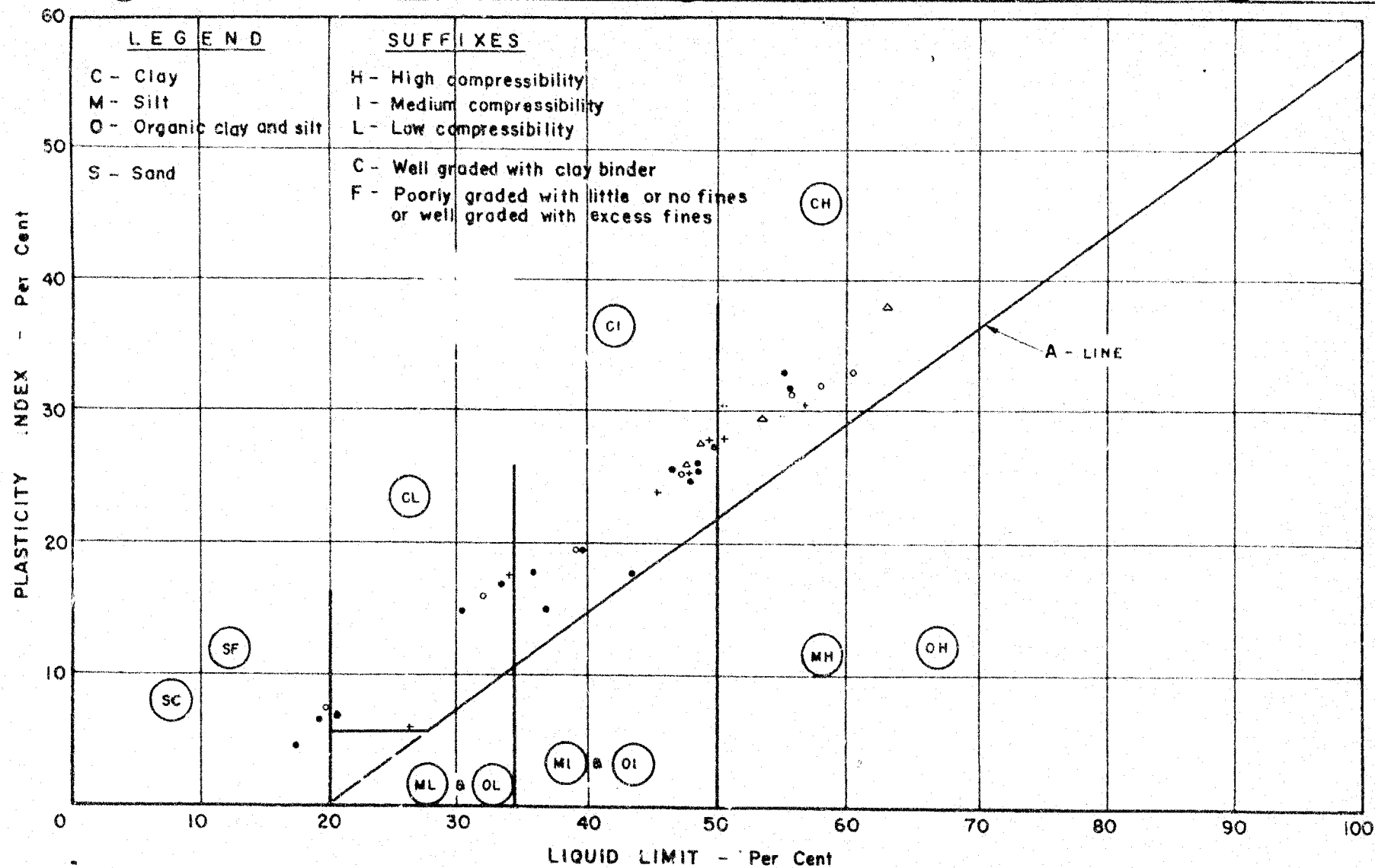
HOLE NO.	SAMP. NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
7	VANE	33'		-	-	-	-	>2000	-	Sens: >2.3
	T8	35'-36.5'	Silty clay, stiff, grey-brown.	Levered	24.7	20.7	43.6	1330	121.0	
	VANE	38'		-	-	-	-	1840	-	Sens: 2.9
	T9	40'-41.5'	"	Levered	27.4	21.5	38.7	1520	121.1	
	S10	45'-46.5'	"	11	29.6	-	-	-	-	
	T11	50'-51.5'	"	Levered	34.7	21.4	50.0	685	112.9	
8	S1	3'-4.5'	Silty clay, small pockets of silt, some root fibers, med. stiff to stiff. (fill)	18	19.2	-	-	-	-	
	S2	6'-7.5'	"	5	24.4	-	-	-	-	
	S3	9'-10.5'	"	10	27.0	22.0	49.5	-	-	
	S4	12'-13.5'	"	7	26.0	28.1	64.0	-	-	
	S5	15'-16.5'	Silty clay, med. stiff, grey-brown with some topsoil.	6	30.6	24.6	63.4	-	-	
	T6	18'-19.5'	Silty clay, med. stiff, grey-brown.	Levered	26.9	23.4	50.3	640	120.0	
	VANE	21'		-	-	-	-	>2000	-	Sens: >7.1
	T7	23'-24'	Silty clay, stiff, grey brown.	Levered	26.0	25.3	54.9	1540	122.0	

SUMMARY OF FIELD & LABORATORY TESTS

JOB 60-F-100B

W.P. 47-61

HOLE NO.	SAMP. NO.	SAMPLE DEPTH (FEET)	MATERIAL DESCRIPTION	PENET'N RESIST. BLOWS/FT.	MOIST. CONT. %	PLASTIC LIMIT %	LIQUID LIMIT %	SHEAR STRENGTH p.s.f.	UNIT WEIGHT p.c.f.	REMARKS
8	S8	28'-29.5'	Silty clay, v. stiff.	21	28.6	24.2	49.6	-	-	
			S denotes split spoon sample. T " shelby tube "							



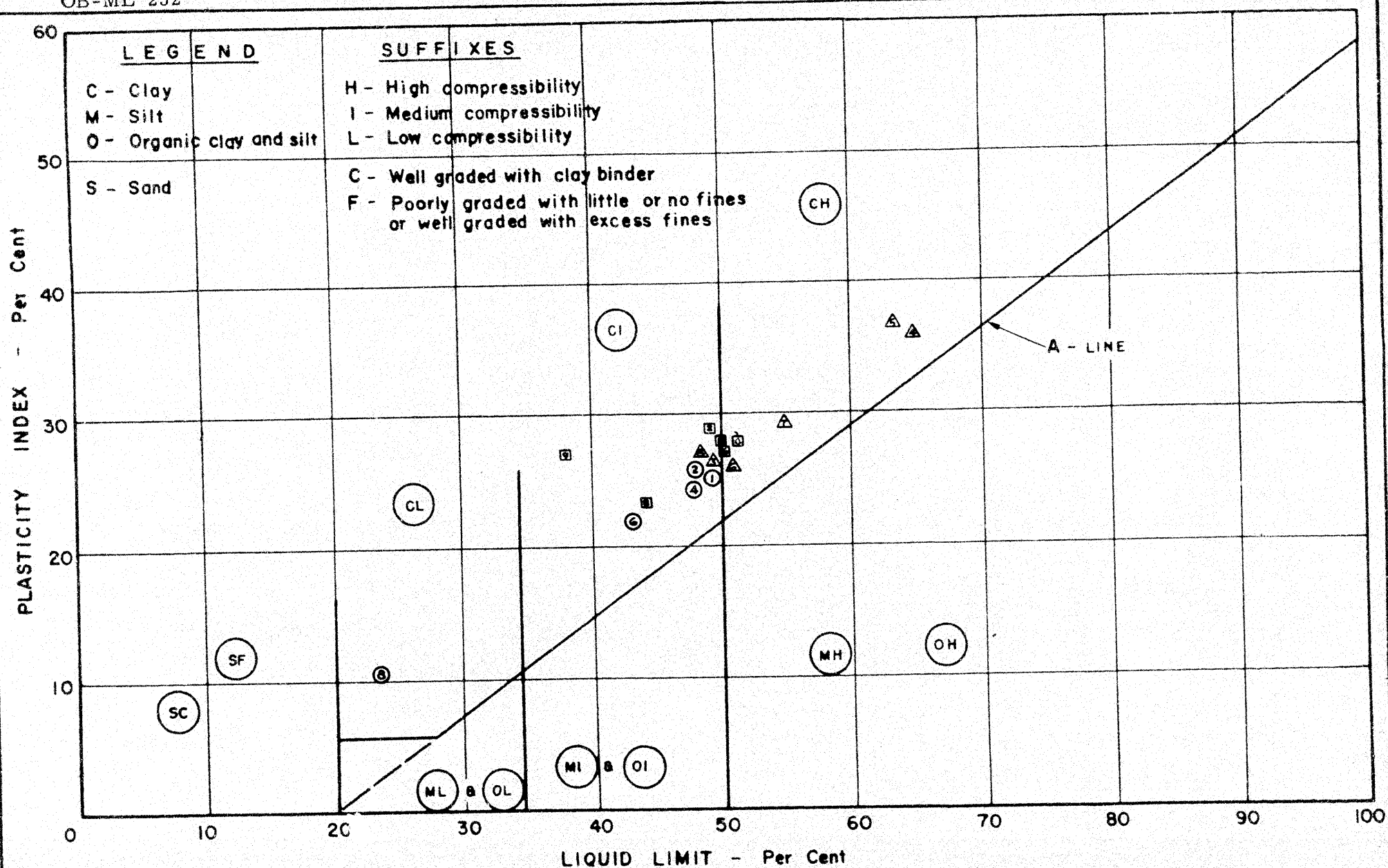
NOTES

● - BH#2
○ - BH#3
+ - BH#4
△ - BH#5

Figure A1

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION
PLASTICITY CHART

Job No. 60-F-100 W.P. No. 47-61
Location Hwy. 56 at Westchester Ave.



NOTES

BORE HOLE NO 6 - SAMPLES - ① ② ④ ⑥ ⑧

" " " 7 - " - ② ④ ⑥ ⑧ ⑩ ⑪

" " " 8 - " - ▲ ▲ ▲ ▲ ▲ ▲

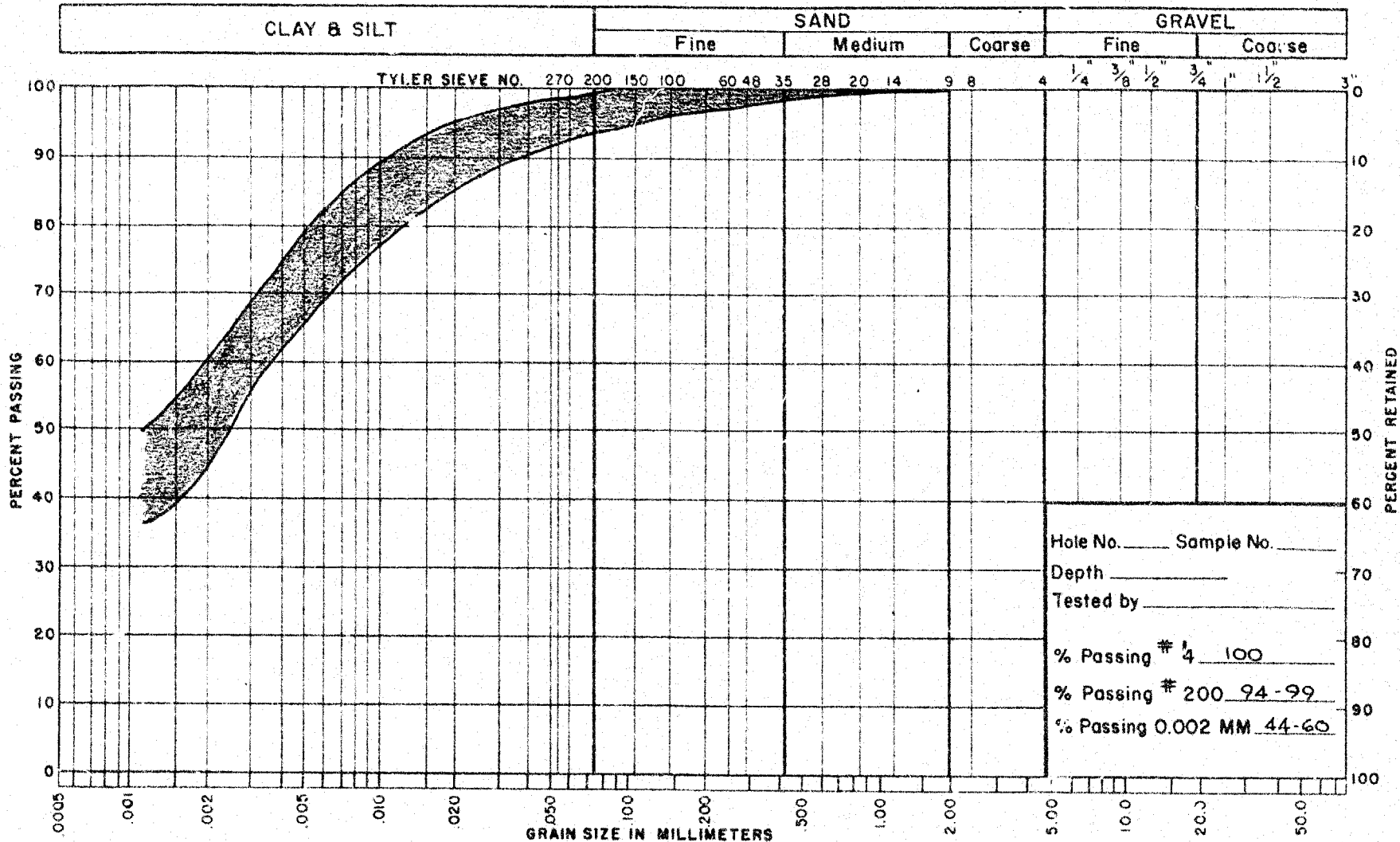
Figure A2

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION
PLASTICITY CHART

Job No. 60-F-100 B W.P. No. 47-61

Location WESTCHESTER CRES.

UNIFIED SOIL CLASSIFICATION SYSTEM



NOTES Composite Grading Chart

Brown to Grey Brown Clay to Silty Clay
(C.L. - C.I. - C.H. Material)

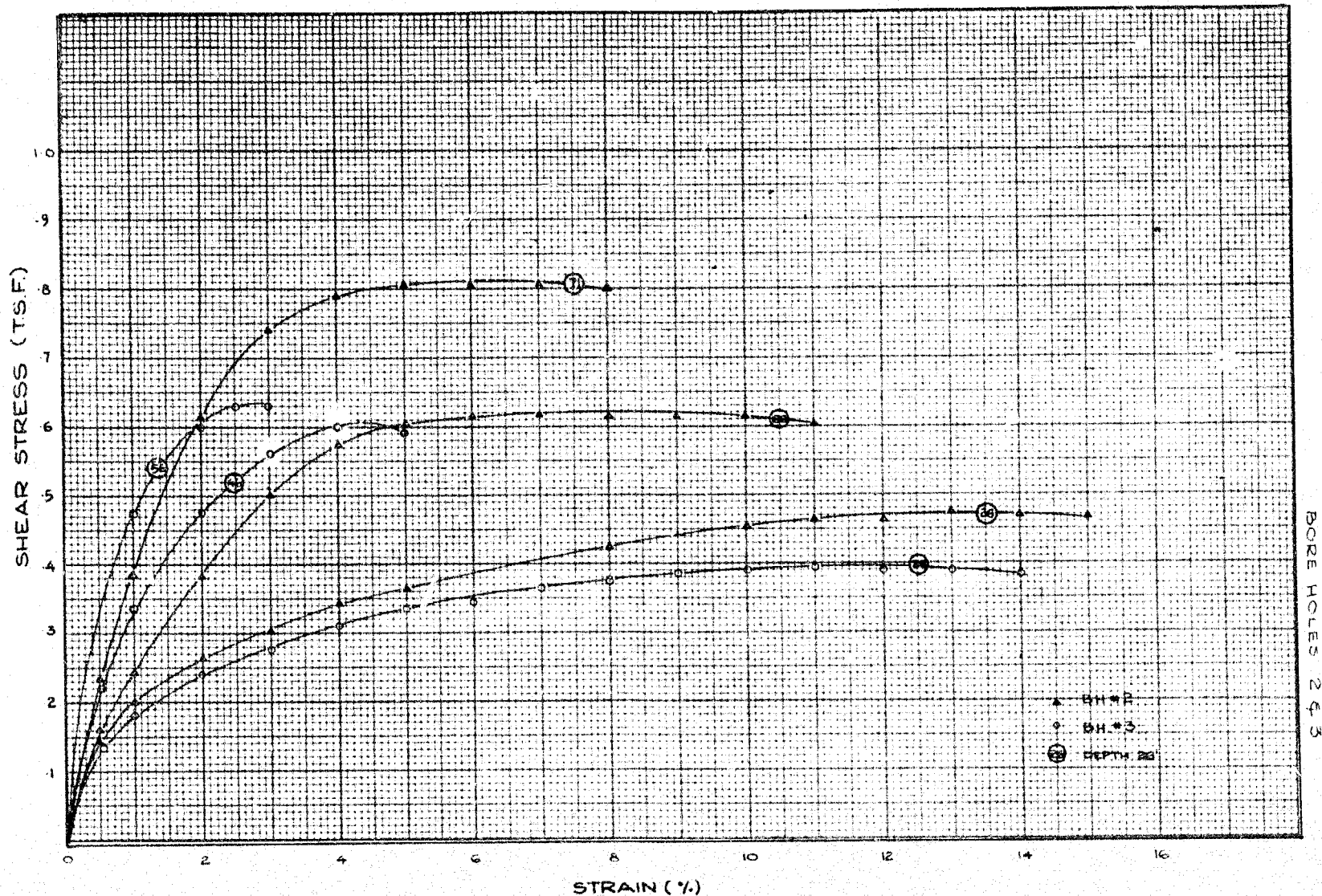
Figure A3

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION
GRAIN SIZE DISTRIBUTION

Joh No. 60-F-100 W.P. No. 47-61
Location Hwy. 58 at Westchester Ave.

STRESS - STRAIN CURVES

Figure A4



W.P. 47-61 JOB 60-F-100
Hwy 58 at Westchester Ave.
BORE HOLES 2 & 3

TYPICAL $e - \log p$ CURVES

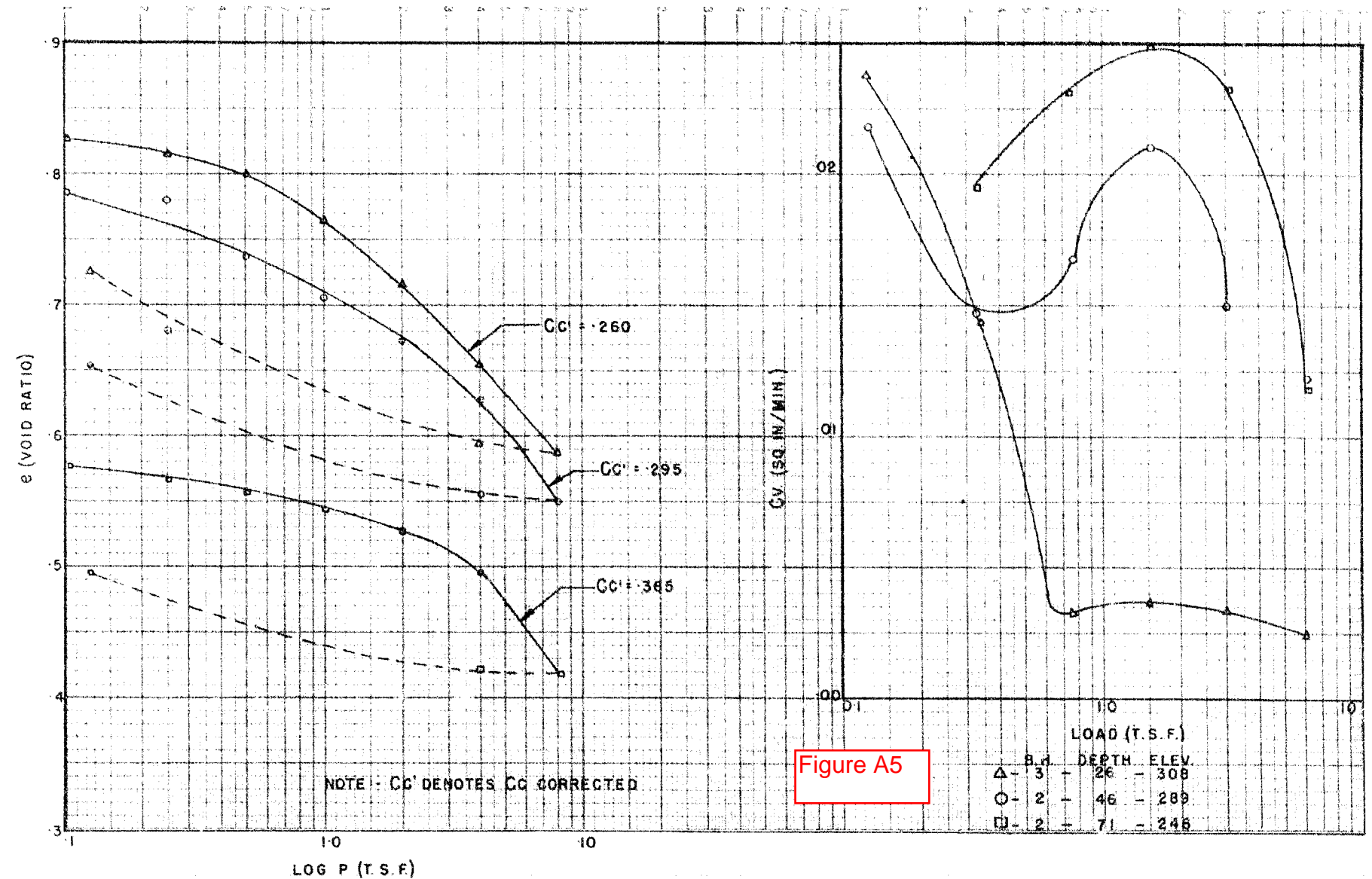
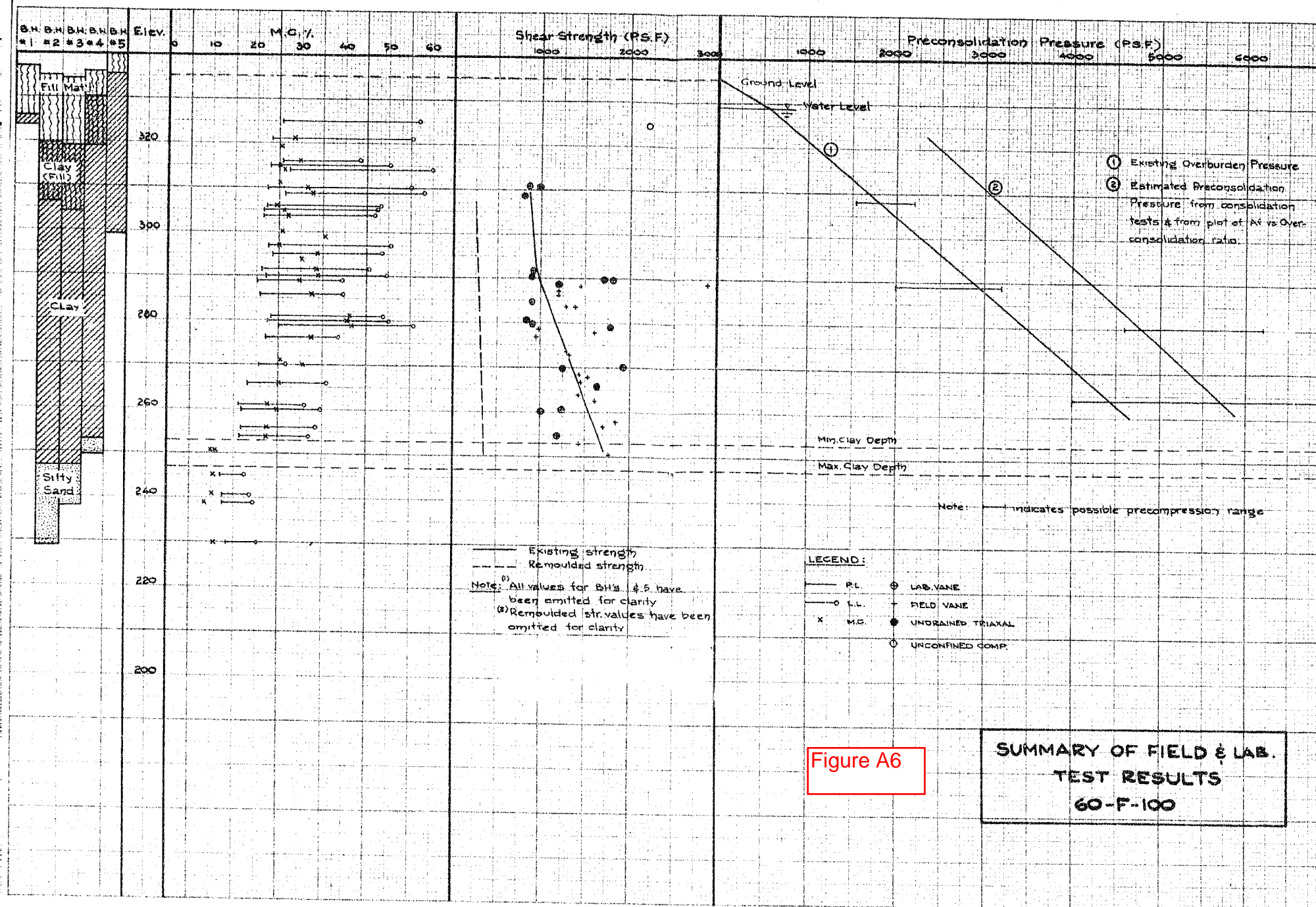
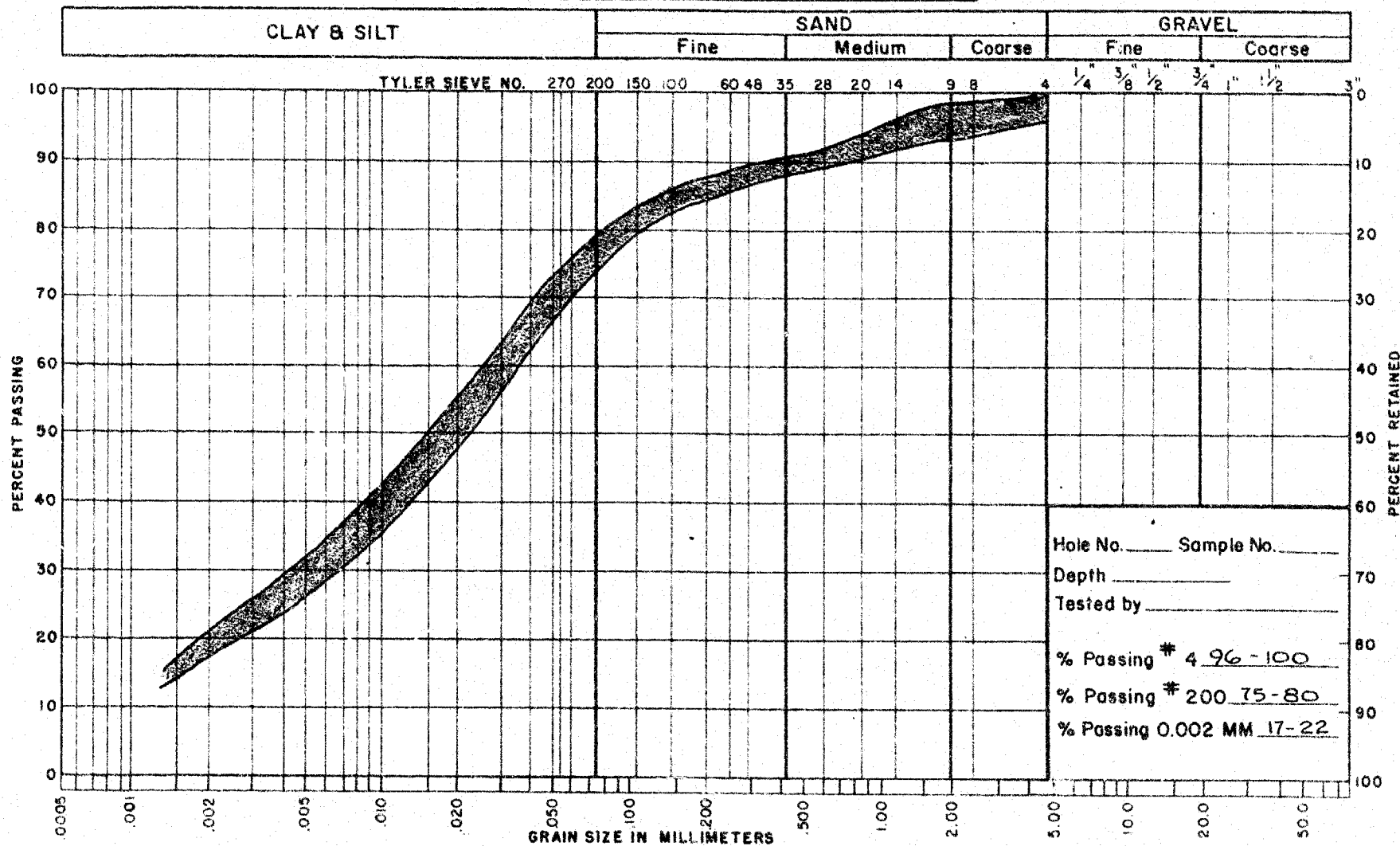


Figure A5



UNIFIED SOIL CLASSIFICATION SYSTEM

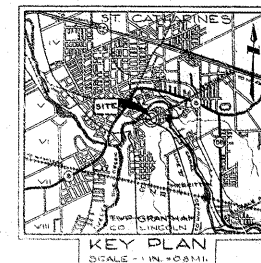
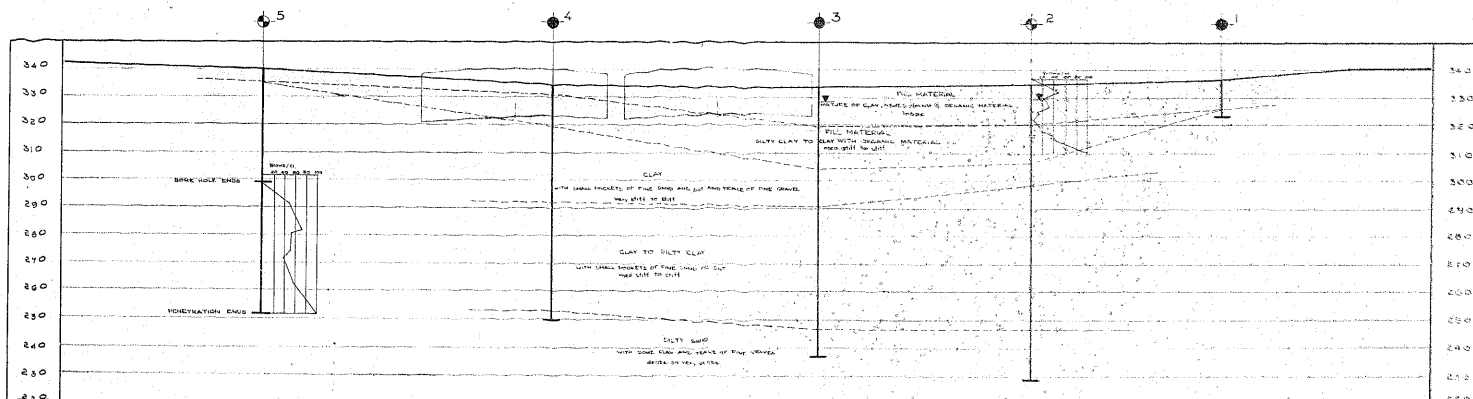
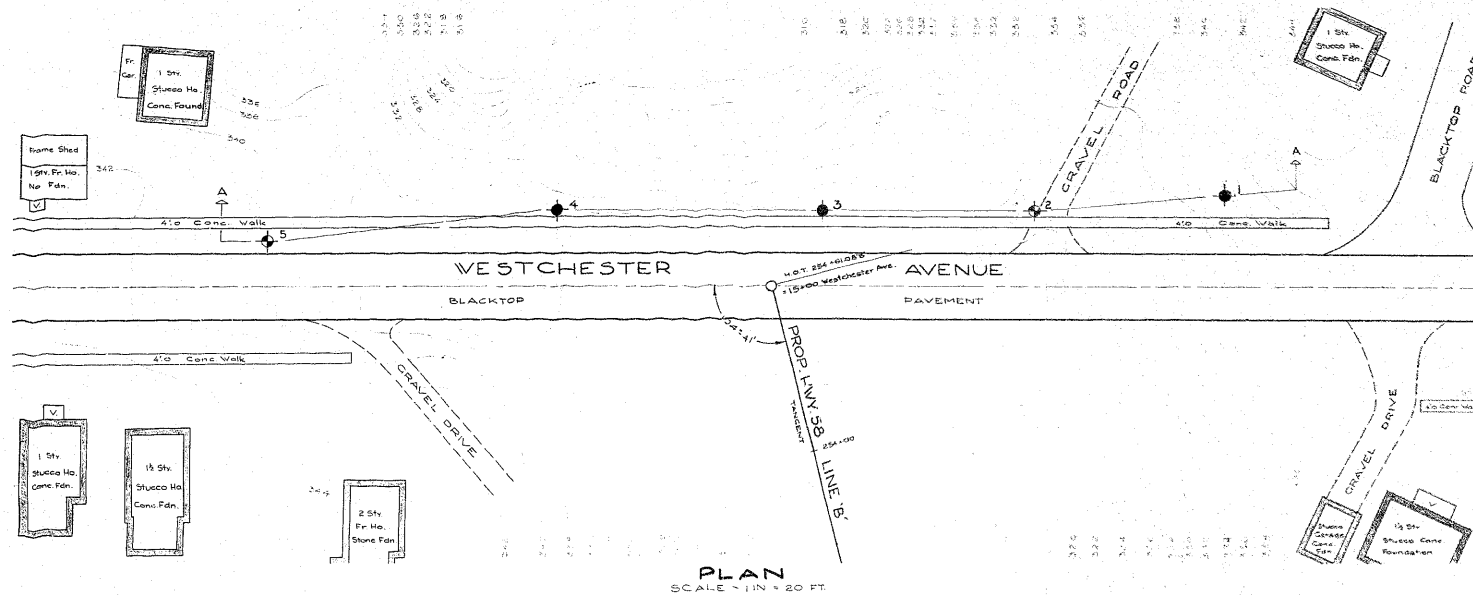


NOTES Composite Grading Chart
Red Silty Sand (S.F.-CL Material)

Figure A7

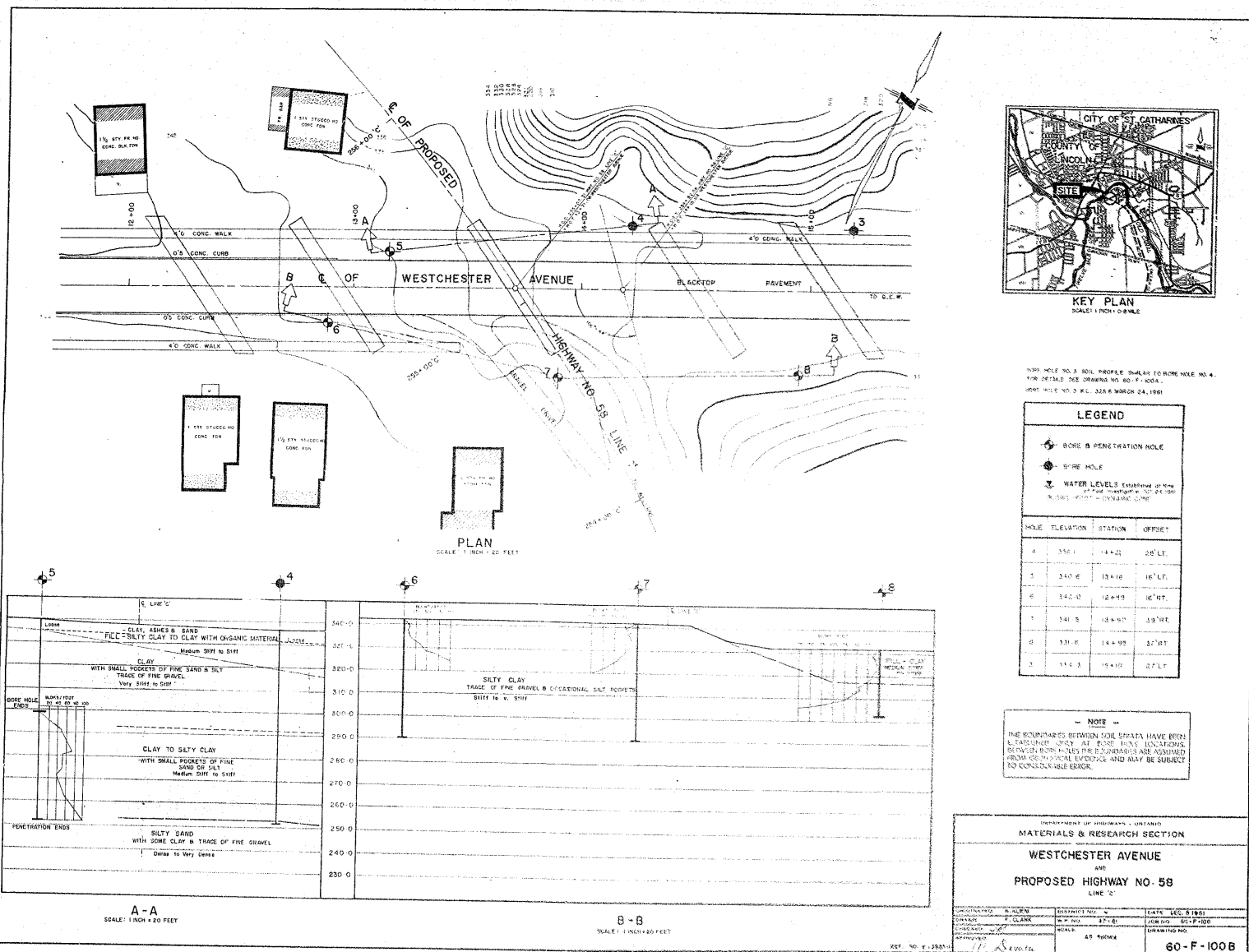
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION
GRAIN SIZE DISTRIBUTION

Job No. 60-F-100 W.P. No. 47-61
Location Hwy. 58 at Westchester Ave.



LEGEND				
BORE HOLE				
BORE AND PENETRATION HOLE				
HOLE	ELEVATION	STATION	DISTANCE FROM TANGENT	
1	326.6'	254+05	165' RT	
2	324.9'	254+64	95' RT	
3	324.5'	254+85	24' RT	
4	324.1'	255+07	72' LT	
5	320.6'	255+24	175' LT	

DEPARTMENT OF HIGHWAYS - ONTARIO			
MATERIALS & RESEARCH SECTION			
WESTCHESTER AVE			
AND PROPOSED HIGHWAY NO. 58 LINE 'B'			
ORIGINATED BY: CHERRINGTON	DESIGNED BY: A	DATE: 24 MARCH 1951	
DRAWN BY: M. S. LEVINSKY	W. P. NO.: 47-51	JOB NO.: 50-F-100	
CHECKED BY: J. J. GORDON	SCALE: 1 IN. = 20 FEET	DRAWING NO.: 60-F-100A	
APPROVED BY: J. J. GORDON			





APPENDIX B

**GEOCRE 30M3-42 - Letter Dated November 29, 1965 and
Drawing D-4965-1 General Plan, dated December 1961 and
Drawing D-4965-2 Details of Abutment and Pier Footings**

Department of Highways Ontario

Copy for the information of

Mr. A. Stermac, Principal Foundation Engineer,
Room 107, Lab. Building.

Mr. H. Greenland,
District Engineer,
Hamilton District.

Bridge Division,
Downsview, Ontario.

November 29, 1965.

Westchester St. Bridge
W.P. 47-61, Contract No. 62-316
Hwy. #406, District No. 4

During a recent tour of inspection of the bridges in the St. Catharines area we observed that the east abutment Andre bearings of the above structure were distorted and the anchor bolts bent. As the distortion was by no means uniform, we came to the conclusion that this could only come about by a combination of differential settlement and rotation of the footing, although the latter is sitting on piles.

We suggest that this abutment be kept under observation and measurements taken periodically to determine the actual movements.

The west abutment bearings do not appear to be affected.

WMcF:rd

W. McFarlane,
Senior Bridge Design Services Engineer.

c.c. A. Stermac
W. Birch

For the above mentioned project the District people ^{to}~~should~~ be contacted are as follows:-

- 1) Joe Castellona MU4-0252 (St. Catharines) - Project Supervisor during construction
- 2) Roger Verschure 684-5245 (")
- 3) Lynn Fisher Dist Office Hamilton

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

For more information, visit golder.com

Africa	+ 27 11 254 4800
Asia	+ 86 21 6258 5522
Australasia	+ 61 3 8862 3500
Europe	+ 44 1628 851851
North America	+ 1 800 275 3281
South America	+ 56 2 2616 2000

solutions@golder.com
www.golder.com

Golder Associates Ltd.
110 Hannover Drive, Building A, Suite 203
St. Catharines, Ontario, L2W 1A4
Canada
T: +1 (905) 688 8217

