

June 28, 2011

PML Ref.: 10KF029

Mr. Chris Spere CET
MTE Consultants Inc.
520 Bingemans Centre Drive
Kitchener, Ontario
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Dear Mr. Spere

**Response to MTO Comments on
Foundation Investigation and Design Report
Blair Business Park Sanitary Sewer – Highway 401 Crossing
Cities of Cambridge and Kitchener, Ontario**

We received a copy of the memorandum dated October 15, 2010 from MTO Pavements and Foundations Section with the comments on the Foundation Investigation and Design Report that Peto MacCallum Ltd. (PML) prepared for the above referenced project. PML reviewed the MTO comments and are pleased to submit the following answers. At the same time, we are pleased to submit to you our Revised Final Foundation Investigation and Design Report for the project.

1. The MTO GEOCREC number 40P8-190 is shown on the title pages of the Final Report and on the Foundation Drawing.
2. The chainage along the sewer forcemain alignment on Drawing 1 has been changed to match the chainage on Drawing PP4.3.
3. (a) No comment; (b) The Final Report addresses the embankment stability shown on Drawing PP4.3 between station 8+320 and 8+340.
4. (a) The Final Report addresses soil removal options within the pipe; and, (b) the potential for liquefaction of wet silty soils encountered in Boreholes 21 and 22.
5. (a) The Final Report comments on the monitoring program; (b) estimated settlements/heave as per MTO Guidelines in Appendix B; and (c) the potential impacts of the trenchless crossing to the concrete storm sewer located within the Hwy 401 median.
6. The Final Report addresses the potential effects of aggressive dewatering.
7. The Final Report presents the applicable soil parameters for each different soil type required for the steel sheet piling shoring design.
8. The Final Foundation Investigation Report and Foundation Drawings have been signed and stamped by at least two Professional Engineers licensed by PEO, one of which is PML's Designated Principal Contact for Foundation Engineering projects.



We thank Mr. David Staseff, P.Eng. and Mr. T.C. Kim, P.Eng., of the MTO Pavements and Foundations Section for their review notes. We believe that the Final Report has been completed within our terms of reference. Please contact our office should you have any questions.

Sincerely

Peto MacCallum Ltd.



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FOUNDATION INVESTIGATION AND DESIGN REPORT

for

**BLAIR BUSINESS PARK SANITARY SEWER — HIGHWAY 401
CROSSING
CITIES OF CAMBRIDGE AND KITCHENER, ONTARIO**

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Mr. Chris Spere CET
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520 Bingemans Centre Drive
Kitchener, Ontario
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Dear Mr. Spere

**Foundation Investigation and Design Report
Blair Business Park Sanitary Sewer – Highway 401 Crossing
Cities of Cambridge and Kitchener, Ontario**

Peto MacCallum Ltd. (PML) is pleased to present our revised foundation investigation and design report for the proposed twin sanitary forcemain crossing of Highway 401 by trenchless methods.

The crossing is part of the proposed sanitary sewage system which is to connect servicing of the future Blair Business Park and Conestoga College South Campus in the City of Cambridge to the Kitchener Wastewater Treatment Plant.

As the project involves the crossing of Highway 401, the crossing must comply with the Ministry of Transportation (MTO) "Guidelines for Foundation Engineering – Tunnelling Specialty for Corridor Encroachment Permit Application" dated April, 2008.

The attached foundation investigation and design report has been prepared as per the project requirements and the above noted MTO guidelines.

This report supplements our geotechnical investigation report for the entire sewage system, issued as Report 1, PML Ref. 10KF029.

We trust this report has been completed within our terms of reference and is sufficient for your current needs. Should you have further questions, please do not hesitate to contact our office.

Sincerely

Peto MacCallum Ltd.



Romin Agahzadeh P.Eng.
Manager, Geotechnical Services



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Explanation of Terms Used in Report

Record of Borehole Sheets

Drawing 1 - Borehole Locations and Soil Strata

Appendix A – Copy of drawing provided by MTE outlining the proposed crossing
 (Drawing No. PP4.3)

Appendix B – Copy of Ministry of Transportation's "Guidelines for Foundation Engineering –
 Tunnelling Specialty for Corridor Encroachment Permit Application"

FOUNDATION INVESTIGATION AND DESIGN REPORT

For

Blair Business Park Sanitary Sewer – Highway 401 Crossing
Cities of Cambridge and Kitchener, Ontario

1. INTRODUCTION

This report pertains to the foundation investigation and design for the installation by trenchless methods of a proposed twin sanitary forcemain crossing of the Highway 401, between the Cities of Cambridge and Kitchener. The crossing is part of the proposed sanitary sewage system which is to connect servicing of the future Blair Business Park and Conestoga College South Campus in the City of Cambridge to the Kitchener Wastewater Treatment Plant (KWTP).

Detailed design for the proposed crossing was not available at the time of preparation of this report; however, based on preliminary information provided by MTE Consultants Inc. (MTE), the crossing configuration proposed could include one casing pipe housing both forcemains or two casing pipes each housing one forcemain. The proposed forcemains will each be 300 mm in diameter. At present, consideration is being given to housing the two forcemains in a minimum 900 mm diameter casing pipe, which is to be installed by trenchless methods with at least 4.0 m of cover and inverts of not greater than 6.5 m below the travelled lanes of Highway 401 (elevation 285.2 to 284.0). The proposed casing(s) will extend the entire width of the 90 m highway right of way (from the south to the north property lines). A drawing (Drawing No: PP4.3 provided by MTE) outlining the proposed crossing has been included in Appendix A.

As the project involves the crossing of Highway 401, the crossing must comply with the Ministry of Transportation (MTO) "Guidelines for Foundation Engineering – Tunnelling Specialty for Corridor Encroachment Permit Application" dated April, 2008, a copy of which has been provided in Appendix B. This foundation investigation and design report has been prepared as per the project requirements and the above noted MTO guidelines.



2. SITE DESCRIPTION AND GEOLOGY

The Highway 401 crossing site is located approximately 1km east of the Homer Watson Boulevard overpass structure, bordering the Cities of Cambridge and Kitchener in the Regional Municipality of Waterloo, as shown on the appended Drawing 1. The proposed crossing will extend from the south side of Highway 401 (Morningside Drive and Conestoga College South Campus, City of Cambridge) to the north side of Highway 401 (Conestoga College Parking Lot, City of Kitchener). The land on the Cambridge side (south side of the highway) is currently being used for agricultural purposes while the land on the Kitchener side (north side of the highway) is being used as a parking lot for the existing Conestoga College Doon Campus. An existing storm water management pond (SWMP) is also located adjacent to the parking lot on the north side of the highway.

Grades along the highway corridor (adjacent to the highway ditch lines) have been lowered such that the travelled lanes of the highway are about 1.5 to 2.0 m below the level of the natural topography of the lands located to the north and south of the highway.

The Highway 401 right of way (property limits) at the crossing site is approximately 90 m in width. The highway is a divided paved freeway, with a tall wall median barrier, with four west bound and four east bound lanes (including one speed change lane in each direction). A storm sewer is located beneath the median barrier nearest to the west bound lanes. It is understood that the storm sewer is a 450 mm diameter concrete pipe. The actual invert of the sewer is not known, however, it is understood that its invert is approximated to be at 2.6 m below existing grade (approximate elevation of 287.7). It is further understood that a minimum of 2 m of separation will be maintained at all times between the base of the existing sewer and the top of the casings.

The site is located in an outwash plain bordered by the Grand River to the north, south and east, with kame moraine topography of the Waterloo Sand Hills to the west. Generally the subsurface stratigraphy in this area is highly variable. Coarser outwash sand and gravel deposits are common in proximity to the Grand River. Fine sand and silt deposits are commonly found in the Waterloo Sand Hills. Tills from the Port Stanley, Mary Hill, and Catfish Creek formations are also



expected in this area. Bedrock is known to be dolostone of the Salina Formation, and is expected at about 30 m depth.

3. INVESTIGATION METHODOLOGY

The field work for the proposed crossing was carried out on July 15 and 16, 2010 and comprised the drilling of three boreholes advanced to depths between 9.5 and 11.1 m at the locations shown on the appended Drawing 1, Borehole Locations and Soil Strata. The boreholes for the tunnel were drilled concurrently with the boreholes drilled as part of the geotechnical investigation for the overall project and were identified as Boreholes 20, 21 and 22.

The proposed forcemain alignment was marked in the field by MTE. The borehole locations were established in field by PML in advance of drilling. The survey and tie in of the boreholes was completed by MTE.

The fieldwork was supervised throughout by a member of our engineering staff who directed the drilling and sampling operations, prepared the stratigraphic logs, monitored ground water conditions and processed the recovered samples.

The boreholes were advanced using a CME 75 track mounted drill rig equipped with continuous flight hollow stem augers. The drilling equipment was supplied and operated by a specialist drilling contractor working under subcontract to PML. Traffic protection in accordance with the MTO road occupancy permit and Ontario Traffic Manual - Temporary Conditions Manual, Book 7, was provided by Steed and Evans for Borehole 21, which was located on the highway.

Representative samples of the overburden were recovered at frequent intervals throughout the depths explored. Standard penetration testing was conducted simultaneously with split spoon sampling operations to assess the strength of the subsurface strata.



The ground water conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and drill rods as samples were retrieved and when appropriate, by measurement of the water level in the borehole. Monitoring wells were installed in Boreholes 20 and 22 to more accurately measure ground water levels. Reference is given to the appended Record of Borehole sheets for details of the well configurations. The drilling and decommissioning of the boreholes and the installation of the monitoring wells were carried out in accordance with O. Reg. 903 and as amended. It should be noted that the installed monitoring wells will require decommissioning in accordance with the above noted regulation prior to construction.

Soils were identified in the field according to the Unified Soil Classification System and adjusted to the MTO Soil Classification system, after detailed examination of recovered samples and laboratory testing.

The laboratory testing program comprised visual examination and moisture content determination on all recovered samples. One Atterberg plasticity limits test and nine particle size distribution analyses were conducted on selected soil samples to determine specific properties of the main soil types encountered. Atterberg plasticity limits were not attempted on samples deemed to be non-plastic by visual and tactile examination. Results of the particle size distribution analyses and Atterberg plasticity limits test are presented on the appended Figures GS-1 to GS-5 and PC-1, respectively.

4. SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the field work including soil descriptions, inferred stratigraphy, standard penetration test (SPT) N values, ground water observations, well configurations and the laboratory test results.

In general, the subsurface soil and ground water conditions encountered along the proposed crossing alignment at the borehole locations consisted of surficial pavement structure and topsoil overlying native deposits of sandy gravel, sand and silt deposits which in turn are underlain by a clayey silt till deposit that extended to the termination depths of all the boreholes.



4.1 Pavement Structure

The surficial pavement structure was contacted in Borehole 21 and extended to 2.0 m depth (elevation 288.1). The pavement structure consisted of a 140 mm thick layer of asphaltic concrete overlying 1860 mm of gravelly sand pavement fill. SPT N values of 45 and 68 blows per 0.3 m penetration of the split spoon sampler were measured within the gravelly sand pavement fill.

The moisture content of the pavement fill was about 5%.

4.2 Topsoil

Surficial sandy silt and silt topsoil was encountered in Boreholes 20 and 22. The topsoil layers were measured to be 400 and 600 mm thick and terminated at elevation 291.7 and 291.0 (Boreholes 20 and 22, respectively).

4.3 Sandy Gravel

Sandy gravel deposits were encountered at depths of 0.4 and 1.2 m (elevation 291.7 and 290.4) underlying the topsoil and silt layers in Boreholes 20 and 22, respectively. The deposit generally comprised trace to some silt, trace clay and cobbles and extended to depths of 2.9 and 3.7 m (elevation 289.2 and 287.9) in Boreholes 20 and 22, respectively. SPT N values measured in the sandy gravel were in excess of 50 blows per 0.3 m penetration, indicating very dense conditions.

The moisture content of the sandy gravel deposit ranged from about 2 to 4%. The results of grain size distribution analyses conducted on samples of the deposit are presented on Figure GS-1, attached.



4.4 Sand

A sand deposit was encountered at a depth of 3.7 m (elevation 287.9) below the sandy gravel deposit in Borehole 22. The deposit comprised trace gravel, trace silt, trace clay and cobbles and extended to 8.4 m depth (elevation 283.2). SPT N values measured in the sand deposit ranged from 22 to 64 blows per 0.3 m penetration, indicating compact to very dense conditions.

The moisture content of the sand deposit ranged from about 2 to 18%. The results of grain size distribution analyses conducted on samples of the deposit are presented on Figure GS-2, attached.

4.5 Silt

Silt was encountered at depths of 2.9, 2.2 and 0.6 m (elevation 289.2, 287.9 and 291.0) below the sandy gravel, pavement fill and topsoil layers in Boreholes 20, 21 and 22, respectively. The deposit generally comprised trace to some sand, trace gravel and trace clay and extended to depths of 4.8, 4.5 and 1.2 m (elevation 287.3, 285.6 and 290.4) in Boreholes 20, 21 and 22, respectively. SPT N values measured in the silt deposit ranged from 19 to 34 blows per 0.3 m penetration, indicating compact to dense conditions.

The moisture content of the silt deposit ranged from about 13 to 22%. The results of grain size distribution analyses conducted on samples of the deposit are presented on Figure GS-3, attached.

4.6 Silt/Clayey Silt Till

Silt / clayey silt till deposits were encountered underlying the above noted deposits at depths of 4.8, 4.5 and 8.4 m (elevation 287.3, 285.6 and 283.2) in Boreholes 20, 21 and 22, respectively. The deposits generally comprised trace to some sand, trace to some gravel and trace clay and extended to the borehole termination depths. SPT N values within the tills were typically in excess of 40 blows per 0.3 m penetration, indicating dense/hard conditions.



The moisture content of the deposit ranged from about 8 to 11%. Layers of wet silty sand were also encountered within the till soils. The results of grain size distribution analyses conducted on samples of the deposit are presented on Figure GS-4 and GS-5, attached. The results of an Atterberg plasticity limits test conducted on a sample of the clayey silt till deposit is presented on Figure PC-1, attached. The liquid limit and plastic limit of the clayey silt till was 18 and 13, respectively, with a plasticity index value of 6.

4.7 Ground Water

Wet conditions were generally observed during the drilling operations below depths of 2.2 to 4.3 m (below approximate elevation 288). Furthermore, layers of saturated non-cohesive soil were encountered in the clayey silt till soils which extended to the termination depths of all three boreholes. On August 23, 2010, ground water levels in the monitoring wells installed in Boreholes 20 and 22 were measured at depths of about 4.1 and 4.2 m (elevation 288.0 and 287.4), respectively.

The ground water levels at the site are subject to seasonal fluctuations and precipitation patterns. It should be noted that the SWMP located north of the highway in close proximity to Borehole 22 is most likely founded on silt and sand deposits, and the rapid development of perched conditions to the level of the water in the stormwater pond, should be anticipated in the onsite soils.

5. DISCUSSION AND RECOMMENDATIONS

The Highway 401 crossing site is located approximately 1km east of the Homer Watson Boulevard overpass structure, bordering the Cities of Cambridge and Kitchener in the Regional Municipality of Waterloo.



As noted previously, detailed design for the proposed crossing was not available at the time of preparation of this report; however, based on preliminary information provided by MTE, the crossing configuration proposed could include one casing pipe housing both forcemains or two casing pipes each housing one forcemain. The proposed forcemains will each be 300 mm in diameter. At present, consideration is being given to housing the two forcemains in a minimum 900 mm diameter casing pipe, which is to be installed by trenchless methods with at least 4.0 m of cover and inverts of not greater than 6.5 m below the travelled lanes of Highway 401 (elevation 285.2 to 284.0). The proposed casing(s) will extend the entire width of the 90 m highway right of way (from the south to the north property lines). A drawing (Drawing No: PP4.3 provided by MTE) outlining the proposed crossing has been included in Appendix A.

Based on the preliminary details described above, the anticipated tunnelling area will be located at depths of between 4.0 and 6.5 m (approximate elevation 285.2 and 284.0) below the travelled lanes of the highway. The final design invert elevation of the crossings will ensure that a minimum cover of 2 m is provided between the base of the existing 450 mm diameter storm sewer and the top of the casings. The subsurface soils encountered in the boreholes within the anticipated tunnelling depths comprised native compact to very dense or hard native sand, silt and or clayey silt till deposits. The sand and silt deposits were found to be wet. In addition, wet silty sand layers were noted within the clayey silt till deposit. Ground water observations carried out during the drilling operations as well as in the installed monitoring wells indicate that the tunnel will generally be located below the ground water table at the crossing site (approximate elevation 288). The ground water level will vary seasonally as well as be affected locally by the presence of the existing SWMP located adjacent to the north side of the highway crossing.

Based on the available project information and the subsurface soil and ground water conditions encountered at the crossing site, tunnelling methods which could be used on this project include jack and bore, pipe ramming, and horizontal directional drilling (HDD).

It should be noted that the following discussion and recommendations are presented respective of the preliminary crossing details provided by MTE. Alternative configurations and installation methods may also be considered, provided they meet the needs of the project proponent and the MTO.



Further, the recommendations presented are based on the boreholes drilled during the subsurface investigation carried out along the currently proposed alignment. Additional subsurface investigation will be required if the crossing alignment is altered/shifted.

5.1 Tunnelling Methods

Regardless of the method used, it is recommended that the contractor prepare a plan in advance of construction outlining the details of the installation to provide instructions for the construction crews, and provide a possible contingency action plan should difficulties occur during the tunnelling operations. The plan should also be reviewed by the project proponent and the MTO prior to construction. Upon request, PML can assist in reviewing the plan to check that assumptions regarding soil and ground water conditions are appropriate.

It should be noted that the stratigraphy between boreholes may vary and areas of weaker or denser soil may be present along the planned route.

It should further be noted that the tunnelling operations should take into account the presence of existing infrastructure at the site. In particular, the existing 450 mm diameter concrete storm sewer which is located within the Highway 401 median area with an invert level that is understood to be at approximate elevation 287.7. The actual invert elevation of the storm sewer must be verified prior to construction. It is understood that a minimum 2 m of separation between the sewer invert and the top of casings will be provided once the final invert level of the existing sewer has been determined.

In addition, care should be taken to ensure that the existing Highway 401 south ditch (between Station 8+325 and 8+345) is not undermined by the tunnelling activities. Based on the current proposed tunnelling depths (as shown on Drawing PP4.3) it is understood that the south ditch invert will be approximately 2 to 2.5 m above the casings. Based on the above, we do not anticipate any embankment stability issues for the ditch, however, we recommend that the ditch embankment be monitored during the tunnelling operations.



Reference is also given to Ontario Provincial Standard Specifications (OPSS) 415, Construction Specifications for Pipeline and Utility Installation by Tunnelling.

A general description of the tunnelling methods which could be used on this project are presented below.

5.1.1 Jack and Bore

Jack and bore typically involves the simultaneous advancement of a continuous flight auger and conduit pipe. The auger is used to excavate soil in advance of the casing and transport cuttings back to the receiving pit where they are removed. Rotary power to auger and pushing force is provided by a drill rig located within a jacking pit. Jack and bore is a common method of trenchless installation and in appropriate site and soil conditions may be preferable from a cost perspective.

Jack and bore installation(s) should be conducted in accordance with OPSS 416, Construction Specifications for Pipeline and Utility Installation by Jacking and Boring.

For this site the relatively shallow ground water level could hinder or prevent a jack and bore installation. In wet soils there is potential for ground surface subsidence due to running of wet soil into the bore, which could result in voids. To eliminate this potential, significant ground water control measures would be required at the jacking and receiving pits as well as along the length of the bore. Ground water control along the boring path may require the installation of well points, collection pipes and pumping systems located within the Highway 401 right of way.

In addition, the presence of cobbles and boulders in the site soils will increase the risk for alignment deviations to occur and a significant disadvantage of this method is that the alignment cannot be corrected during pipe advancing. Ideally, a single 1 m bore could be used to allow removal of cobbles and or boulders.

Cognizant of the significant dewatering requirements and associated impact on Highway 401 traffic, a jack and bore method is not recommended for the proposed crossing.



5.1.2 Pipe Ramming

Pipe ramming installation is analogous to driving an open ended tube pile horizontally. Impact forces from a percussive hammer are used to advance a conduit pipe from an entry pit to a receiving pit. During the advance, most of the soil being penetrated fills the conduit rather than being excavated. The rammed conduit is terminated in a receiving pit at which point the soil contained in the pipe is removed.

When the driving has been completed, soil within the pipe can be removed via augering or a pipe shovel. Augering is expected to be the preferred method provided cobbles and boulders can be loosened and cleared between the flighting.

If soil within the pipe cannot be augered, use of a pipe shovel will be necessary. A pipe shovel is essentially a special scoop made from a pipe which fits inside the liner. Excavation via pipe shovel involves advancing the shovel into the soil plug using impact hammer (mole), then pulling the shovel and its contents out with a chain or cable, the process is repeated as required.

Minimal ground water control should be needed along the installation path because the soil within the pipe is not removed until after the crossing has been completed. The retained soil will tend to act as a plug, reducing the potential for ground water seepage and running of soil into the pipe.

Reference is given to the Staging Excavations section for recommendations pertaining to of the construction of entry and receiving pits. Significant dewatering measures will be required at the entry and receiving pits.

Pipe ramming can be conducted though soils with cobbles and boulders, however, difficult driving can be expected.

Given the depth of the pipe, the compact to dense and hard conditions of the native soils, liquefaction of the soil from pipe ramming is not expected to be a concern.



Given the compact to very dense nature of the site, sand and sandy gravel soil, stability of the cut back slopes and ditch areas is not a concern for a pipe ramming installation.

5.1.3 Horizontal Directional Drilling

HDD involves the boring and enlargement of an uncased near horizontal tunnel which is kept open through use of drilling fluids. Upon completion of boring a conduit pipe is pulled through the bore. The process starts by advancing a relatively small diameter hole along the proposed path. During the pilot bore the cutter head at the lead of the drill string is steered by the drillrig, forming a curved boring path. After the pilot hole has been completed the borehole is enlarged using reaming tools until the desired bore diameter is achieved. The conduit is typically pulled through the borehole on the final reaming pass. Water based drilling fluids containing bentonite and/or polymers are used during the pilot bore and reaming processes to convey cuttings out of the borehole and to stabilize the borehole.

With HDD there is potential for inadvertent drilling fluid returns to the ground surface via hydrofracture of the soil surrounding the bore or if the bore crosses pre-existing fissures/preferential seepage paths. Inadvertent drilling fluid returns could cause loss of drilling fluid circulation along the bore which may hinder or prevent completion of an HDD installation. It is also understood that inadvertent fluid returns would be a potential environmental concern if drilling fluid migrated to a wetland environment. Therefore, prevention and mitigation of inadvertent drilling fluid returns should be part of planning and construction for an HDD installation.

There is an elevated potential for inadvertent drilling fluid returns at the end of the boring path, in the ditch areas and on the cut back embankment slopes; where overburden soils will be thinnest. If inadvertent drilling fluid returns occur at the cut back slopes and ditch areas, there is potential for localized failure of the slope/highway embankment due to surficial erosion and possible collapse of the borehole.



HDD boring is typically done from the ground surface without the use of deep staging excavations, reducing the extent of ground water control required. At the proposed crossing site the travelled lanes of Highway 401 are approximately 1.5 m to 2.0 m below the ground level of the staging areas. Given the site grades longer boring route(s) may be required to reach the desired installation depth under the highway. Consideration could be given to excavating staging pits for the drilling equipment to lower the start and end elevations of the bore path, thus shortening the boring length. Reference is given to the Staging Excavations section for further recommendations.

The presence of large soil constituents such as cobbles and boulders could also hinder an HDD installation and may necessitate the use of specialized tooling and/or larger HDD drillrigs.

It is expected that the size of the conduit(s), length of the drilling run, compact to very dense consistency of the subsurface soils, and the presence of cobbles and possible boulders will dictate the size of the HDD equipment used. Considering that the bore would be unlined during the HDD process, there would be a potential for loss of ground/sink holes to result on the highway. We therefore do not recommend utilization of HDD.

HDD installations should be carried out in accordance with OPSS 450, Construction Specifications for Pipeline and Utility Installation in Soil by Horizontal Directional Drilling.



5.1.4 Assessment of Tunnelling Methods

The following table summarizes the advantages and disadvantages of the tunnelling methods described.

TUNNELLING METHOD	ADVANTAGES	DISADVANTAGES
Jack and Bore	<ul style="list-style-type: none"> ▪ Contractor availability ▪ Good for shorter tunnel lengths ▪ Good gradient control 	<ul style="list-style-type: none"> ▪ Requires tunnel shafts ▪ Ground water control is required for the bore and shafts ▪ Elevated potential for ground subsidence ▪ Ideally a single 1 m bore should be used to allow removal of cobbles/boulders
Pipe Ramming	<ul style="list-style-type: none"> ▪ Minimal ground water control required along the installation route ▪ Can penetrate soils containing cobbles and boulders 	<ul style="list-style-type: none"> ▪ Requires staging pits ▪ Ground water control is required for the staging pits
Horizontal Directional Drilling	<ul style="list-style-type: none"> ▪ Does not require staging pits ▪ Minimal ground water control required ▪ Alignment can be adjusted to avoid obstructions 	<ul style="list-style-type: none"> ▪ Site grades may require longer bore or staging pits ▪ Potential for inadvertent drilling returns ▪ Larger drilling equipment may be required ▪ Requires drilling fluid to maintain the bore which could allow subsidence

All three installation methods are technically feasible for the crossing. Cognizant of the, site ground water levels and dewatering requirements, the presence of cobbles in the subsurface soils, and relative cost associated with contractor availability, it is recommended that pipe ramming be used for the crossing.



5.1.5 Monitoring

The ground surface over the tunnel route may become distorted and distressed by tunnelling. The most common type of distress is settlement caused by loss of ground around the tunnel. Heave of the ground surface and or inadvertent drilling fluid returns are also possible depending on the type of installation. Mitigation of the distress or distortion on the travelled lanes of Highway 401 would be a major inconvenience to highway users and possibly a safety issue.

Distress at the ground surface is generally prevented or minimized by good construction practices and proper planning. In this regard, preparation of an installation plan as noted above is recommended.

It is also recommended that the project proponent implement a monitoring program to check the condition of the ground over the tunnel before, during and upon completion of construction. The monitoring program should be carried out by a qualified geotechnical consulting firm that is MTO RAQS approved and should conform to the MTO Settlement Monitoring Guidelines for Tunnelling which are presented in Appendix B. As noted in the appendix, monitoring points should be installed over the proposed tunnelling route at a maximum interval of 5 m. Monitoring period should begin prior to tunnelling, extend throughout the duration and continue at least 2 weeks after completion of tunnelling. Measurement of the monitoring points should be done at least 3 times a day for everyday in the monitoring period. A pavement condition survey should also be carried out prior to commencement of construction and following completion of construction.

Monitoring points should be marked using a method approved by MTO. Monitoring points should also be functional throughout the monitoring period and should not deteriorate because of highway traffic, maintenance activities and weather conditions. The storm sewer should also be checked for disturbance from tunnelling, this should involve visual inspections of the pipe prior to construction, during construction when the tunnelling will be done directly below the pipe, and after tunnelling has been completed.



If distress is observed during construction the contractor should be informed and corrective action should be undertaken immediately. Specific corrective action will be dependent on the nature of the distress and type of installation. Regardless, the process should be outlined in the monitoring program and be part of the contingency actions in the contractor's installation plan.

Settlement or heave of the roadway from a pipe ramming installation carried out in accordance with the recommendations noted in this report should be less than 10 mm. If settlement or heave of the ground surface exceeds 10 mm, the construction process should be reviewed and adjusted to mitigate further disturbances for the remainder of the tunnelling work.

If total settlement or heave exceeds 15 mm, tunnelling operations should be terminated, the site secured against further deterioration, and mitigative action should be undertaken immediately to reinstate the roadway, ditches and/or the existing storm sewer.

All actions to prevent, secure, or mitigate destruction or damage to the highway and associated features should be done in accordance with, and approved, by MTO.

5.2 Staging Excavations

It is anticipated that open cut excavations will be used for staging areas and tie points to the tunnelling segment. These excavations are understood to be located outside of the Highway 401 right of way.

Clearing and grubbing will be required to remove trees prior to excavation on the Doon Campus grounds north of the highway. Reference is given to OPSS 201, 503 and 565 for specifications associated with site preparation.



5.2.1 Excavation and Ground Water Control

Excavations for access pits and tie-in locations will extend through surficial topsoil and into predominantly non-cohesive native soils. Provided adequate ground water control is achieved, the onsite soils are classified as Type 3 materials as defined in the Occupational Health and Safety Act (OHSA). Excavations within Type 3 soil that are to be entered by workers, may not be steeper than one horizontal to one vertical (1H:1V) from the base of the excavation. Workers should not enter an unprotected excavation if there is evidence of ongoing ground water seepage in the banks.

Excavations for staging pits are expected to extend about 2 m below the ground water level and will require sophisticated ground water control measures such as keg wells, well point dewatering or perimeter sheet pile cut-offs. The extent of ground water control will depend on the depth of excavation below the ground water level. The actual dewatering methods should be established at the contractor's discretion within the context of a performance specification for the project. Regardless of the dewatering method chosen, the hydraulic head and ground water inflow must be properly controlled to ensure a stable and safe excavation and to facilitate construction. The design of the dewatering system should be specified to maintain and control ground water at least 0.3 m below the excavation base level, in order to provide a stable excavation base throughout construction.

It should be noted that, under the Ontario Water Resources Act, the Water Taking and Transfer Regulation 387/04, a Permit to Take Water (PTTW) from the Ministry of Environment (MOE) is required if the dewatering discharge is greater than 50,000 L/day.

Excavations that extend deeper than 1 m below the ground water level into the wet sand and silt soils will require rigorous dewatering and pumping rates will likely exceed 50,000 litres per day. In this case, a PTTW will be required together with a hydrogeological study in support of the PTTW application. A detailed review of the final design invert levels relative to the observed ground water levels is recommended to determine if detailed hydrogeologic work will be required.



Reference is also given to OPSS 517 and 518 which pertain to construction dewatering.

Construction stage dewatering is expected to have negligible impact on existing infrastructure, provided the existing infrastructure is founded on competent native soils such as compact to dense sandy gravel, sands and silts, or hard clayey silt till. Aggressive dewatering from staging excavations is not expected to impact the ground surface or the median sewer.

Braced excavations may be desired at staging pits to reduce the excavation area, for the predominately cohesionless soil stratigraphy encountered, the bracing system may be designed using a rectangular stress distribution in accordance with methods outlined in the Canadian Foundation Engineering Manual (CFEM). The system design should also consider load effects from the adjacent embankments, structures, construction equipment and ground water pressure. Reference is given to OPSS 538 and 539 which pertains to excavation support and protection systems.

Partial sheet pile cutoffs may be used for braced excavations and to limit ground water inflow. The soil parameters presented in the following table may be used for the design of the sheet piling at the site. The parameters are based on the Rankin method of analysis which ignores wall friction. To provide adequate ground water control and to prevent piping in the base, the sheeting should extend at least 0.5 m into the clayey silt till soils, although greater penetration depths may be necessary to develop adequate passive resistance at the toe.

SOIL TYPE	SANDY GRAVEL	SAND AND SILT	CLAYEY SILT TILL
Coefficient of Active Earth Pressure K_a	0.28	0.33	0.36
Coefficient of Passive Earth Pressure K_p	3.54	3.00	2.77
Angle of Internal Friction ϕ	34	30	28
Cohesive Strength c_u (kPa)	--	--	200
Bulk Unit Weight γ (kN/m ³)	22.0	21.0	20.0
Buoyant Unit Weight γ' (kN/m ³)	12.2	11.2	10.2



Although boulders were not encountered in the boreholes during the field investigation their presence should be anticipated within the native deposits in addition to the presence of cobbles.

It is recommended that test pits be excavated during the tendering stage so contractors bidding on the project can evaluate for themselves the soil and ground water conditions to be encountered and assess the dewatering requirements.

6. CLOSURE

The field work was carried out under the supervision of Mr. D. Brice working under the direction of Mr. W. Loghrin, P.Eng. The drilling equipment was supplied and operated by Geo-Environmental Drilling. The laboratory work was carried out in the PML Kitchener laboratory.

This revised report was prepared by Mr. R. Agahzadeh, P. Eng., Manager, Geotechnical Services, and reviewed by Mr. G. Mitchell, MEng, P.Eng., Director, Branch Manager, Kitchener. Independent review of the report was carried out by Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

We trust this report has been completed within the terms of reference and is sufficient for your current needs. Should you have further questions, do not hesitate to contact our office.

Sincerely

Peto MacCallum Ltd.



Romin Agahzadeh, P.Eng.
Manager, Geotechnical Services

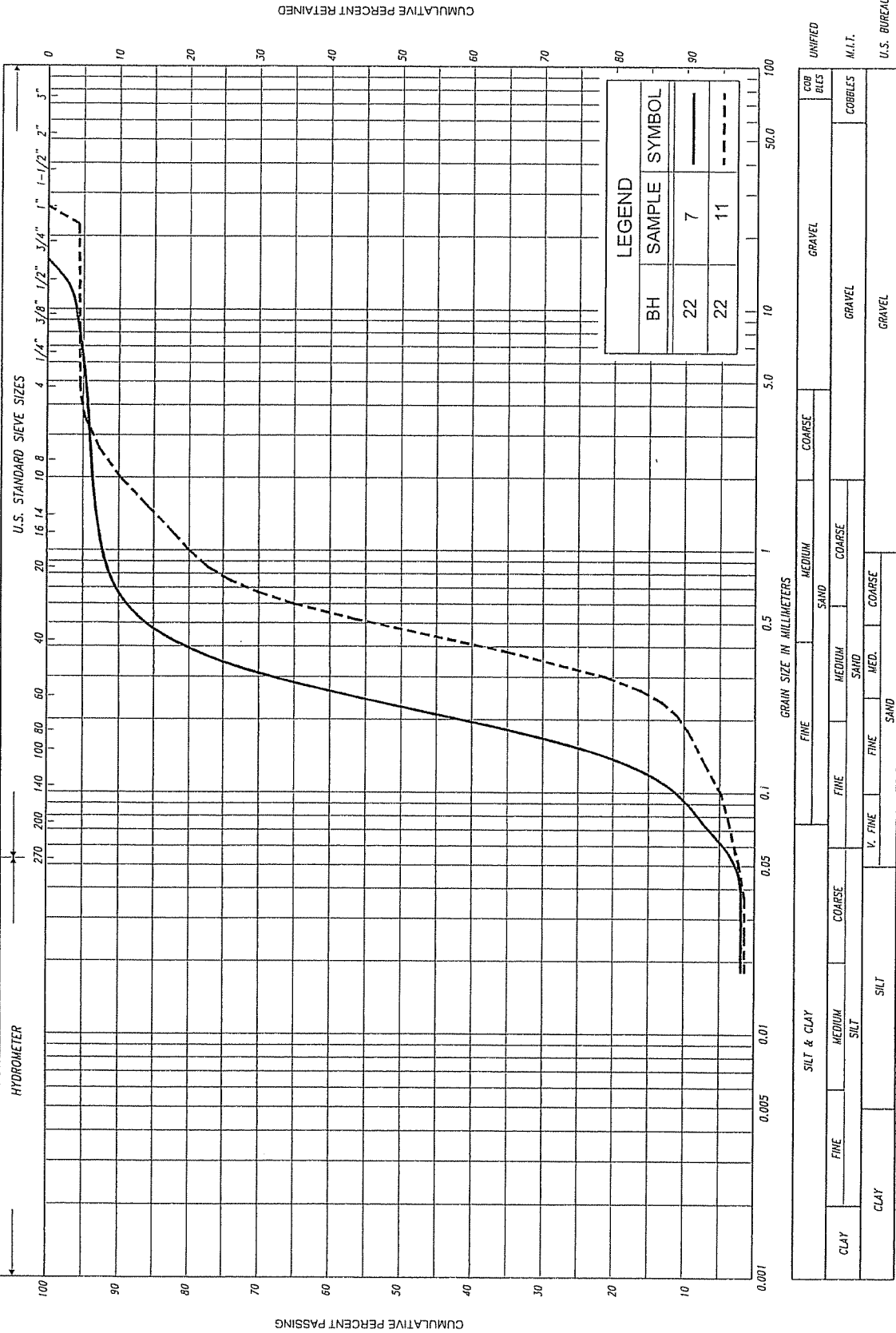


Gerry Mitchell, MEng, P.Eng.
Director
Branch Manager



Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

RA/GPM/BRG.sh



GRAIN SIZE DISTRIBUTION

SAND, trace gravel, trace silt, trace clay

FIG No. GS-2

HWY: 401

G.W.P. No.

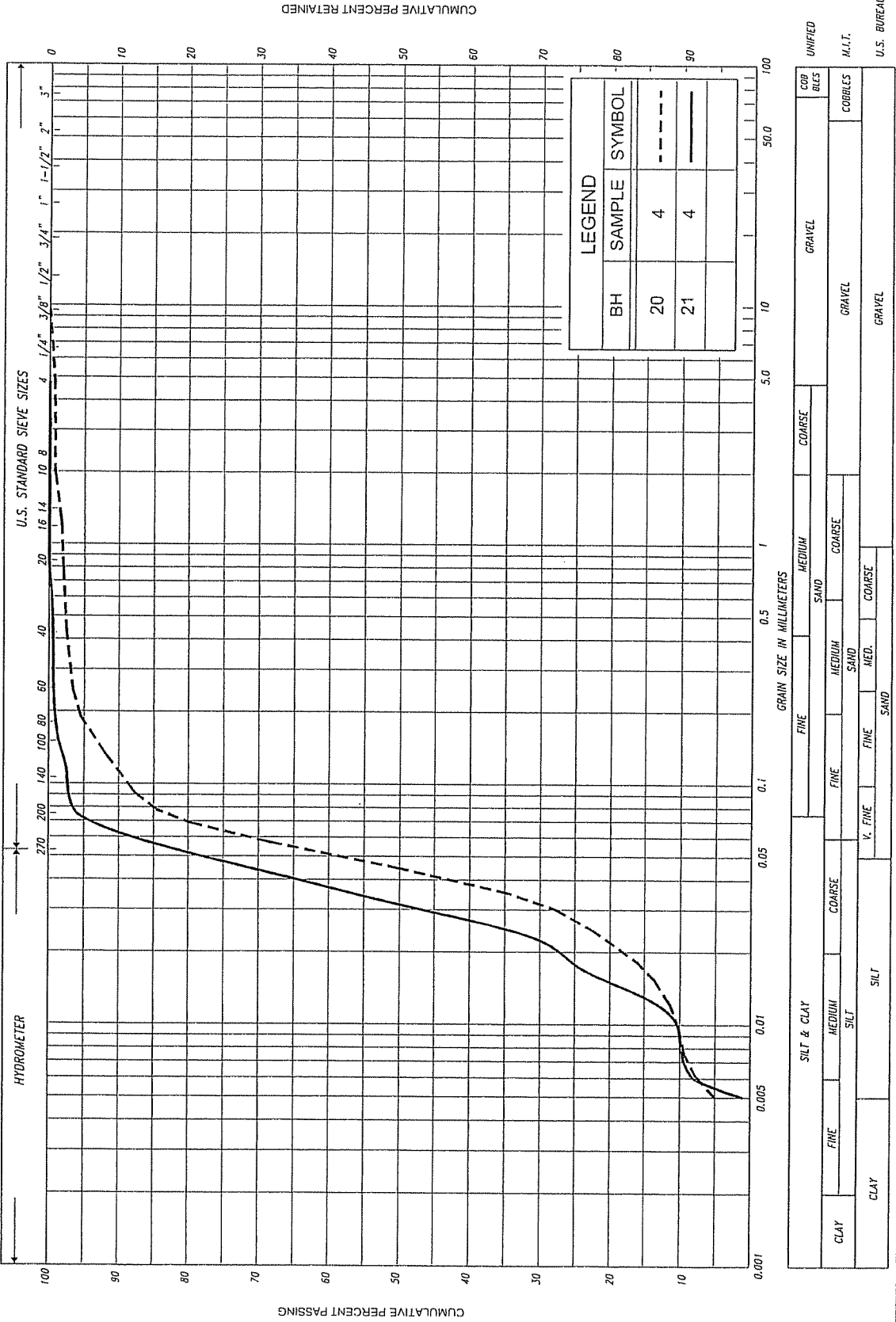


FIG No. GS-3

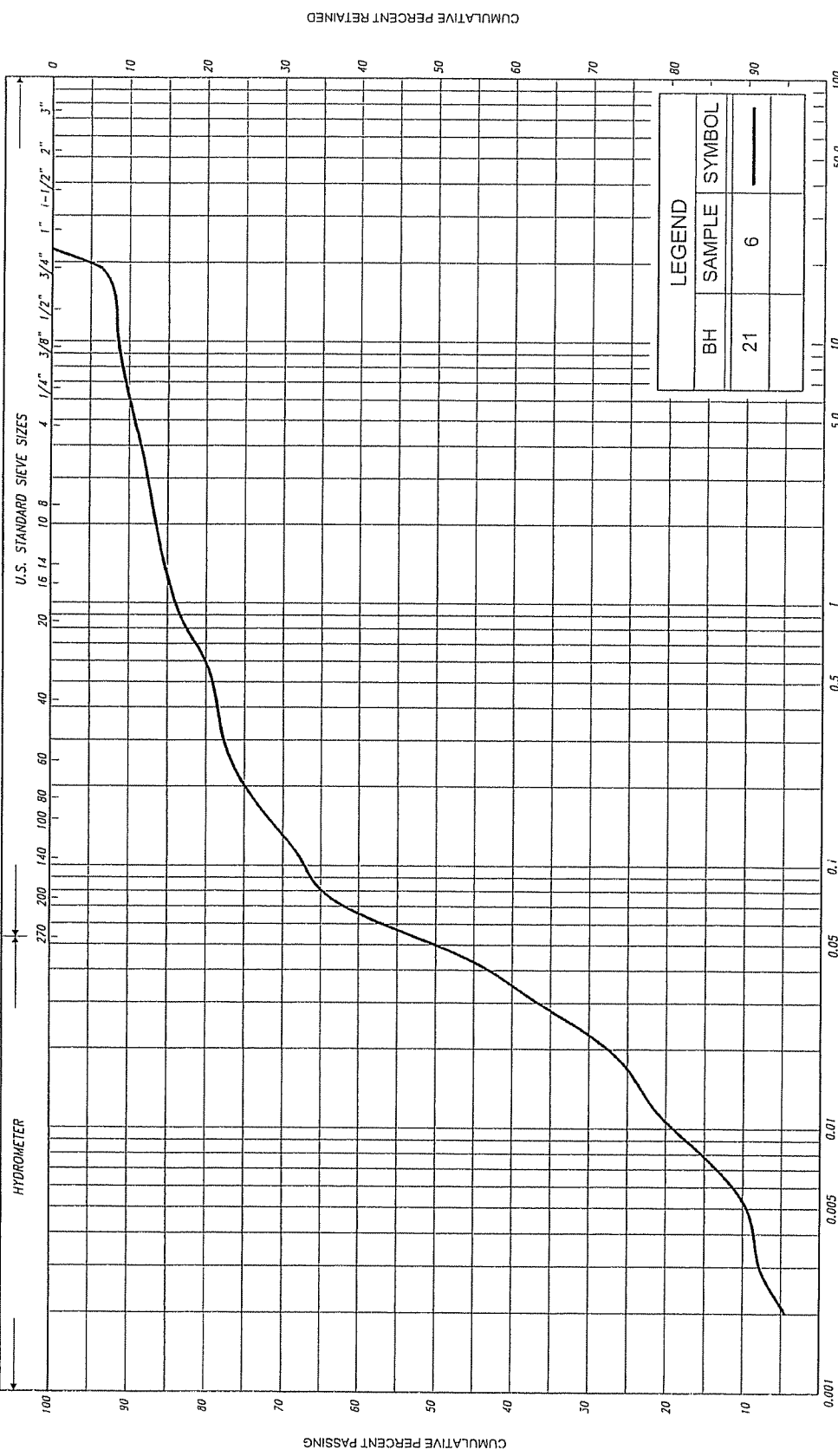
HWY: 401

G.W.P. No.

GRAIN SIZE DISTRIBUTION

SILT, trace to some sand, trace gravel

U.S. BUREAU



BH	SAMPLE	SYMBOL
21	6	—

CLAY		SILT & CLAY		SAND		GRAVEL		COBBLES		UNIFIED	
FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE	
V. FINE		FINE		MEDIUM		COARSE		FINE		MEDIUM	
SILT		SAND		GRAVEL		COBBLES		GRAVEL		COBBLES	
CLAY		SILT		SAND		GRAVEL		COBBLES		UNIFIED	
CLAY		SILT		SAND		GRAVEL		COBBLES		M.I.T.	
CLAY		SILT		SAND		GRAVEL		COBBLES		U.S. BUREAU	

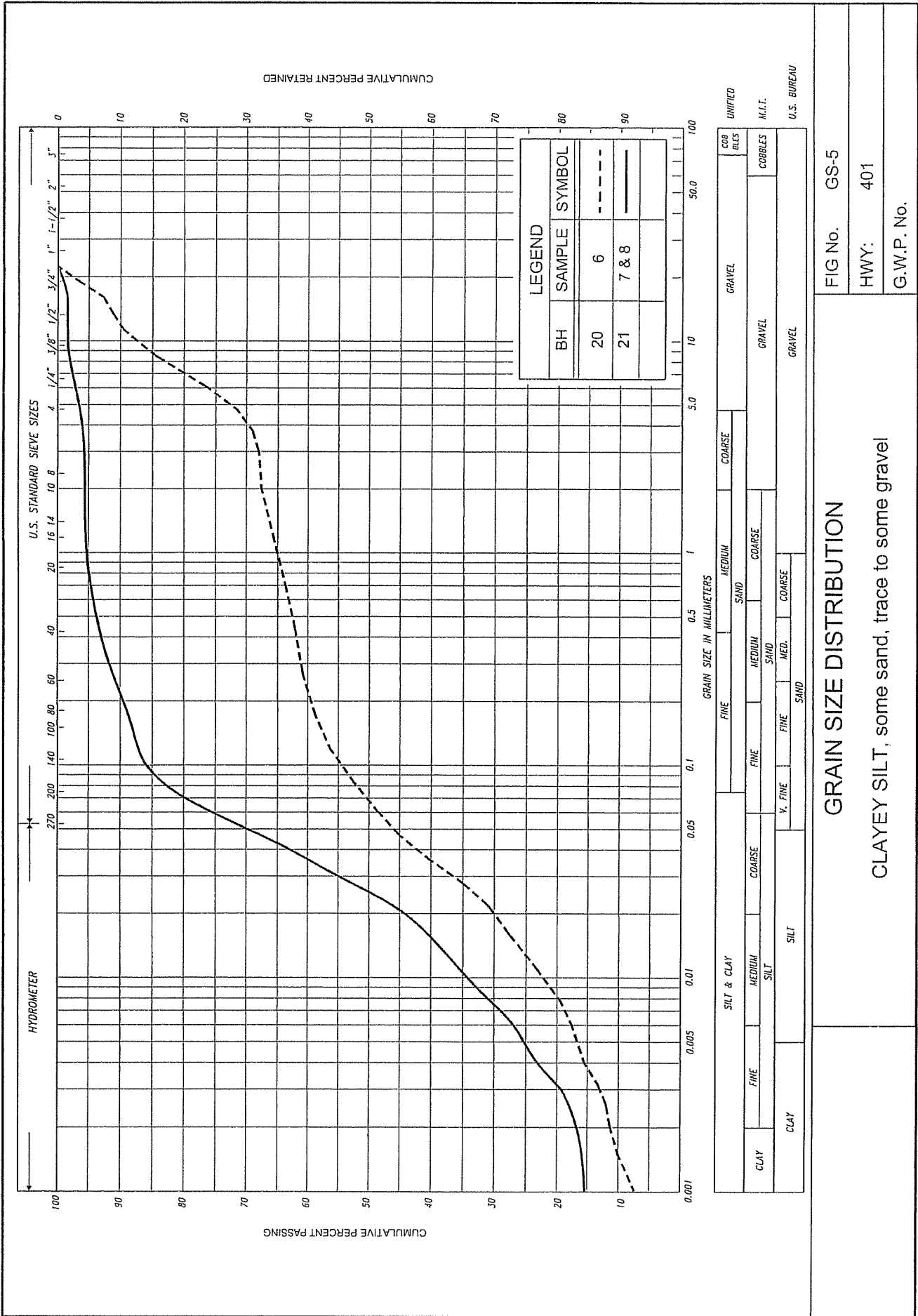
GRAIN SIZE DISTRIBUTION

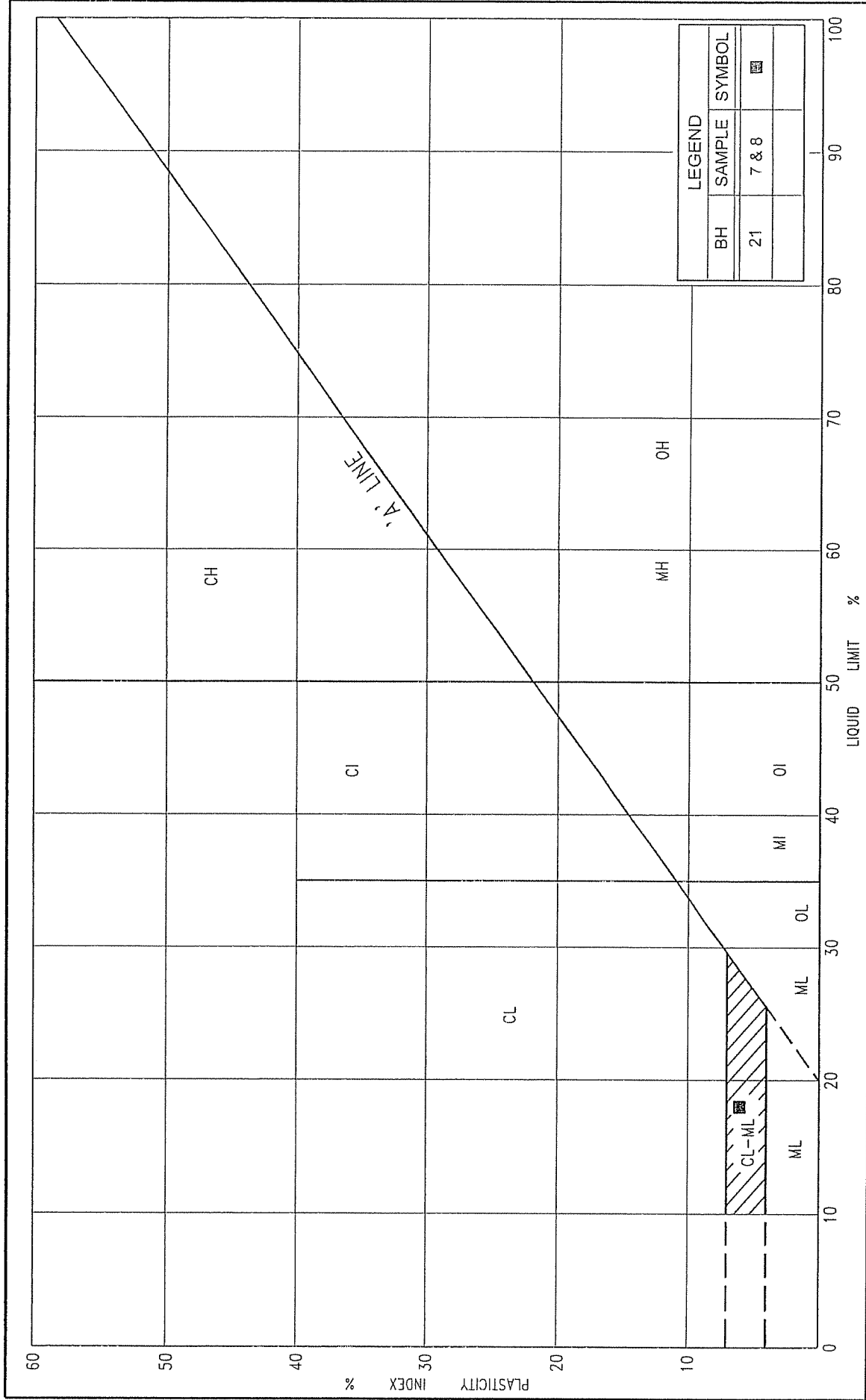
SILT, with sand, some gravel, trace clay

FIG No. GS-4

HWY: 401

G.W.P. No.





PLASTICITY CHART		FIG No.	PC-1
CLAYEY SILT, some sand, trace gravel		HWY:	401
		G.W.P. No.	

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{v0}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
T_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL




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

RECORD OF BOREHOLE No 20

1 of 1

METRIC

DIST Waterloo LOCATION Sta. 8+351, o/s 1.0m Rt. of Forcemain CL ORIGINATED BY D. Brice
 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY W.L.
 DATUM Geodetic DATE July 15, 2010 CHECKED BY R. A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
292.1 0.0	Ground Surface						20	40	60	80	100						
291.7 0.4	Topsoil		1	SS	14												
1.0	Sandy gravel some silt, trace clay cobbles																
2.0	Very dense Brown Moist		2	SS	50/13cm											40 33 (17)	
3.0	289.2 2.9																
4.0	Silt some sand, trace gravel		3	SS	34												
5.0	Compact Brown Moist to dense to wet		4	SS	21											1 16 83	
6.0	287.3 4.8																
7.0	Clayey silt some sand, trace gravel silty fine sand layers		5	SS	36												
8.0	Hard Brown Moist to grey		6	SS	45											28 20 41 11	
9.0	(TILL)		7	SS	50/8cm												
10.0			8	SS	59												
11.0			9	SS	91/28cm												
12.0			10	SS	50/13cm												
13.0			11	SS	91/28cm												
14.0	281.0 11.1																
15.0	End of borehole																
	Samples 2, 7, 9, 10 & 11: Sampler bouncing																
	* 2010 07 15																
	▽ Water level observed during drilling																
	Piezometer Legend:																
	 Bentonite seal with two 25mm dia. pipes																
	 Filter sand																
	 PVC screen																
	Water Level Reading:																
	Date Depth Elev.																
	08/23/2010 4.1 288.0																

RECORD OF BOREHOLE No 21

1 of 1

METRIC

DIST Waterloo LOCATION Sta. 8+306, o/s 2.5m Lt. of Forcemain CL ORIGINATED BY D. Brice
HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY W.L.
DATUM Geodetic DATE July 15, 2010 CHECKED BY R. A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
290.1	Ground Surface													
0.0	140mm asphaltic concrete over 1860mm of gravelly sand, trace silt													
1.0	Compact to Brown Moist very dense (PAVEMENT FILL)		1	SS	68									
2.0			2	SS	45									
287.9	Silt, trace sand													
2.2	Compact Brown Moist to wet		3	SS	21									
3.0			4	SS	19									
4.0			5	SS	19									
285.6	Silt with sand, some gravel		6	SS	42									
4.5	Dense Brown Moist to wet (TILL)		7	SS	50/13cm									
5.0	Clayey silt some sand, trace gravel layers of silt and sand		8	SS	90									
5.2	Hard Brown 'Moist to grey to wet (TILL)		9	SS	66									
6.0			10	SS	54									
7.0			11	SS	81									
8.0			12	SS	86/23cm									
280.6	End of borehole													
9.5	Samples 7 and 12: Sampler bouncing													
10.0														
11.0														
12.0	* 2010 07 15 ▽ Water level observed during drilling													
13.0														
14.0														
15.0														

RECORD OF BOREHOLE No 22

1 of 1

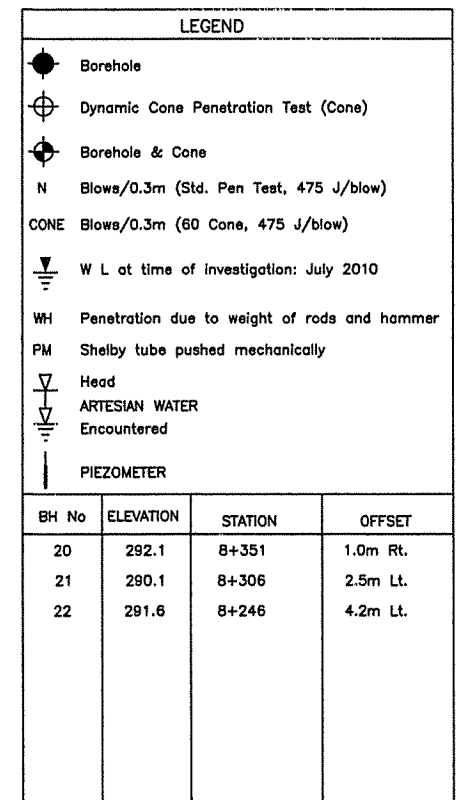
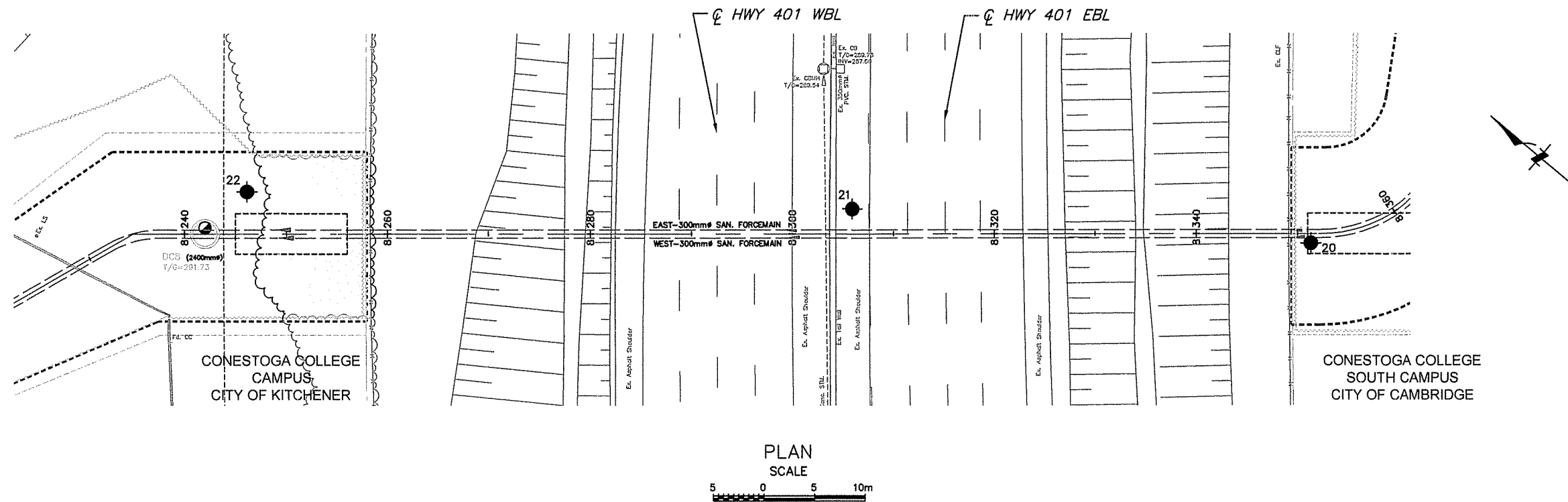
METRIC

DIST Waterloo LOCATION Sta. 8+246, o/s 4.2m Lt. of Forcemain CL ORIGINATED BY D. Brice
 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY W.L.
 DATUM Geodetic DATE July 16, 2010 CHECKED BY R. A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
					WATER CONTENT (%)												
291.6 0.0	Ground Surface Topsoil																
291.0 0.6	Silt, some gravel trace sand, trace clay		SS	1	10												
290.4 1.2	Compact Brown Moist		SS	2	26												
	Sandy gravel trace silt, trace clay cobbles		SS	3	60												
	Very dense Brown Moist		SS	4	91/25cm												
			SS	5	85												
287.9 3.7	Sand, trace silt trace clay, trace gravel cobbles		SS	6	33												
	Compact Brown Wet to dense		SS	7	22												
			SS	8	26												
			SS	9	32												
			SS	10	64												
			SS	11	37												
283.2 8.4	Clayey silt some sand, trace gravel		SS	12	50/13cm												
	Hard Grey Moist																
	(TILL)		SS	13	75												
281.4 10.2			SS	14	50/13cm												
	End of borehole																
11.0	Samples 4, 12 & 14: Sampler bouncing																
12.0	* 2010 07 16 ▽ Water level observed during drilling																
	<u>Piezometer Legend:</u>																
13.0	■ Bentonite Seal																
	□ Filter sand																
	□ PVC screen																
14.0	<u>Water Level Reading:</u>																
	Date Depth Elev.																
	08/23/2010 4.2 287.4																
15.0																	

CONT No
GWP No

BLAIR SANITARY FORCEMAIN CROSSING
UNDER HIGHWAY 401
BOREHOLE LOCATIONS AND SOIL STRATA

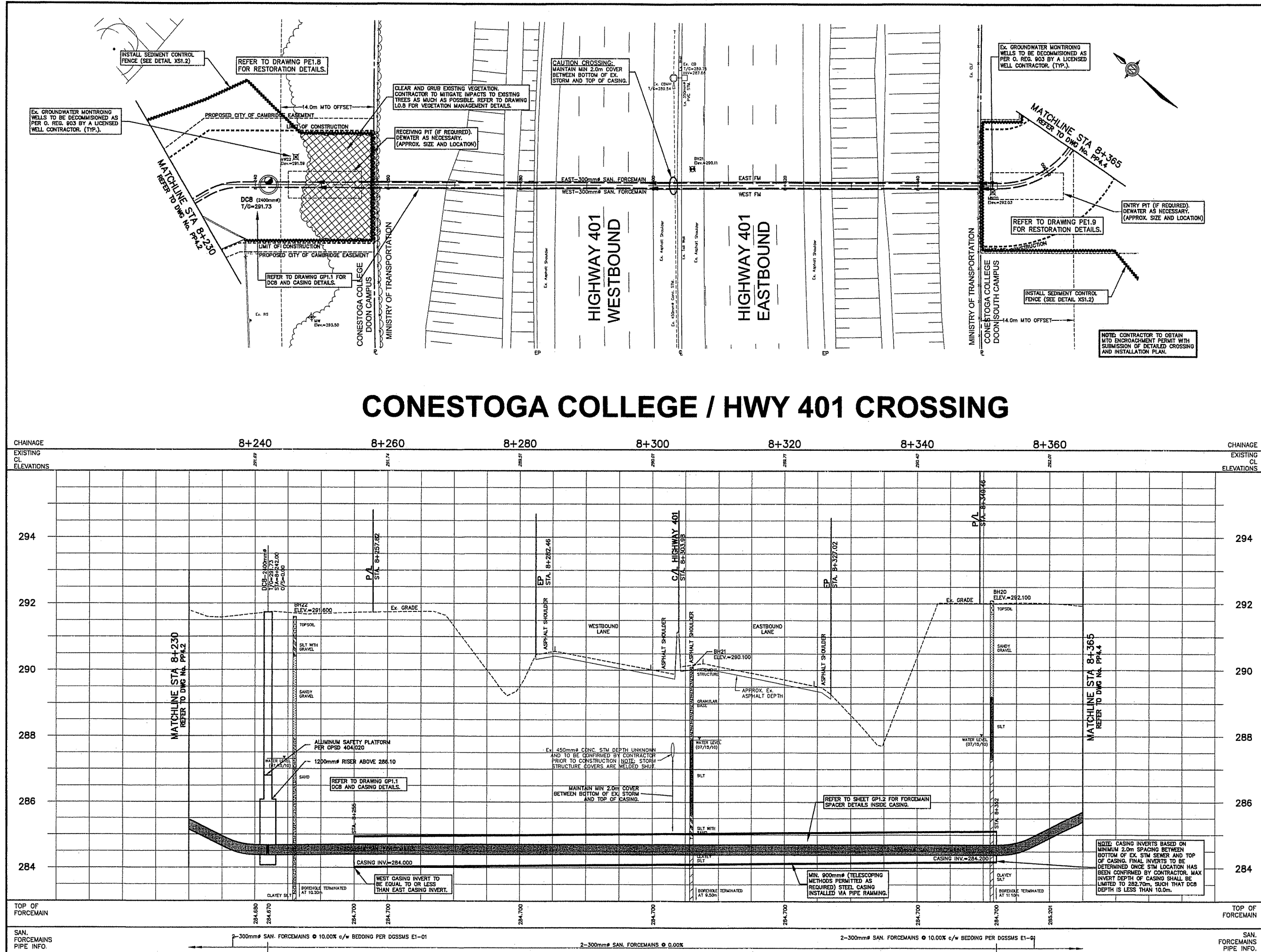


REVISIONS							
DATE		BY		DESCRIPTION			
Geocres No. 40PB-190							
HWY No. 401						DIST	Waterloo
SUBM'D	RA	CHECKED	RA	DATE JUNE 24, 2011		SITE	
DRAWN	NA	CHECKED		APPROVED		DRAWN	1



APPENDIX A

DRAWING PROVIDED BY MTE



City of
Kitchener

THIS
SHEET

City of
Cambridge

KEY PLAN N.T.S.

THE PARCEL INFORMATION SHOWN ON THIS MAP IS COMPILED FROM VARIOUS SOURCES & IS NOT WARRANTED AS TO ITS ACCURACY BY THE MUNICIPALITY. VERTS AND REMAINS THAT LOT LINES & LEGAL DESCRIPTIONS MUST BE CONFIRMED AT THE LAND REGISTRY OFFICE. THIS IS NOT A LEGAL DOCUMENT.

THE POSITION OF POLE LINES, CONDUITS, WATERMANS, SEWERS, & OTHER UNDERGROUND & ABOVE GROUND UTILITIES ARE NOT PRESENTLY SHOWN ON THE MAP. THE CONTRACTOR SHALL BE RESPONSIBLE FOR LOCATING ALL SUCH UTILITIES & STRUCTURES & SHALL ASSUME ALL LIABILITY FOR DAMAGE TO THEM.

BENCHMARK: ELEVATION 275.402
MONUMENT CA-184 (MTE C.P. 9035)
CONCRETE MONUMENT BY FOUNTAIN STREET BRIDGE OVER GRAND RIVER ON EAST END, SOUTH SIDE OF ROAD, 4.5m EAST OF BRIDGE END AND 0.3m SOUTH OF EXTENSION OF SOUTH SIDE OF BRIDGE.

NOTE:
1. REFER TO DWG No. XS1.1 & XS1.2 FOR TYPICAL CROSS SECTIONS, DETAILS & NOTES.
2. GEOTECHNICAL INFORMATION SHOWN IS FOR REFERENCE ONLY. REFER TO PETO MACGILLUM LTD. GEOTECHNICAL INVESTIGATION REPORT 10KF029, "FOUNDATION INVESTIGATION AND DESIGN REPORT FOR BLAIR BUSINESS PARK SANITARY SEWER-HIGHWAY 401 CROSSING" (SEPT. 7 2010) FOR FURTHER DETAILS.

BOREHOLE LEGEND

- EXISTING PAVEMENT STRUCTURE
- TOPSOIL
- GRANULAR BASE
- SILT
- SILT AND SAND
- SANDY GRAVEL
- CLAYEY SILT
- SAND

NO.	REVISION	BY	DATE
1	ISSUED FOR TENDER	DJW	02/11/11
2	ISSUED FOR MTO REVIEW	DJW	01/18/11
3	ISSUED FOR MDE APPROVAL	DJW	01/07/11
4	ISSUED-TRENCHLESS REQUALIFICATION	DJW	09/30/10

MTE
25 YEARS 1985-2010
Municipal Division
(519)743-6500 www.mte85.com

D. J. WILHELM
PROFESSIONAL ENGINEER
PROVINCE OF ONTARIO
STAMP

SURVEY DATA:
SURVEY BY: BKH
DATE: APRIL 2010
FIELD BOOK: 1738SDR
REQUISITION NO:
CHECKED BY: DJW
DATE:

DRAWING INFORMATION:
DRAWN BY: JXR
DATE: JULY 2010
DESIGN BY: CLS/NPD
DATE:
CHECKED BY: DJW
DATE:

The Corporation of the City of Cambridge
Transportation & Public Works Department

BLAIR SANITARY FORCEMAINS CONSTRUCTION
CONESTOGA COLLEGE / HWY 401 CROSSING
STA. 8+230 TO STA. 8+365
PROPOSED CONDITIONS PLAN

SCALE:
HORIZONTAL: 1:250
VERTICAL: 1:50

DRAWING NO:
PP4.3



APPENDIX B

MINISTRY OF TRANSPORTATION'S “GUIDELINES FOR FOUNDATION ENGINEERING – TUNNELLING SPECIALTY FOR CORRIDOR ENCROACHMENT PERMIT APPLICATION”

Guidelines For Foundation Engineering – Tunnelling Specialty For Corridor Encroachment Permit Application

These guidelines specify MTO's minimum requirements for the Foundation Engineering – Tunnelling Specialty component of submissions from proponents of development within the Ministry of Transportation's (MTO) corridor permit control area. The Foundation Engineering – Tunnelling Specialty component of submissions is a requirement for the permit application only and do not cover all the design requirements.

The complexity ratings of Foundations Engineering services are defined in Table 1.

Table 1: Complexity ratings for tunnelling specialty services

Highway Classification	Tunnel Excavation Diameter (ϕ)					
	≤ 1 m		>1 m & ≤ 2 m		>2 m	
	Minimum Overburden Cover * (m)					
	≥ 3 ϕ (or 1.5 m whichever is greater)	< 3 ϕ (or 1.5 m whichever is greater)	≥ 3 ϕ	< 3 ϕ (or 1.5 m whichever is greater)	≥ 3 ϕ	< 3 ϕ (or 1.5 m whichever is greater)
Kings Highway	Low	Medium	Medium	High	High	High
400 Series Freeway	Medium	High	High	High	High	High

*Minimum overburden cover is the vertical distance measured from the lowest ground elevation to the crown of the tunnel.

Foundations Engineering consultants that are registered in the MTO consultant acquisition system (RAQS) at complexity ratings identified in Table 1 are eligible to provide Foundations Engineering services for this project. Alternatively, the proponents may propose a Foundations Engineering consultant that is not registered in RAQS, in which case, the proponent must submit sufficient documentation to demonstrate that the consultant's qualifications meet or exceed the RAQS complexity requirements.

For Engineering Materials Testing and Evaluation, the consultant shall be qualified for Soil and Rock testing of complexity level at least equal to that identified for this project.

Consultant services shall be provided in accordance with the most recent editions of the Canadian Highway Bridge Design Code (CHBDC), and the 'Guideline for Professional Engineers Providing Geotechnical Engineering Services' published by the Professional Engineers of Ontario.

The designated principal contact identified for Foundations Engineering services by MTO shall sign, and where required, seal, all submissions and correspondence that are submitted to MTO.

Services include, but are not restricted to, conducting a site investigation that shall be of sufficient scope to verify design assumptions and to provide the contractor with adequate subsurface information for design and construction planning.

Sufficient subsurface (factual) information is required to determine the vertical and horizontal extent of subsurface materials (including both soil and rock) and their pertinent engineering properties and groundwater conditions.

Subsurface information is usually acquired by advancing boreholes, laboratory testing of soil samples and rock core samples, performing in-situ tests such as standard penetration tests, dynamic cone tests, and piezocone tests (CPTU) and test pits.

Minimum requirements for Subsurface Investigation and Recommendations

A minimum of one borehole shall be advanced at each end of tunnel crossing. The boreholes shall be located outside but within 2 m of the tunnel's excavated footprint.

Spacing between the boreholes shall not exceed 50 m. In case of larger spacing between the boreholes, additional boreholes shall be advanced except where significant traffic disruptions might occur and where consistent conditions are evident.

Boreholes shall be advanced to 3 tunnel diameters (excavated diameters) below invert. If bedrock is encountered earlier, the borehole shall advance to at least 3 m below the invert of tunnel into the bedrock.

The investigations, if required, shall be supplemented with additional and deeper boreholes to verify consistent conditions and existence of boulders within critical foundation zones.

Sampling and testing, consisting of Standard Penetration Test, thin wall tube sample, rock cores, and MTO Field Vane Test where appropriate, shall be conducted to develop a comprehensive subsurface model. Semi-continuous sampling at 0.75m (2.5ft) intervals is required within overburden; whereas, sampling interval of 1.5m (5.0ft) is required below the tunnel invert.

Where encountered, the bedrock-soil interface shall be determined by geological definition and not by the material properties.

All aspects of implementation of means of subsurface investigations including, but not limited to, planning, licensing, construction, maintenance, abandonment, and reporting, shall be in accordance with Ministry of the Environment Regulation 903 and its amendments (the water well regulation under the OWRA).

Boreholes and piezometer tubes shall be backfilled with a suitable bentonite/cement mixture. Test pits shall be backfilled with suitable material and either re-vegetated or otherwise protected from erosion. Temporary open holes shall be adequately covered.

Holes in roads shall be backfilled as required to prevent future settlement and acceptably patched where pavement surfaces have been damaged. Backfilling requirements shall be described in the Foundation Investigation and Design Report.

Where encountered, artesian groundwater conditions shall be sealed. Details of the artesian condition and the sealing operation shall be included in the Foundation Investigation Report.

Fieldwork shall be carried out in accordance with the Occupational Health and Safety Act.

Traffic protection in accordance with MTO requirements shall be provided during the course of any field investigations. However, where significant traffic disruptions might occur, boreholes may be relocated or numbers reduced with MTO's approval.

The locations and ground surface elevations of all boreholes, test pits and soundings shall be surveyed and referred to fixed reference points and data. Locations are to be identified by co-ordinates (Northing and Easting). The vertical accuracy of survey readings shall be within 0.1m; whereas, horizontal accuracy shall be within 0.5m.

Minimum Laboratory Testing Requirements:

Laboratory testing shall consist of routine testing of 25% of samples. One routine lab test is defined as natural water content plus Atterberg Limit plus grain size distribution tests. Complex laboratory testing is defined by all other tests including compressive strength, shear strength, consolidation, permeability and triaxial testing. Laboratory testing requirements shall be supplemented with additional routine and complex tests if required to verify strata boundaries and properties and behaviour of critical subsurface zones.

Borehole Log Preparation and Foundation Drawing:

Borehole log sheets, figures and drawings shall be prepared in accordance with MTO standards. The Foundation Drawing shall consist of a plan showing the locations of all borings, test pits and soundings and various stratigraphical longitudinal profiles and stratigraphical cross-sections at each tunnel structure foundation element and groundwater levels.

Minimum Requirements for the Foundation Investigation and Design Report:

A Foundation Investigation and Design Report shall consist of the factual subsurface information (including the field and laboratory test information) and the recommendations required for foundation design.

The report shall be signed and sealed by two professional engineers, registered with the Professional Engineers of Ontario, representing the consulting firm; one of them shall be the firm's designated principal contact for MTO's Foundations Engineering projects.

- The Foundation Investigation component of the report shall contain:
- Site Description - including topography, vegetation, drainage, existing land use, and structures.
- Investigation Procedures - including site investigation and lab testing procedures.
- Description of Subsurface Conditions - including soil, boulders, rock and groundwater conditions.
- Miscellaneous Section - that identifies the name of the drilling company, the laboratory where testing was performed, the persons who carried out the field supervision, and those who wrote and reviewed the report.

The Foundation Design component of the report shall present discussion and recommendations for design. The consultant shall analyse field data and test results and make comprehensive and practical recommendations pertaining to temporary, interim and permanent conditions at the Project.

The consultant shall identify and evaluate all reasonable and appropriate alternatives for the proposed tunnel crossing. Alternatives may include, but not limited to, jack & bore, pipe jacking using TBM, pipe ramming, micro-tunnelling (if economically feasible), utility tunnelling using TBM (two pass system), Horizontal Directional Drilling (HDD) and cut and cover methods.

The consultant shall identify and present overview assessments of the advantages, disadvantages, costs and risks/consequences of alternative tunnelling methods in a table. The report should conclude a preferred alternative from foundation engineering and cost effectiveness perspective.

In the development and design of the preferred alternative, the Consultant shall, as applicable, address:

- impacts on the land use and property, traffic and transportation, and environment,
- length and diameter constraints
- control of face stability
- capability of boulder excavation
- evaluation of temporary and permanent support
- alignment control
- estimated settlements and heave and management of these deformations
- special access and egress requirements for TBM's and other similar equipment such as those used for the Jack & Bore method including recommendations for vertical shafts and jacking pits;
- shored and un-shored alternatives for open-cut excavation;
- groundwater control & dewatering;
- the long-term stability of the tunnel;

- relative costs; and
- traffic management and contractor access for each alternative.

If borehole logs available from previous projects are included to meet the requirements of field investigations then the accuracy of subsurface information from these boreholes remains the responsibility of consultant except in situations where MTO specify the use of previous boreholes. Borehole logs from previous studies that are appended to the report shall be reformatted to meet the MTO's requirements.

The final foundation recommendations shall detail the geometric, material and strength properties of the new tunnel crossing plus the liner, bedding and backfill requirements, and slope and embankment restoration requirements. The invert elevation should be assessed in view of the subsurface conditions and the anticipated open face stability control.

The consultant is responsible for developing contract documents sufficient to implement the design. This typically includes:

- Contract specifications for materials and specialized construction activities, and
- Recommendations for methods of overcoming anticipated construction problems, in particular, those relating to dewatering, boulder excavation, alignment control and the stability of excavations and embankments. .

The consultant shall develop a detailed instrumentation and monitoring program that meets the requirements of these guidelines. (see Appendix for typical settlement monitoring guidelines).

The consultant is responsible for preparing Traffic Control Plans and to obtain approvals and an Encroachment Permit from the Ministry, which are required for lane closures necessary to install the settlement monitoring points.

The tunnelling consultant shall ensure that the foundations engineering component of the project is adequately reflected in the design drawings, specifications and related contract documents.

Written confirmation is required from the Proponent and the tunnelling consultant that the design package submitted to MTO have been reviewed by the tunnelling consultant and that all recommendations have been satisfactorily incorporated in the contract package.

APPENDIX: SETTLEMENT MONITORING GUIDELINES - TUNNELING

The purpose of settlement monitoring is to prevent damage to existing utilities and highway structures along the tunnel alignment. Ground settlement include settlement due to lost ground and dewatering/drainage.

Instrumentation Arrays

All measurement points shall be installed and surveyed before the start of excavation to establish benchmarks/baseline.

Surface Monitoring Points

Surface monitoring points will be installed to cover the whole length of the tunnel with in the right of way under the jurisdiction of MTO (Figure 1).

Surface monitoring points will be located at not greater than 5m intervals along the tunnel alignment. The surface monitoring will be identified using paint marks on the pavement. Surface monitoring points installed on the unpaved right of way shall be founded below frost penetration depths. The interval and/or marking of the points should be changed with MTO's approval where traffic disruptions might occur.

The final instrumentation plan should be finalised when Contractor's proposed construction method is available.

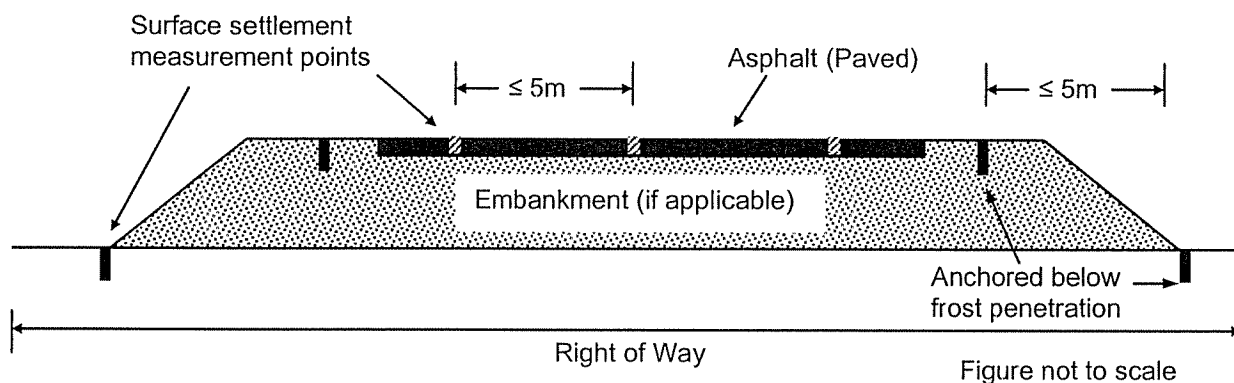


Figure 1: Typical configuration of surface settlement monitoring points along the tunnel alignment.

Condition Survey

A condition survey for the pavement will be carried out prior to commencement of construction and documented for the purpose of requirement of restoration. The condition survey shall document visible flaws such as cracks, distortions and deviations, heaves, and depressions. This surface survey will be completed during the installation of the monitors and again once the tunnel has been completed.

Reading Frequency

An average of at least two readings shall be taken to establish the initial conditions.

The reading and collection of data from the surface monitoring points shall be read and recorded by the Contractor during the construction period and after construction for period of at least 2 weeks provided that further settlement has stopped.

A minimum of three (3) sets of reading be taken daily, provided that movements are within anticipated limits. Otherwise, the frequencies should increase according to a pre-planned interval.

Monitoring of movements is required during work stoppages, such as during non-operation period (off-shifts) or weekends. A minimum of three (3) sets of readings should be taken daily.

Measurements of the monitoring points shall be reported promptly to MTO for review.

Data Collection and Data Transfer

A procedure is required to be established in consultation with MTO so that the monitoring data and the interpreted data will reach all parties as soon as necessary. The contract administrator/consultant and the Contractor should interpret monitoring data as needed for the purpose of on-going construction. The Foundation Engineer should be contacted for technical support to the prime Consultant in the interpretation of ground movements and review of the Contractor's response when Review and Alert Levels are reached.

Criteria for Assessment

The acceptable surface settlement (or heave) will be according to criteria as specified below.

Baseline Reading – A baseline reading of the instrumentation shall be taken prior to commencement of the work. An average of at least two initial readings shall be recorded as baseline reading.

Review Level – A maximum value of 10 mm relative to the baseline readings is suggested for this project. If this level is reached, the method, rate or sequence of construction, or ground stabilization measures should be reviewed or modified to mitigate further ground displacements.

Alert Level – A maximum value of 15mm relative to the baseline readings is suggested for this project. If this level is reached, the Contractor shall cease construction operations and to execute pre-planned measures to secure the site, to mitigate further movements and to assure safety of public and maintain traffic.

Review of Contractor's Proposed Method

MTO, the Proponent's prime consultant and Foundation Engineer should review the Contractor's proposed method of construction. The proposed method should include a description of the potential loss of ground, and calculation of the maximum settlement in relation to the Contractor's procedure and equipment, alternative/remedial measures when review level of measurement is reached; and contingency/remedial measures when alert level of measurement is reached.

Contractor's Responsibility For Restoration and Warranty Provision

In addition to the monitoring program to assess the adequacy of the construction method to control potential ground movements and groundwater, the Contractor is responsible for reinstatement (such as surface paving) should movements or other surface distress occur, and provide a reasonable warranty period acceptable to MTO. Remedial measures shall be approved by MTO; however, MTO maintains the right to perform the maintenance at the proponent's expense.

Construction Monitoring

The Proponent shall retain a qualified Geotechnical Consultant to supervise the installation of surface settlement points on site and to provide direction, technical input and field inspection on this project.