

FOUNDATION INVESTIGATION AND DESIGN REPORT

PROPOSED SANITARY TRUNK SEWER AND WATERMAIN CROSSING OF HIGHWAY 401, COURTICE, ONTARIO

Regional Municipality of Durham

GEOCRES NO. 30M15-276

Project No.: 131-12884-00

Date: January 5, 2016

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January 5, 2016

Paul Storms
Regional Municipality of Durham
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**Subject: Foundation Investigation and Design Report
Proposed Sanitary Trunk Sewer and Watermain Crossing of Highway
401, Courtice, Ontario
Project No. 131-12884-00**

Dear Mr. Storms,

We are pleased to submit this geotechnical investigation and design report for the proposed Courtice Trunk Sanitary Sewer and Watermain to be constructed beneath the MTO Highway 401 corridor near Clarington, Ontario. This report was prepared to provide recommendations for tunnel, shaft and open cut excavations for the trunk and watermain pipes and related manhole structures. The report is based on information from a borehole drilling and laboratory testing program completed in late July 2013.

Since the proposed tunnel is within MTO jurisdiction (high complexity RAQS requirement), WSP retained Hatch Mott MacDonald (HMM) to complete a formal review. The HMM report is appended, and all recommendations included in it have been addressed in this latest draft.

We trust that this report meets with your current requirements. Please contact us if you require anything further.

Yours truly,
WSP Canada Inc.

J. Stephen Ash, P. Eng., P. Geo.
Director, Environment



c. Hatch Mott MacDonald

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January 05, 2016

J. Stephen Ash, P.Eng., P.Geo.
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Dear Mr. Stephen Ash

**Subject: Peer Review of Foundation Investigation and Design Report for Tunnelling
Sanitary Trunk Sewer and Watermain Crossing of Highway 401, Courtice
Region of Durham
HMM Project: 358397
WSP Project: 131-12884-00**

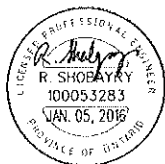
Dear Sir,

This is to confirm that we have reviewed the above noted Foundation Investigation and Design Report (FIDR) completed by WSP Canada Inc. for the Region of Durham. We are satisfied that our comments (contained therein and made relative to previous draft of that document) as well as Ministry of Transportation Ontario (MTO) comments (made relative to draft dated July 9, 2015) have been addressed and that the current report now meets the requirements of an MTO High Complexity Tunnelling under RAQS submission.

It is understood that the revised FIDR dated January 5, 2016 that contains our comments will be provided to MTO as part of encroachment Permit Application and that the report will form part of the contract package for bidders.

Please contact us if you have any further questions.

Yours faithfully,



G.J.E. Kramer, P.Eng.
Senior Vice President

Reza Shobayry, P.Eng.
Senior Project Engineer
GK:gd

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Corridor Management Office

1 INTRODUCTION

WSP Canada Inc. (WSP – formerly GENIVAR Inc.) was retained by the Regional Municipality of Durham (Region) to complete a geotechnical subsurface investigation for the proposed construction of the Courtice Trunk Sanitary Sewer and Watermain near Clarington, Ontario.

This report was prepared for the portion of the proposed alignment that will cross beneath the MTO Highway 401 corridor (project chainage 0+300 to 0+580). This portion has been designated Phase One Stage 2 (P1S2) of the overall Courtice Trunk Sewer project, and extends from the southern limit of Phase Two (P2) to the northern extents of Phase One Stage One (P1S1). The crossing itself is within the southern-most portion of P1S2. An overall plan for Phases 1 and 2 of this project has been provided as Figure 1.

The trunk sanitary sewer is to be installed via tunneling across the entire alignment presented in this report. The watermain will be advanced by tunneling immediately below Highway 401 (approx. 100 m in length), and by open cut installation elsewhere. Figure 2 provides plan and profile details for each of the proposed tunnels.

This report summarizes the field investigation procedures and findings, and provides our recommendations intended for designers and for MTO approval. Recommendations should not be construed as instructions to contractors who should form their own opinions about site conditions for tendering and determination of equipment and procedural requirements. This document is not intended to be a geotechnical baseline report for contract administration purposes.

2 DESCRIPTION OF THE SITE

The P1S2 component of the project (presented herein) exists within the Lake Iroquois Plain physiographic region¹, which has been classified as a clay plain. Surficial geology mapping has identified Pleistocene age glacial till deposits over much of the trunk sanitary sewer alignment, locally overlain by glaciolacustrine deposits of sand, silt and clay². The till deposits typically consist of undifferentiated, stone-poor, sandy silt to silty sand-textured till, typically rich in clasts, and are classified as Newmarket Till. These till deposits are often high in total matrix carbonate content and are also known to contain isolated sand and gravel deposits, which may be used as a source of water supply by local well users.

Shallow groundwater is expected to range between 2 m and 6 m below ground level (mBGL), based on history and published information.

No bedrock outcrops or significant surface water bodies exist in the investigated area. Bedrock beneath the tunnel site exists at a reported depth of approximately 100 mBGL (*Ontario Oil Gas and Salt Resources Library*, 2014), and is comprised of Middle Ordovician age limestone, dolostone, shale, belonging to the Whitby and Lindsay Formations.

¹ The Physiography of Southern Ontario, Chapman and Putnam, 1972; The Physiography of Southern Ontario, Third Edition, Chapman and Putnam, 1984.

² Quaternary geology, Province of Ontario, Data Set 14 Revised, Ontario Geological Survey, 2000.

Ground elevations along the P1S2 tunnel route generally range from approximately 101 m elevation to 104 m elevation, with an average elevation of approximately 102.5 m. Ground elevation increases gradually from south to north.

The P1S2 portion of the proposed Trunk Sanitary Sewer transects MTO land dedicated to Highway 401 and the proposed Highway 407 connection. Adjacent private lands include parking and laydown areas (both abandoned and in use), as well as agricultural lands to the southwest of this portion of the alignment.

3 METHOD OF INVESTIGATION

3.1 FIELD INVESTIGATION

The borehole investigation in support of the Highway 401 crossing was completed between Courtice Court and 70 meters south of South Service Road. The field investigation consisted of advancing three (3) exploratory boreholes, designated as Boreholes 13-6, 13-7 and 13-8 to depths of up to 21.8 mBGL. The boreholes were drilled close to entry/exit shafts and data were used to develop a stratigraphic cross-section for site interpretation relative to the proposed service invert alignment and depths (Figure 2). Investigation depth and borehole location criteria were considered. Boreholes were terminated in native soil and did not encounter bedrock. A borehole location plan alongside a subsurface section is provided as Figure 2.

Buried utility locates were obtained in advance of drilling, and where necessary prescribed borehole locations were moved slightly to avoid conflicts with known services. WSP worked with Mr. Marek Wiesek of MTO to secure the Controlled Access Encroachment Permit (EC-2013-20T-148) required for the work area.

Boreholes advanced for the Highway 401 crossing are summarized in the following table:

Table 3-1: Borehole Location Summary

Borehole	Project Chainage (approx.)	Date Drilled	Easting	Northing	Surface Elevation (m)
BH13-6	0+335	July 26, 2013	680198	4860862	101.697
BH13-7	0+440	July 29, 2013	680197	4860916	102.904
BH13-8	0+520	July 30, 2013	680201	4860991	102.951

Drilling and soil sampling was completed using a drilling subcontractor (Strong Soil Search Ltd.), operating a track-mounted commercial drill rig under the full-time supervision of an experienced WSP field engineer/technician. Boreholes were advanced to sampling depths by means of continuous flight hollow-stem augers. Sampling was initiated at intervals varying from 0.75 m to 1.5 m using a split-spoon core barrel driven in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). SPT results (N-values) were recorded for the sampled intervals as the number of blows required to drive a split-spoon sampler 300 mm into the soil, using a 63.5 kilogram (kg) drop hammer falling 750 mm. N-values are used in this report to assess consistency for cohesive soils and relative density for non-cohesive materials.

The samples were taken at intervals. The conventional interval sampling procedure used for this investigation does not recover continuous samples of soil at any borehole location. There is consequently some interpolation of borehole stratigraphy between samples, as provided on the borehole logs.

Sampled soil materials were logged in the field using visual and tactile methods, and were then sealed for transport to the WSP laboratory for laboratory testing, future reference, and storage. Soils for moisture content testing were placed in sealed laboratory jars for transport. All samples will be stored at the Peterborough WSP Soils Laboratories for one year.

Borehole elevations and locations are provided relative to geodetic datum (NAD 83), and were surveyed using the Trimble DGPS system referencing the CanNet network. Surveyed locations are considered accurate to within 1 cm.

Groundwater observations within the boreholes were made during drilling and prior to backfilling. Monitoring wells (50 mm diameter) were installed in Boreholes 13-7 and 13-8 upon completion of drilling. WSP returned to the site on September 20, 2013 (approximately 2 months after the completion of drilling) to monitor stabilized groundwater levels. The boreholes and wells were installed / abandoned in accordance with O. Reg. 903 requirements, as amended.

3.2 LABORATORY TESTING PROGRAM

Analytical laboratory investigations were completed to determine the properties of the soil samples recovered from the borehole program.

Table 3-2 below summarizes the laboratory testing program completed for soil samples obtained from the boreholes. Selected soil samples were submitted for laboratory tests to confirm the textural field descriptions in the borehole logs.

Table 3-2: Summary of Physical Tests

Laboratory Test	ASTM Standard	Number of Samples
Natural Moisture Content	ASTM D2216	75
Particle Size Analysis	ASTM D422, D4318	17
Atterberg Limit Test	ASTM D4318	1

Soil tests were completed at WSP's RAQ's certified soil laboratory. Test results are presented on the borehole logs included in Appendix A.

4 SUMMARIZED SUBSURFACE CONDITIONS

In general, underlying some topsoil and other shallow surficial deposits the boreholes encountered deposits of compact to very dense glacial till with a matrix ranging from silty sand to sandy silt (broadly, silt and sand till). The silt and sand till overlies deposits of gravel, sand, and silt (broadly, the sands and gravels) which are saturated and under hydrostatic pressure, as noted on the borehole logs. Soil cave was observed at all borehole locations during drilling and sampling between depths of 10 mBGL and 15 mBGL, within silt and sand till layers.

5 DETAILED STRATIGRAPHY

Borehole logs are provided as Appendix A and the geotechnical laboratory results are included in Appendix B.

The description of the subsurface soil profile has been simplified in terms of major soil strata, for the purposes of geotechnical evaluation. Stratigraphic boundaries and groundwater conditions indicated on the borehole logs are based on field observations, which are interpreted; stratigraphic contacts may vary between and beyond the borehole locations.

5.1 STRATIGRAPHY

5.1.1 TOPSOIL

Each of the three subject boreholes encountered a surficial layer of slightly organic silty topsoil, being loose to compact, and ranging from 0.3 m to 1.4 meters in thickness.

5.1.2 UPPER SILTY CLAY

Boreholes 13-6 and 13-8 encountered a layer of silty clay beneath the topsoil, extending to 2.9 metres below existing grade (mBG) where observed (101.4 m to 102.1 m elevation). The material contains a trace of sand and was brown, and generally drier than the plastic limit (DTPL). Particle size distribution measurements (referring to MTO/USCS grading curves) conducted on two (2) samples of the upper silty clay indicate that it contains 0% to 1% gravel, 2% to 3% sand, and 97% silt and clay (finer than 0.075 mm size).

Standard Penetration Test (SPT) results (N-values) measured within the silty clay ranged from 5 to 28 blows per 300 mm of penetration, indicating a firm to very stiff (but generally very stiff) consistency.

5.1.3 SILT AND SAND TILL

The boreholes encountered deposits of undisturbed native glacial till, with a cohesionless matrix of silty sand to sand and silt (generally, silt and sand). The till was encountered at depths ranging from 1.4 to 2.9 mBG (98.7 m to 101.1 m elevation). Within the boundaries of the Highway 401 crossing, the proposed tunnel will generally be advanced through and founded within this unit.

The silt and sand till contains a trace to some gravel (occasionally becoming gravelly, and with some gravel overall), and a trace to some clay. Cobbles and boulders are inferred to be present throughout the deposit, based on observations made during drilling and from rock fragments that were recovered in the split-spoon sampler. Cobble and boulder content cannot be estimated with certainty in drilling investigations of this nature, but may exceed 25% of the deposit, with quantities expected to vary laterally and vertically. Some boulders may exceed 300 mm (12 in) in diameter. The till was observed to be brownish grey to grey, and was moist to wet. Soil particle size analyses for ten (10) selected samples of the material are summarized in Table 5-1 (referring to MTO/USCS grading curves).

Table 5-1: Particle Size Summary, Silt and Sand Till Samples

Parameter	Percent Composition
Gravel (> 4.75 mm)	1 to 26
Sand (0.075 mm to 4.75 mm)	34 to 52
Silt (0.002 mm to 0.075 mm)	26 to 44
Estimated Clay Content (< 0.002 mm)	8 to 17

SPT N-values measured in the silt and sand till ranged from 10 to over 100 (blows per 305 mm of penetration), indicating that the material is compact to very dense (generally being very dense). In some cases it is expected that split-spoon samplers met high resistance due to cobbles and boulders in the material. Specifically referring to proposed tunnel elevations:

- within the proposed 2100 mm diameter trunk sewer tunnel zone, below approximately 95 m elevation, SPT N-values in the silt and sand till ranged from 20 to over 100 blows (on average, over 100 blows), indicating a compact to very dense (but generally very dense) material, and

- within the proposed 1200 mm diameter watermain tunnel zone, SPT N-values in the silt and sand till ranged from 20 to over 100 blows (on average, over 50 blows), indicating a compact to very dense (but generally very dense) relative density.

5.1.4 SANDS, SILTS, AND GRAVELS

Underlying the silt and sand till, borehole locations BH13-6 and BH13-7 encountered layers of interbedded sands, silts, and gravels. This material was encountered at 13.3 mBG and 13.2 mBG (88.2 and 88.9 m elevation) at borehole locations BH13-6 and BH13-7, respectively. The unit is 6.2 m thick at borehole location BH13-7.

The material was grey in colour, and was observed to be wet at the time of the investigation. SPT N-values measured in the sands and gravels ranged from 33 to over 50 blows, indicating very dense conditions.

Particle size distribution analysis of three (3) selected samples indicate the following (MTO/USCS): 2% to 26% gravel, 23% to 45% sand, 40% to 53% silt and clay (finer than 0.075 mm size). Fines were mainly silt material.

5.1.5 LOWER SILTY CLAY

Beneath the granular materials in BH13-7 and the till unit in BH13-8, a second layer of silty clay was observed. The lower silty clay was encountered at elevations 82.7 m and 85.6 m at boreholes BH13-7 and BH13-8, respectively, and extended beyond the depth of investigation where encountered. The lower silty clay was grey, with moisture generally at the plastic limit (APL) based on field observations and tests.

Particle size distribution measurements conducted on two (2) samples of the lower silty clay indicate that it contains 0 to 1% gravel, 1 to 6% sand, and 93% to 99% silt and clay. Atterberg Limit testing (LS-703/704) determined a plastic limit of 21% and a liquid limit of 42%, giving a plasticity index of 21%. Thus, the material is classified as a clay of intermediate plasticity (CI) according to the Unified Soils Classification System (USCS). Laboratory results are appended.

SPT N-values in the lower silty clay were mainly over 40 blows, indicating that the material is hard ($C_u > 200$ kPa inferred).

6 GROUNDWATER

Unstabilized groundwater seepage was observed in each of the open boreholes upon completion of drilling. Significant soil caving or upheaval was not observed. Groundwater levels were measured on September 20, 2013, nearly two months after the monitoring wells were installed, and are summarized in the following table.

Table 6-1: Summary of Groundwater Levels

Location	Measured Static Groundwater Depth/Elevation Sept 20, 2013 (mBG / m elevation)
BH13-6	No monitor (water at 4.9 mBG on completion of drilling)
BH13-7	4.1 / 98.8
BH13-8	2.9 / 100.1

Groundwater levels may fluctuate with time and seasonal conditions depending on the amount of precipitation and surface runoff. For practical purposes, groundwater at this site is approximately 3 m below present grade (approximately 99 m to 100 m elevation). It is noted that the proposed trunk sewer tunnel will be advanced below the groundwater table, within relatively low permeability glacial till material.

In situ hydraulic conductivity testing (rising head tests) was attempted in each of the two wells closest to Highway 401 where static groundwater levels were found (BH13-7 and BH13-8). Both wells did not recover sufficiently to provide useful data for the calculation of in situ hydraulic conductivity. The hydraulic conductivity (K) of the major soil strata was therefore estimated using the grain size distribution data available and Hazen's Method, and is summarized in the following table. The results indicate that the silt and sand till is less permeable than the underlying sands and silts while the coarser sands and gravels are several orders of magnitude more permeable.

Table 6-2: Summary of Estimated Hydraulic Conductivity for Soil Units

Soil Type / Unit	Hydraulic Conductivity, K (m/s)
Silt and Sand Till	10^{-7} to 10^{-9}
Sands, Silts Sands and Gravels*	10^{-7} to 10^{-8} 10^{-2} to 10^{-4}
Silty Clay Strata**	10^{-9} to 10^{-11}

* Based on experience and typical permeability's for these soils.

** Hazen's method provides rough estimates only for cohesive soils, as hydraulic conductivity depends to a high degree on the amount of fissuring.

7 DESCRIPTION OF THE PROJECT

As outlined in the Terms of Reference, the subsurface investigation in this area was conducted to provide geotechnical information in consideration of the following proposed structures crossing beneath MTO lands:

- a 2100 mm diameter trunk sanitary sewer, advanced via tunneling for approximately 240 m at proposed invert elevations ranging from ± 89 to ± 90 m, and
- a 400 mm diameter watermain, installed within a 1200 mm diameter tunnel for approximately ± 100 m, advanced at a proposed invert elevation of ± 93 m.

As shown in Figure 2, for the distance of the alignment beneath Highway 401, the vertical separation between the watermain and the trunk sewer pipes is less than 1 m. The sanitary sewer alignment is offset approximately 6 metres to the west of the watermain alignment.

Circular 6 metre diameter shafts are proposed at 250 metre intervals along the alignment of the trunk sanitary sewer tunnel, whereas circular 5 m diameter shafts are proposed for the watermain pipe.

Two circular 6 m diameter shafts are proposed at both ends of the 2100 mm trunk sanitary sewer tunnel (MH 3 and MH 4, in Figure 2). Two circular 5 m diameter shafts are proposed for the 400 mm diameter watermain crossing (1200 mm diameter tunnel). The locations and invert elevations of the shafts are summarized in the following table.

Table 7-1: Summary of Shaft Locations

Shaft	Approx. Chainage	Nearest Borehole	Approx. Invert Elevation (m)
(TSS) MH 3	0+312	BH13-6	89.4
(TSS) MH 4	0+555	BH13-8	89.8
(WM) 401-S	0+400	BH13-7	92.5
(WM) 401-N	0+500	BH13-8	92.6

8 TRUNK SEWER TUNNEL DESIGN

8.1 ANTICIPATED GEOTECHNICAL CONDITIONS

The anticipated subsurface conditions for the proposed trunk sewer tunnel are very dense silt and sand glacial till. Occasional compact zones are possible within the tunnel zone, as they were observed in close proximity to the tunnel zone in borehole location BH13-7. The lower sands and gravels were observed to be less than 2 m below the trunk sewer invert elevation in borehole locations BH13-6 and BH13-7, and also may be expected within the tunnel zone.

The glacial till may contain cobbles and boulders that were not encountered in the boreholes. Tunneling contractors could encounter cobbles or boulders and provision must be made in excavation and shoring contracts to allocate risks associated with time and costs to remove or penetrate obstructions to conventional drilling, when encountered.

Groundwater is approximately 7 m to 9 m above the trunk sewer tunnel springline. Seepage from the very dense till unit is not expected to be significant unless large pockets of saturated sand and silt are encountered.

The coarse material unit (sand, gravel) that is within a metre of the proposed tunnel invert in some locations (e.g. BH13-6) could yield considerable seepage if penetrated. The proximity of this layer to the proposed invert level indicates that this is likely to occur in some locations. Specific excavation requirements must be evaluated in terms of the expected mixed soil and groundwater seepage conditions. Monitoring wells screened within the underlying sands and gravels aquifer indicate that this unit is under sub-artesian groundwater pressure. Standard TBM tunneling will be prone to destabilization and raveling, unless a TBM utilizing earth pressure balance (EPB) technology is adopted. Care should also be taken at shaft/pit excavations to prevent basal heave (see Section 8.6.1).

8.2 ANTICIPATED GROUND BEHAVIOUR

The geotechnical investigation indicates that the tunnel will be advanced through very dense silt and sand till, with approximately 10 m of overburden soil cover (to obvert), that is generally very dense with occasional compact zones (as encountered in Boreholes 13-7 and 13-8). A tunnel of the proposed diameter excavated through the very dense silt and sand till will be stable, when made by using tunnel boring machine and where full and immediate support is provided behind the excavation face.

Referring to Terzaghi's Classification for Soils in Tunnelling, the very dense cohesionless till below the groundwater table falls within the "slow to rapidly raveling" soil type. The cohesionless till is well-graded and generally very dense, which will positively contribute to stability. However, high groundwater pressures and the large diameter of the tunnel will contribute to potential instability where support is not provided in a timely manner. Depending on the percentage of clay size particles present as a binder, and the compactness condition of the till, the estimated stand-up time of an unsupported vertical tunnel face could vary from 5 minutes to 30 minutes. Flowing ground conditions can be expected in areas where the sand and gravel intrudes into the zone of tunneling or where the till contains no or very little clay binder. Hence to assure safety full and immediate support will be required.

Beneath MTO lands, the proposed trunk sewer tunnel with an estimated 2.7 m excavated diameter will have an earth cover above the tunnel obvert equal to approximately 3.7 tunnel diameters (or ± 10 m). This depth of cover is sufficient to assure a safe operation and to minimize surficial settlements.

8.3 TUNNELLING METHOD

Tunnelling shall be undertaken in accordance with OPSS 415, OPSS 416, and OPSS 450 as appropriate as well as any applicable regional standards. The choice of equipment, the method of tunnelling and protection of public property is the Contractor's responsibility.

Consideration was given to tunnelling methods such as jack and bore, pipe ramming, horizontal directional drilling, and tunnel boring machine (TBM) with and without earth pressure balance technology to provide ground support and mitigate the risk of soil loss. A discussion on each method follows:

- Jack and bore installation involves jacking an oversized liner pipe from a launching shaft and removing the soil by augering inside the pipe as it is advanced.
- Tunnelling by pipe ramming involves driving a sleeve pipe from the access point to the exit point using an air-powered percussion hammer. After the sleeve is fully driven, the soil is removed by augering inside the pipe.
- Tunnelling by horizontal directional drilling (HDD) involves advancing a small diameter pilot hole along the alignment, and then connecting the daylighted drill stem at the exit point to a large diameter sleeve, which is then pulled back through the pilot hole while simultaneously enlarging the pilot hole by reaming a larger diameter hole.
- Tunnel boring machine techniques involve assembling a rotating cutter head at tunnel elevation, which is advanced by hydraulic jacking or "thrust cylinders". Excavated soils are recovered through the cutter head and transferred through the rear conveyor to haulage equipment. Earth Pressure Balance (EPB) technology refers to a closed shield TBM, which is capable of holding up soil under pressure and by adding suitable conditioning agents (e.g. bentonite, polymers etc.) to the ground ahead at the face. The TBM operator and automated systems keep the rate of soil removal equal to the rate of machine advance, maintaining a stable environment within the excavated tunnel.

A summary table of options is provided as follows:

Table 8-1: Tunneling Method Comparison Table, Trunk Sewer Tunnel

Tunnelling Method – Trunk Sewer	Advantages	Disadvantages	Costs Ranking*	Risks / Consequences
Jack and Bore	Least expensive alternative	Face not fully supported		May encounter pockets of highly pervious soils under water pressure resulting in face destabilization. The size of the tunnel may also be too large for this operation. This method should not be considered by the contractor owing to cobble and boulder content in the soil and potential for raveling. Further details are provided in Appendix D.
Tunnelling using TBM	Can utilize EPB for face stabilization	Greater expense and specialized. The minimum tunnel face pressure is 140 kPa, with a target face pressure of 190 kPa (HMM, 2015; Appendix D).	1	Encountering large boulders that the machine cannot easily remove.
Pipe Ramming	N/A	Likely cannot construct in the required size (> 2100 mm)		This method should not be considered by the contractor owing to cobble and boulder content in the soil and potential for raveling. Further details are provided in Appendix D.

Tunnelling Method – Trunk Sewer	Advantages	Disadvantages	Costs Ranking*	Risks / Consequences
Micro-Tunnelling	Can be applied to wide range of soils	Not typically used for 2100 mm. The minimum tunnel face pressure for microtunneling is 140 kPa, with a target face pressure of 190 kPa (HMM, 2015; Appendix D).	2	Encountering large boulders that the machine cannot remove.
Horizontal Directional Drilling	N/A	Cannot construct sizes 2100 mm or larger. Curvatures required during entry and exit.		This method should not be considered by the contractor owing to cobble and boulder content in the soil and potential for raveling. Further details are provided in Appendix D.

Notes: * Costs ranking proceeds from 1 (lowest cost) to 2 (highest cost), for valid options.

The diameter, length and anticipated subsurface conditions limit the range of trenchless installation techniques that would be economically viable for this project. Due to the larger diameter of the trunk sewer tunnel (2.7 m or larger) and the fact that the tunnel will extend north of the HWY 401 R.O.W., the method that is considered to be the most suitable for the construction of the tunnel is excavating with a tunnel boring machine (TBM) and utilizing EPB technology.

As noted previously, the contractor shall be advised that the subsurface till soils are very dense and contain cobbles and boulders that may exceed 300 mm in size and comprise over 25% of the material in some zones.

Excess hydrostatic head exists within the underlying sands and gravels, as observed in monitoring wells screened within this unit. The hydrostatic head is approximately 7 m to 10 m above the proposed trunk sewer invert. The use of EPB technologies will be required to maintain the stability of the trunk sewer tunnel under this hydrostatic load.

The final choice of tunnel boring equipment and the method of tunnelling is the responsibility of the Contractor, however, as summarized in Table 8-1, it is recommended that only EPB Tunnelling and Micro Tunneling be considered. The MTO, the Proponent's prime consultant, and the Foundation Engineer should review the Contractor's proposed method of construction with respect to interpreted geotechnical conditions. The proposed procedures should include a description of the potential loss of ground, and calculation of the maximum settlement in relation to the Contractor's method and equipment, alternative/remedial measures when review level of measurement is reached; and contingency/remedial measures when alert level of measurement is reached.

8.4 TEMPORARY AND FINAL SUPPORT

The geotechnical investigation indicates that the trunk sewer tunnel will be advanced under approximately 10 m of overburden soil cover, through silt and sand till, which is generally very dense with occasional compact zones (as encountered in Boreholes 13-7 and 13-8). A tunnel of the proposed diameter excavated through the very dense till will be stable, when made by tunnel boring machine excavation and where full support is immediately provided at and behind the excavation face.

Beneath MTO lands, the proposed trunk sewer tunnel will be excavated below approximately 3.7 tunnel diameters (or ± 10 m) of overburden soils. Given the proximity of the proposed tunnel to Highway 401 as well as the raveling nature of the soils, final support in the form of a concrete liner will be required. Design of the segmented concrete liner is the responsibility of the Contractor. Vertical and horizontal stresses for

the design of the segmented lining are to be provided by WSP during detailed design. The vertical in situ pressure (σ) at the springline of the tunnel is given by:

$$\sigma = \gamma(h - h_w) + \gamma' h_w + q + \gamma_w h_w$$

where,

- σ = the vertical pressure at depth, h (m)
- h = the depth below grade at which the pressure is calculated (m)
- h_w = the depth below the groundwater level (m)
- γ = the bulk unit weight of soil (See Section 8.6.2), (kN/m^3)
- γ' = the submerged unit weight of the exterior soil, (kN/m^3)
- γ_w = the unit weight of water (9.8 kN/m^3)
- q = the complete surcharge loading (kPa)

Based on the Terzaghi's Classification for Soils in Tunnelling analysis presented in Section 8.2, the estimated unsupported stand-up time will be sufficient to allow the precast concrete liner to be installed behind the tunnel header within the TBM.

The proposed trunk sewer tunnel will be constructed with a precast segmental concrete tunnel liner to provide final and permanent support. The trunk sewer liner annulus shall be grouted according to the design specifications. In the completed tunnel liner, the maximum induced stress would be expressed at the spring-line of the tunnel, given that the principal in situ stress is vertical.

8.5 ANTICIPATED GROUND DEFORMATION

The trunk sewer tunnel will be advanced under approximately 10 m of native overburden soils (Highway 401 to springline), generally comprising very dense silt and sand till and stiff to very stiff silty clay. The anticipated ground deformation may be calculated based on an equation for ground settlement above the centreline of tunnels as provided by Peck (1969), which is given by:

$$S_{max} = V_s / (2.5i)$$

where,

- S_{max} = the maximum surface settlement at the centre of the tunnel (m)
- V_s = volume of anticipated ground loss (assumed to be 1% of total volume excavated)
- i = the distance from the point of inflection to the maximum point on the settlement trough (related to tunnel geometry and ground condition through charts provided in Peck, 1969³).

The calculation assumes a tunnel inside diameter of 2.7 m (3.2 m mined diameter), and a depth to springline of 12.05 m. Based on the reviewed analysis, the maximum settlement at ground surface above the tunnel centreline (i.e. at the travelled portion of Highway 401) is estimated to be 4 mm to 7 mm. An additional 3 mm to 5 mm of settlement is expected at the launch shaft location south of Highway 401, nearer to South Service road, due to excavation. It is noted that these estimates could vary based on the contractor's means and methods and workmanship.

HMM has also conducted an independent settlement analysis using Rankine's method (1988), and results are provided in Appendix D of this report. The HMM method predicts 4 mm to 6 mm settlement due to the trunk sanitary sewer tunnel, 1 mm due to the watermain tunnel, and 4 mm to 7 mm from the combined tunnels. This is less than the allowable maximum settlement of 15 mm, and the review level settlement of 10 mm.

³ Peck R. B. (1969) *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering* (Mexico City), Deep excavations and tunnelling in soft ground, pp 225–290.

As recommended by HMM in Appendix D, tunnel shafts shall be setback at least 10.5 m from Highway 401 and the South Service Road to eliminate concerns of localized ground settlement around the shafts and in the adjacent roadways.

8.6 SHAFTS

Two circular 6 m diameter shafts are proposed at the two ends of the 2100 mm trunk sanitary sewer within the area of the Highway 401 crossing (MH 3 and MH 4, as shown in Figure 2). For each of the proposed shaft excavations, overburden soils above the groundwater table may be cut back to a stable inclination if space restrictions permit. Protection systems and associated performance level for the entry and exit shafts shall be designed in accordance with OPSS 539. OSHA safe slopes for open cut excavations are provided in Section 10.2.

Where excavations cannot be sloped, or where open cuts are not economical, they may be supported through the overburden using a shored excavation designed by a professional engineer. Water tight shoring may also be considered as a seepage control alternative, but is not required. No excavation shall extend below the foundations of any existing structures without adequate alternative support being provided.

8.6.1 BASAL STABILITY AND DEPRESSURIZATION

Basal instability can occur when a high hydraulic gradient creates seepage pressure through the base of an excavation, or where a more pervious deposit containing water under high hydrostatic head underlies the base of the shaft at a shallow depth (as is the case at the south shaft).

The trunk sewer shaft bases are to be located approximately 10 m below the potentiometric surface. Both of the shafts have a moderate risk of basal instability due to the relative proximity of the underlying pressurized and more permeable granular layers, which have at least 10 m of expected hydrostatic head. For safety, the underlying water-bearing strata (granular material) must be depressurized using an eductor well system around the shafts, and the potentiometric head must be brought down to at least 1 m below the base of excavation. Positive dewatering is therefore required to maintain a stable base of excavation for the shafts.

Each of the shafts shall be founded in very dense sand and silt till. Following depressurization, a dry excavation must be properly maintained by pumping from a conventional sump pump arrangement. Exposed soil should be protected by a skim coat of lean concrete.

The choice of equipment and the method of dewatering is the Contractor's responsibility as, but not limited to, OPSS 517 and 518. Consideration should be given to using sump pumps at the entrance and exit shafts to capture water brought from the tunnel. Dewatering discharge should be filtered through a geotextile bag or geotube, and disposed of by sheetflow into nearby grassed areas south of South Service Road, or the grassed area located between Courtice Court and Highway 401.

A professional dewatering contractor should be consulted to review the subsurface conditions and to design a site specific dewatering system. It is the dewatering contractor's responsibility to make an assessment of the factual data and to provide recommendations on dewatering system requirements, including water treatment requirements and control of discharge. Water quality sampling will be required to assess discharge options for dewatering systems.

The effective stress increase from dewatering at the mid-depth of the shaft excavation will be less than 25%, and resulting ground settlement of the very dense till, if any, is expected to be less than 1 mm or 2 mm, and only in the immediate vicinity of the shaft and wellpoint/eductor systems. Settlement of the roadway pavements as a result of combined shaft dewatering and tunnelling is expected to be less than the review level settlement of 10 mm, with the shafts to be located more than 10.5 m from the edges of the pavements as noted.

The quantity of seepage expected must be reviewed at a later time, when construction methods and details are known. Combined depressurization and dewatering volumes for trunk sewer shafts may exceed 50,000 L/day, and owing to the expected duration of dewatering a Category 3 Permit to Take Water (PTTW) from the Ministry of Environment will be required. Additional hydrogeological investigations including installation of monitoring wells in granular layers and hydraulic conductivity analysis will be required to support a permit application. This application should be submitted for review and approval at least 4 months before construction starts. WSP should be contacted to facilitate submission of this application.

8.6.2 EARTH PRESSURE DISTRIBUTION

The anticipated excavations will be advanced through surficial soils and silt and sand till at each of the shaft locations.

The appropriate geotechnical parameters for use in the design of shoring system at this site are tabulated as follows:

Table 8-2: Earth Pressure Parameters (unfactored)

Stratum	Φ (Friction angle, deg.)	γ (bulk unit weight, kN/m ³)	K_a (active earth pressure)	K_o (at-rest earth pressure)	K_p (passive earth pressure)
Upper Silty Clay	28	19	0.36	0.53	2.77
Silt and Sand Till	36	23	0.26	0.41	3.85
Sands and Gravels	36	22	0.26	0.41	3.85

The above earth pressure parameters pertain to a horizontal grade condition behind the retaining structure or shoring wall. Values of earth pressure parameters for an inclined retaining grade condition will vary.

Walls subject to lateral earth pressures (P) must be designed to resist a pressure that can be calculated based on the following equation:

$$P = K[\gamma(h - h_w) + \gamma' h_w + q] + \gamma_w h_w$$

where,

- P = the horizontal pressure at depth, h (m)
- K = the earth pressure coefficient
- h_w = the depth below the groundwater level (m)
- γ = the bulk unit weight of soil, (kN/m³)
- γ' = the submerged unit weight of the exterior soil, (kN/m³)
- γ_w = the unit weight of water (9.8 kN/m³)
- q = the complete surcharge loading (kPa)

Where the soil behind the wall is drained effectively to eliminate hydrostatic pressures that would otherwise act in conjunction with the earth pressure, a trapezoidal earth pressure distribution is applicable and the above equation is reduced to:

$$P = K[\gamma h + q]$$

8.6.3 SOLDIER PILE TOE DESIGN

Soldier pile toes will be made in dense to very dense silt and sand till, and potentially in the underlying sands and gravels (such as encountered by BH 13-6). The horizontal resistance of the soldier pile toes will be developed by embedment below the base of excavation, where resistance is developed from passive earth pressure.

Such zones of material in the subsurface soils are expected that are sufficiently wet, cohesionless, and permeable that augered holes for soldier piles may become unstable. Furthermore, augered boreholes are expected to become destabilized by the piezometric heads in the underlying granular materials that are also prone to caving. In these cases, it will be necessary to advance temporarily cased holes to prevent excess caving during the soldier pile installations.

8.6.4 SHORING SUPPORT

Shoring configurations for shaft applications typically involve the entire perimeter of a circular or square shaft being shored. Internal ring beams, walers, or corner bracing are typically used to support shored shaft walls, especially where space restrictions will not allow a tie-back rig to enter the shaft. This type of shoring support is usually preferred to the use of tie-backs or earth anchors in shafts. Internal bracing such as rakers will not be feasible for small shaft applications.

If anchor support is necessary and determined to be feasible, the shoring system should be supported by pre-stressed soil anchors extending beneath the adjacent lands. Pre-stressed anchors are installed and stressed in advance of excavation and this limits movement of the shoring system as much as is practically possible. The use of anchors on adjacent properties requires the consent of the adjacent land owners, expressed in encroachment agreements.

Multi-level supported shoring utilizing pre-tensioned anchors or internal bracing, and drained soil conditions behind the wall, can be designed in cohesionless soils for a trapezoidal earth pressure distribution with a maximum pressure defined by:

$$P = 0.65K[\gamma h + q]$$

Shafts that are constructed using internal ring beam arrangement, as discussed above, are over-designed against active earth pressures. Ground deformations adjacent to this type of shaft, at surface and along the shoring wall, are considered to be negligible. If other methods of shoring support are used, these must be reviewed for deformation prior to being constructed adjacent to the travelled lanes of Highway 401.

8.6.5 SEEPAGE CONTROL

With the possible exception of localized coarser grained lenses, it is expected that the dense glacial till will produce a nominal amount of seepage through the sidewalls of excavated shafts for the trunk sewer tunnel. As such, permeable shoring may be considered for the proposed trunk sewer tunnel shafts. Seepage would be allowed to drain into the shaft excavation through a conventional soldier pile and lagging wall, and then removed from the base using a sump pump arrangement (capable of handling the expected pumping rates) to maintain a safe working surface. The use of water tight shoring (in the form of an interlocking caisson wall) may also be considered, to control seepage through the shaft sidewalls. Water volumes collected in shaft excavations due to seepage from sidewalls is unlikely to exceed 50,000 L/day. For safety, it is recommended that a PTTW be obtained to depressurize and dewater materials beneath and adjacent to the shaft locations. A liberal quantity of water should be considered in the PTTW application to cover sidewall seepage handling in each shaft location. WSP should be consulted with regards to dewatering and permitting requirements related the project.

9 WATERMAIN TUNNEL DESIGN

9.1 ANTICIPATED GEOTECHNICAL CONDITIONS

The anticipated subsurface conditions for the proposed 1200 mm diameter watermain tunnel liner installed at an invert elevation of ± 93 m are very dense silt and sand glacial till. Occasional compact zones are possible within the tunnel zone, as they were observed in close proximity to the tunnel zones in borehole location BH13-7. Very dense silty sand with an elevated percentage of gravel was encountered in borehole location BH13-8 above 97.8 m elevation, which is less than 4 m above the obvert.

The glacial till is dense to very dense as described, and should be expected to contain variable cobble and boulder size material. Tunneling contractors must make provisions in their methods and procedures, and in contracts, to allocate risks associated with time and costs to remove or penetrate obstructions when encountered.

The invert of the proposed 1200 mm diameter tunnel liner for the watermain will be located approximately 6 m to 7 m below the prevailing groundwater level. Seepage from the till is not expected to be significant if the liner is advanced directly behind the face of excavation to provide immediate support to the tunnel.

9.2 ANTICIPATED GROUND BEHAVIOUR

The geotechnical investigation indicates that the tunnel will be advanced through very dense sand and silt till, very stiff silty clay, and potentially dense sands and gravels, with a minimum of 9 m of overburden soil cover below Highway 401. The native soils within the tunneling zone are generally very dense, with occasional compact zones (as encountered in Boreholes 13-7). A tunnel of the proposed diameter (1200 mm) excavated through the very dense silt and sand till will be stable, when primary support (the proposed concrete liner) is immediately provided behind the excavation face.

Referring to Terzaghi's Classification for Soils in Tunnelling, the very dense till above and below the groundwater table is classified "slow ravelling" and "rapidly ravelling" soil type, respectively. The cohesionless till is well-graded and generally very dense, which will positively contribute to stability. On the other hand the tunnel will be advanced 6 m to 7.5 m below the groundwater table, albeit in relatively low permeability soils and this hydrostatic pressure will have a small destabilizing effect.

Given the proximity of the proposed tunnel to Highway 401, as well as the ravelling nature of the soils, primary support in the form of a liner will be required immediately behind the tunnel face.

9.3 TUNNELLING METHOD

Tunnelling shall be undertaken in accordance with OPSS 415, 416 and OPSS 450 as appropriate, as well as any applicable regional standards. The choice of equipment and the method of tunnelling is the Contractor's responsibility.

Consideration was given to tunnelling methods such as jack and bore, pipe ramming, horizontal drilling, and pipe jacking using tunnel boring machine with and without earth pressure balance technology. A summary table of options is provided as follows:

Table 9-1: Tunneling Method Comparison Table, Watermain Tunnel

Tunnelling Method – Watermain	Advantages	Disadvantages	Costs Ranking*	Risks / Consequences
Jack and Bore	Low cost.	May not cope with ravelling ground conditions should those be encountered. May encounter refusal on large boulder.		Loss of ground and potential increased settlement. This method should not be considered by the contractor owing to cobble and boulder content in the soil and potential for raveling. Further details are provided in Appendix D.

Tunnelling Method – Watermain	Advantages	Disadvantages	Costs Ranking*	Risks / Consequences
Tunnelling using TBM	Can use EPB for face stabilization.	Expensive. Small diameter (1200 mm). May be prohibitive. The minimum tunnel face pressure is 140 kPa, with a target face pressure of 190 kPa (HMM, 2015 Appendix D).	2	Encountering large boulders that the machine cannot remove.
Pipe Ramming	Safe method of construction. No loss of ground.	Excessive vibration may cause damage to existing utilities. May cause some ground heaving due to the shallow depth.		This method should not be considered by the contractor owing to cobble and boulder content in the soil and potential for raveling. Further details are provided in Appendix D.
Micro-Tunnelling	Can apply to wide range of soils, provides a safe way of construction with minimal loss of ground.	May be delayed if large boulders encountered. The minimum tunnel face pressure is 100 kPa, with a target face pressure of 150 kPa (HMM, 2015; Appendix D).	1	Encountering large boulders that the machine cannot remove.
Horizontal Directional Drilling	N/A	Curvatures required during entry and exit.		This method should not be considered by the contractor owing to cobble and boulder content in the soil and potential for raveling. Further details are provided in Appendix D.

Notes: * Costs ranking proceeds from 1 (lowest cost) to 2 (highest cost), for valid options.

The final choice of tunnel boring equipment and the method of tunnelling is the responsibility of the Contractor, however, as summarized in Table 9-1, it is recommended that only EPB Tunnelling and MicroTunneling be considered. The MTO, the Proponent's prime consultant, and the Foundation Engineer should review the Contractor's proposed method of construction with respect to interpreted geotechnical conditions. The proposed procedures should include a description of the potential loss of ground, and calculation of the maximum settlement in relation to the Contractor's method and equipment, alternative/remedial measures when review level of measurement is reached, and contingency/remedial measures when alert level of measurement is reached.

9.4 TEMPORARY AND FINAL SUPPORT

As it is currently proposed, the watermain tunnel will be advanced under approximately 9 m of overburden soil cover, through tills comprised of varying silt and sand concentrations, which are generally very dense with occasional compact zones (as encountered in Boreholes 13-7). A tunnel of the proposed diameter excavated through the native soils will be stable, when excavated by a method that allows the placement of full support (the liner) immediately behind the excavation face.

The proposed watermain tunnel will be installed within a 1200 mm reinforced concrete (with gasket) pressure pipe, or a 19 mm thick steel casing, per MTO acceptance policies. External annulus grouting is recommended for this pipe, and it should be designed for an all-round pressure equal to the vertical in situ pressure at the springline of the tunnel, which is given by:

$$\sigma = \gamma(h - h_w) + \gamma' h_w + q + \gamma_w h_w$$

where,

- σ = the vertical pressure at depth, h (m)
- h = the depth below grade at which the pressure is calculated (m)
- h_w = the depth below the groundwater level (m)
- γ = the bulk unit weight of soil (See Section 9.6.2), (kN/m³)
- γ' = the submerged unit weight of the exterior soil, (kN/m³)
- γ_w = the unit weight of water (9.8 kN/m³)
- q = the complete surcharge loading (kPa)

A carrier pipe is required if the annular space is to be grouted.

9.5 ANTICIPATED GROUND DEFORMATION

The springline of the 1200 mm diameter watermain tunnel will be located at a minimum depth of 9.5 m below the paved portion of Highway 401, and will likely be advanced within very dense silt and sand till. The anticipated ground deformation may be calculated based on an equation for ground settlement above tunnels proposed by Peck (1969), which is given by:

$$S_{max} = V_s / (2.5i)$$

where,

- S_{max} = the maximum surface settlement above the centreline of the tunnel (m)
- V_s = volume of anticipated ground loss (assumed to be 1.0 % of total volume excavated)
- i = the distance from the point of inflection to the maximum point on the settlement trough (related to tunnel geometry and ground condition through charts provided by Peck, 1969).

The calculation assumes a tunnel diameter of 1200 mm, and a depth to springline of 9.5 m. Based on reviewed analysis, the maximum settlement at ground surface above the tunnel centreline along Highway 401 was estimated to be 5 mm to 7 mm. An additional 3 mm to 5 mm of settlement is expected at the launch shaft location south of highway 401, nearer to South Service road due to excavation. It is noted that this estimate could vary based on the means and methods of construction selected by the contractor as well as the workmanship.

HMM has also conducted a settlement analysis using Rankine's method (1988), and results are provided in Appendix D of this report. The HMM method predicts 4 mm to 6 mm settlement due to the trunk sanitary sewer tunnel, 1 mm due to the watermain tunnel, and 4 mm to 7 mm from the combined tunnels. This is less than the allowable maximum settlement of 15 mm, and the review level settlement of 10 mm.

As recommended by HMM in Appendix D, shafts shall be setback at least 10.5 m from Highway 401 and the South Service Road to eliminate concerns of localized ground settlement around the shafts on the adjacent roadways.

9.6 SHAFTS

Two circular shafts with diameters of 5 metres are proposed to construct the 1200 mm diameter watermain crossing. The locations of the shafts are given in Table 7-1 and are also shown in Figure 2. For the proposed shaft excavations, overburden soils above the ground water table may be cut back to a stable inclination if space restrictions permit. OHSA soil types for open cut excavations are provided in Section 10.2.

Where excavations cannot be sloped, or where sloped cuts are not economical, the excavations may be supported through the overburden using a shored excavation designed by a professional engineer. Impermeable shoring may also be considered as a seepage control alternative, but likely is not required. No excavation shall extend below the foundations of adjacent existing structures or utilities without adequate alternative support being provided.

Protection systems and associated performance level for the entry and exit shafts shall be designed in accordance with OPSS 539. OSHA safe slopes for open cut excavations are provided in Section 10.2 of this report.

9.6.1 BASAL STABILITY AND DEPRESSURIZATION

Basal instability can occur when a high hydraulic gradient is created as a result of water flowing into the excavation through the base of the excavation.

In consideration of existing pore pressure heads, basal instability of the shaft excavations must be considered and depressurization of the underlying materials may be required. The till subsoils are dense to very dense such that the overall risk of basal uplift is low; however, the till may contain pressurized coarse layers near the foundation grades that may cause localized instability and uplift of the base, and related construction problems. Such layers may be difficult to dewater and a specialty contractor should be consulted for practical approaches. CFEM (2006) recommends that hydraulic pressure head acting on the base of excavations not exceed 70 % of the resisting soil pressure due to self-weight. It is recommended; therefore, that eductor wells be established in the shaft areas to reduce pressure head to the base level of the excavation or to a minimum level to preclude uplift.

The base of the dewatered excavation should be protected by a skim coat of lean concrete.

The choice of equipment and the method of dewatering is the Contractor's responsibility as, but not limited to, OPSS 517 and 518. Consideration should be given to using sump pumps at the entrance and exit shafts to capture water brought from the tunnel. Dewatering discharge should be filtered through a geotextile dewatering bag or geotube, and disposed by sheetflow into nearby grassed areas south of South Service Road, or the grassed area located between Courtice Court and Highway 401.

9.6.2 EARTH PRESSURE DISTRIBUTION

The shaft excavations will be advanced through surficial clayey and sandy soils and silt and sand till. The soil parameters for use in the design of shoring are tabulated below:

Table 9-2: Earth Pressure Parameters (unfactored)

Stratum	ϕ (Friction angle, deg.)	γ (bulk unit weight, kN/m ³)	K_a (active earth pressure)	K_o (at-rest earth pressure)	K_p (passive earth pressure)
Upper Silty Clay	28	19	0.36	0.53	2.77
Silt and Sand Till	36	21	0.26	0.41	3.85
Very Dense Sands and Gravels (encountered at BH13-6)	36	20	0.26	0.41	3.85

The above earth pressure parameters pertain to a horizontal grade condition behind the retaining structure or shoring wall. Values of earth pressure parameters for an inclined grade condition will vary. Walls subject to lateral earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

$$P = K[\gamma(h - h_w) + \gamma' h_w + q] + \gamma_w h_w$$

where,

- P = the horizontal pressure at depth, h (m) [kPa]
- K = the earth pressure coefficient
- h_w = the depth below the groundwater level (m)
- γ = the bulk unit weight of soil, (kN/m³)
- γ' = the submerged unit weight of the exterior soil, (kN/m³)
- γ_w = the unit weight of water (9.8 kN/m³)
- q = the complete surcharge loading (kPa)

Where the soil behind the wall is drained effectively to eliminate hydrostatic pressures that would otherwise act in conjunction with the earth pressure, a trapezoidal earth pressure distribution is applicable, and the above equation is reduced to:

$$P = K[\gamma h + q]$$

9.6.3 SOLDIER PILE TOE DESIGN

Soldier pile toes will be established in dense to very dense silt and sand till. The lateral resistance of the soldier pile toes will be developed by embedment below the base of excavation, where resistance is developed from passive earth pressure.

Such zones of material in the subsurface soils are expected that are sufficiently wet, cohesionless, and permeable that augered holes for soldier piles may become unstable. Furthermore, augered boreholes are expected to become destabilized by the piezometric heads in the underlying granular materials, that are also prone to caving. In these cases, it will be necessary to advance temporarily cased holes to prevent excess caving during the soldier pile installations.

9.6.4 SHORING SUPPORT

Shoring configurations for shaft applications typically involve the entire perimeter of a circular or square shaft being shored. Internal ring beams, top whalers, or corner bracing are typically used to support shored shaft walls, especially where space restrictions will not allow a tie-back rig to enter the shaft. This type of shoring support is usually preferred to the use of tie-backs or earth anchors in shafts. Internal bracing such as rakers will not be feasible for shaft applications.

If anchor support is necessary and determined to be feasible, the shoring system should be supported by pre-stressed soil anchors extending beneath the adjacent lands. Pre-stressed anchors are installed and stressed in advance of excavation and this limits movement of the shoring system as much as is practically possible. The use of anchors on adjacent properties requires the consent of the adjacent land owners, expressed in encroachment agreements.

Multi-level supported shoring utilizing internal bracing struts, under the same drained conditions as above (Section 9.6.2), can be designed in cohesionless soils based on a rectangular earth pressure distribution with a maximum pressure defined by:

$$P = 0.65K[\gamma h + q]$$

where,

- P = the horizontal pressure at depth,
- h = maximum height of excavation (m)
- γ = the bulk unit weight of soil, (kN/m³)
- q = the complete surcharge loading (kPa)

For anchored support systems, as trapezoidal earth pressure diagrams may be considered in accordance with Section 26.10 of CFEM, 4th edition (2006).

Ground deformations adjacent to shafts designed to resist the above earth pressure are considered to be negligible. If other methods of shoring support are used, these must be reviewed for deformation prior to being constructed adjacent to the travelled lanes of Highway 401.

9.6.5 SEEPAGE CONTROL

The watermain shafts will extend 6 m to 7.5 m below the groundwater table, through relatively low permeability till. Seepage through the walls and the base of the shaft should be allowed to drain into the shaft excavation, and then be removed from the base by pumping from filtered sumps. Since a PTTW is required for this project, groundwater control from these shafts (although nominal) may be conservatively included without changing the application requirements substantially.

It should be noted that the upper soil layers may present a risk of perched groundwater conditions at certain times of the year. Appropriate care must be taken if work is to proceed during wet periods. The use of trial excavation may be advantageous in observing any perched water levels which may be present in the upper sands and gravels during construction.

No additional significant ground deformation is expected as a result of watermain shaft dewatering. Shafts shall be located at least 10.5 m away from the edges of pavement, as noted previously.

10 WATERMAIN OPEN CUT DESIGN

The 400 mm diameter CPP watermain will be installed via open cut trench construction through the north and south portions of this Phase that do not cross under Highway 401 or South Service Road. This includes portions of the alignment from Station 0+300 to 0+400 (south side), and Station 0+500 to 0+580 (north side). Details of the proposed watermain are shown on Figure 2, along with the summarized borehole information.

The watermain is to be installed at invert elevation ± 93 m within the south side excavation, and at ± 93.1 m to ± 99.5 m elevation (rising from south to north) within the north side excavation.

10.1 ANTICIPATED GEOTECHNICAL CONDITIONS

The watermain will be installed within very dense silty sand till, which was observed in each of the boreholes at the founding elevations. Borehole location BH13-8 also penetrated an upper layer of silty sand, which may be a coarser pocket of the same glacial till deposit.

The glacial till may cobbles and boulders that were not encountered in the boreholes. Tunneling contractors could encounter cobbles or boulders and provision must be made in excavation and shoring contracts to allocate risks associated with time and costs to remove or penetrate obstructions to conventional drilling, when encountered.

10.2 EXCAVATIONS

Excavations must be carried out in accordance with the *Occupational Health and Safety Act and Regulations for Construction Projects, November 1993 (Part III - Excavations, Section 222 through 242)*. These regulations designate four (4) classifications of soils, based on the internal strength and groundwater characteristics of each soil unit. OHS soil classifications are used to specify appropriate excavation methods to ensure safety.

For practical purposes, the upper silty clay and the silty and sand till are Type 2 soils above the groundwater table (i.e. 100 m elevation), and Type 3 soils below the groundwater table (i.e. 100 m elevation). Granular materials encountered below ± 89 m elevation at borehole locations BH13-6 and BH13-7 are Type 4 soils, as defined in the Regulations.

OHSA regulations indicate that maximum slopes for excavations up to 6 meters in depth for Types 2 and Type 3 soils are 1:1. The use of support system requirements for steeper excavations is stipulated in Sections 235 through 238 and 241 of the Act and Regulations and includes provisions for timbering, shoring and moveable trench boxes. The use of a support trench with a temporary or moveable shoring should be considered at excavation depths below 6 meters. This must be designed by a qualified geotechnical engineer referring to Sections 8.62 and 9.62.

The glacial till soils contain larger particles such as cobbles or boulders that were not directly observed in the boreholes, but which were inferred to be present based on resistance to augering, grinding, and as well as rock fragments found in SPT samples. Shoring, excavation, or tunneling contractors should expect to encounter buried obstructions, and provisions must be made in excavation and shoring contracts to allocate risks associated with time and costs to remove or penetrate larger particles or obstructions to conventional drilling, when encountered.

10.3 GROUNDWATER CONTROL

The watermain trench will extend approximately 6 m to 7 m below the groundwater table. Experience suggests that the very dense silty sand till through which it will be advanced is sufficiently "tight" so as to generally preclude the free flow of groundwater.

It should be noted that the upper granular material (such as observed in BH 13-8) presents the risk of perched groundwater conditions during certain times of the year. Appropriate care must be taken if work is to proceed during wet periods. The use of trial excavation may be advantageous in observing any perched water levels which may be present in the granular materials during construction.

It may be advantageous to consider advancing a series of trial excavations at the site, to assess the need for open cut dewatering in certain locations (i.e. in the vicinity of BH 13-8 adjacent to the upper sand and silt zone, or within the deepest portions of the open cut section).

Manholes will be founded below the groundwater table and must be designed for uplift/floatation pressure originating from an assumed high water level located at the finished ground surface elevation. Although a temporary and short-term occurrence, this water level can be achieved during wet seasons, such as spring and fall.

10.4 BEDDING

In general, native glacial tills at the site will provide adequate support for piping provided with conventional Class 'B' bedding. Bedding materials can be well graded, granular fill, such as Granular "A" (OPSS 1010) or 19 mm Crusher Run Limestone. Clear stone bedding, separated from the subgrade tills by a layer of nonwoven geotextile approved by the Geotechnical Engineer, may be used as backfill if occasional wet pockets are encountered. All granular bedding must be compacted to a minimum of 98 % of Standard Proctor Maximum Dry Density (SPMDD) as per ASTM D698 procedures, or compacted by vibration to a dense state in the case of clear stone bedding.

10.5 BACKFILL

Excavated glacial till soils and clayey soils may be used as backfill, provided that the moisture content is within 2% of the SPMDD optimum moisture content for compaction and adequate workability. If narrow trenches are excavated, the use of aggregate fill is required if there is to be post-construction grade integrity. The trench backfill must be compacted to at least 95% of SPMDD in non-settlement sensitive areas, or 98% of SPMDD otherwise.

Some of the excavated soils will be taken from below the groundwater table, and are expected to be wet. These soils will need to be stockpiled and dried before they may be reused as trench backfill, or wasted. It is recommended that dried soils be used in non-settlement sensitive areas only.

11 HIGHWAY 401 SETTLEMENT MONITORING

The proposed trunk sewer and watermain tunnel alignments cross beneath MTO lands (Highway 401) from approximately Station 0+320 to Station 0+560. The trunk sewer tunnel obvert is at Elevation ± 92 m, which is approximately 11 m below grade (equivalent to about 3.7 sanitary sewer tunnel diameters). The watermain tunnel obvert will be at Elevation ± 94 m or roughly 9 tunnel diameters below grade.

A qualified Geotechnical Consultant shall supervise the installation of settlement points to monitor potential settlements induced by the proposed tunneling.

Settlement monitoring of the roadway will be conducted in accordance with MTO specifications and requirements. Dataloggers capable of transmitting data remotely should be used for safety (e.g. SISGEO Mini OMNIAlog Datalogger), and installations should be done in accordance with OTC Book 7.

Recommendations for this monitoring work are provided as a Non-Standard Special Provision (see Appendix C), which is to be submitted for review by the MTO prior to commencement of the works. The Settlement Monitoring Plan provided on Figure 3 indicates the approximate locations of monitoring instruments and provides typical instrument details. Monitoring points shall be provided on the Highway paved surfaces as shown.

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

The MTO, the Proponent's prime consultant, and the Foundation Engineer should review the Contractor's proposed method of construction. The proposed method should include a description of the anticipated 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 (as laid out in the attached Non-Standard Special Provision, as well as on Figure 3).

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 (e.g. surface paving) should movements or other surface distress occur, and provide a reasonable warranty period acceptable to the MTO. Remedial measures shall be approved by the MTO.

12 LIMITATIONS

12.1 PROCEDURES

Borehole samples were taken at intervals. The conventional interval sampling procedure used for this investigation does not recover continuous samples of soil at any borehole location. There is consequently some interpolation of the borehole layering between samples, and indications of changes in stratigraphy as shown on the borehole logs are approximate. Such interpretations are extended below Highway 401 and risks of variability between the boreholes must be assumed.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. WSP has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information and geotechnical advice to completely identify all aspects of the site and works that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project must be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, and their approach to the construction works, cognizant of the risks implicit in the subsurface investigation activities.

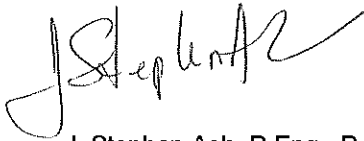
12.2 CHANGES IN SITE AND SCOPE

It must be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. In particular, caution should be exercised in the consideration of contractual responsibilities as they relate to control of seepage, disturbance of soils, and frost protection.

The design parameters provided and the engineering advice offered in this report are based on the factual data obtained from this investigation made at the site by WSP and are intended for use by the owner and its retained design consultants in the design phase of the project. If there are changes to the project scope and development features, the interpretations made of the subsurface information, the geotechnical design parameters, advice and comments relating to constructability issues and quality control may not be relevant or complete for the project. WSP should be retained to review the implications of such changes with respect to the contents of this report.

We trust this report meets with your requirements. Should you have any questions regarding the information presented, please do not hesitate to contact our office.

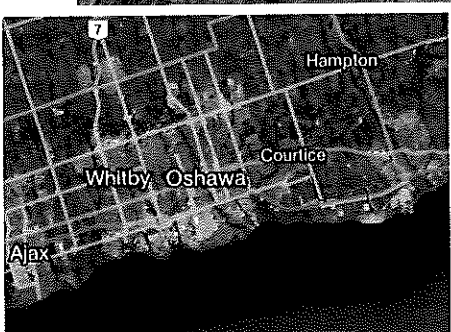
Sincerely,
WSP Canada Inc.



J. Stephen Ash, P.Eng., P.Geo.
Director, Environment



Figures



KEY PLAN
N.T.S.

0 400 800
Meters



294 RINK STREET, SUITE 103
PETERBOROUGH, ONTARIO, CANADA K9J 2K2
TEL: 705-743-8850 FAX: 705-743-8859
WWW.WSPGROUP.COM

PROJECT:

COURTICE TRUNK SANITARY SEWER
PHASE 1, STAGE 2

TITLE:

SITE LOCATION PLAN

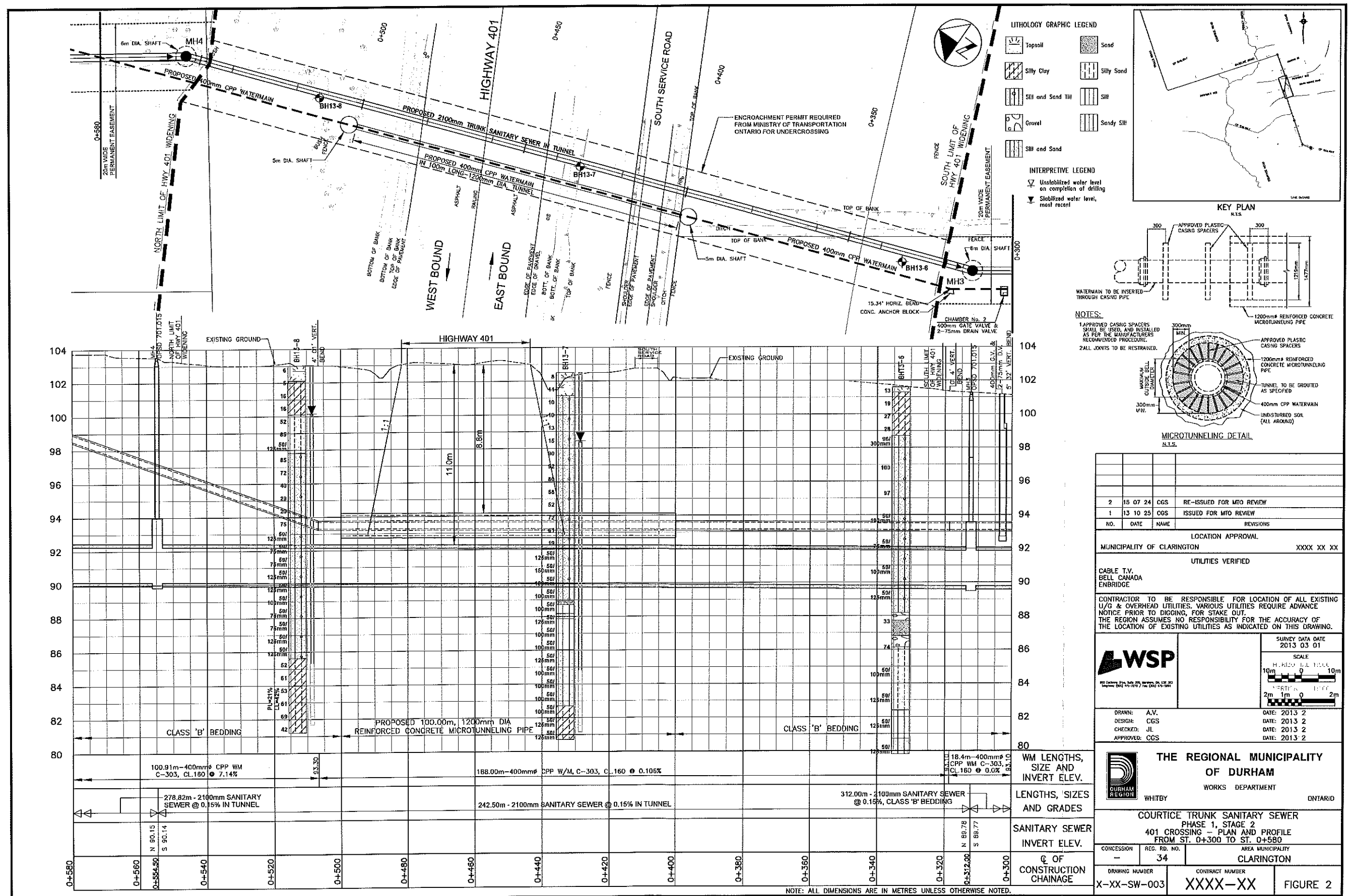
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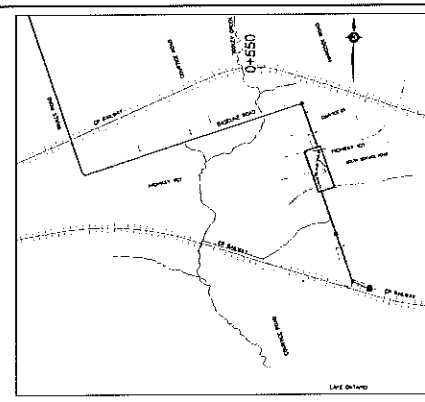
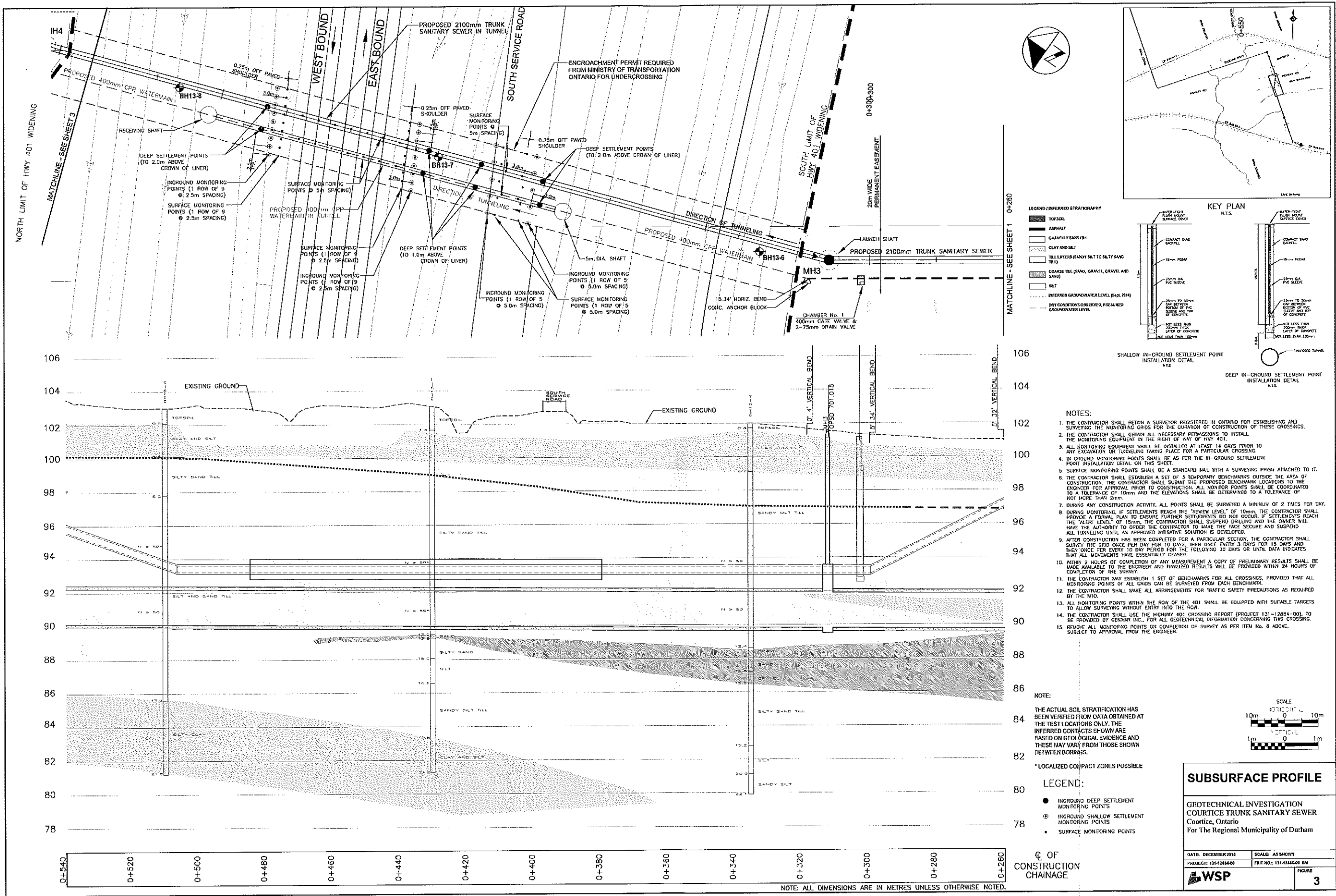
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PROJECT NO:
131-12884-00

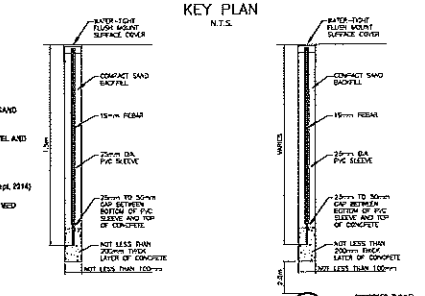
DATE:
APRIL 2014

DRAWING NO:
FIGURE 1



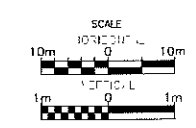


- LEGEND / REFERRED STRATIGRAPHY
- TOPSOIL
 - GRAVELLY SAND/TILL
 - CLAY AND SILT
 - TILL LAYERS (SANDY SILT TO SILTY SAND TILL)
 - COARSE TILL (SAND, GRAVEL, GRAVEL AND SAND)
 - SILT
 - INFERRED GROUNDWATER LEVEL (BASED ON DATA)
 - DAY LOCATIONS OBSERVED, PRESUMED GROUNDWATER LEVEL



- NOTES:
- THE CONTRACTOR SHALL RETAIN A SURVEYOR REGISTERED IN ONTARIO FOR ESTABLISHING AND SURVEYING THE MONITORING GRIDS FOR THE DURATION OF CONSTRUCTION OF THESE CROSSINGS.
 - THE CONTRACTOR SHALL OBTAIN ALL NECESSARY PERMISSIONS TO INSTALL THE MONITORING EQUIPMENT IN THE RIGHT OF WAY OF HWY 401.
 - ALL MONITORING EQUIPMENT SHALL BE INSTALLED AT LEAST 14 DAYS PRIOR TO ANY EXCAVATION OR TUNNELING TAKING PLACE FOR A PARTICULAR CROSSING.
 - IN-GROUND MONITORING POINTS SHALL BE AS PER THE IN-GROUND SETTLEMENT POINT INSTALLATION DETAIL ON THIS SHEET.
 - SURFACE MONITORING POINTS SHALL BE A STANDARD 1/2" WITH A SURVEYING PRISM ATTACHED TO IT.
 - THE CONTRACTOR SHALL ESTABLISH A SET OF 3 TEMPORARY BENCHMARKS OUTSIDE THE AREA OF CONSTRUCTION. THE CONTRACTOR SHALL SUBMIT THE PROPOSED BENCHMARK LOCATIONS TO THE ENGINEER FOR APPROVAL PRIOR TO CONSTRUCTION. ALL MONITORING POINTS SHALL BE COORDINATED TO A TOLERANCE OF 10mm AND THE ELEVATIONS SHALL BE DETERMINED TO A TOLERANCE OF NOT MORE THAN 2mm.
 - DURING ANY CONSTRUCTION ACTIVITY, ALL POINTS SHALL BE SURVEYED A MINIMUM OF 2 TIMES PER DAY.
 - DURING MONITORING, IF SETTLEMENTS REACH THE "ALERT LEVEL" OF 10mm, THE CONTRACTOR SHALL PROVIDE A FORMAL PLAN TO ENSURE FURTHER SETTLEMENTS DO NOT OCCUR. IF SETTLEMENTS REACH THE "ALERT LEVEL" OF 15mm, THE CONTRACTOR SHALL SUSPEND DRILLING AND THE OWNER WILL HAVE THE AUTHORITY TO ORDER THE CONTRACTOR TO MAKE THE FACE SECURE AND SUSPEND ALL TUNNELING UNTIL AN APPROVED MITIGATIVE SOLUTION IS DEVELOPED.
 - AFTER CONSTRUCTION HAS BEEN COMPLETED FOR A PARTICULAR SECTION, THE CONTRACTOR SHALL SURVEY THE GRID ONCE PER DAY FOR 10 DAYS, THEN ONCE EVERY 3 DAYS FOR 15 DAYS AND THEN ONCE PER EVERY 10 DAY PERIOD FOR THE FOLLOWING 30 DAYS OR UNTIL DATA INDICATES THAT ALL MOVEMENTS HAVE ESSENTIALLY CEASED.
 - WITHIN 2 HOURS OF COMPLETION OF ANY MEASUREMENT A COPY OF PRELIMINARY RESULTS SHALL BE MADE AVAILABLE TO THE ENGINEER AND FINALIZED RESULTS WILL BE PROVIDED WITHIN 24 HOURS OF COMPLETION OF THE SURVEY.
 - THE CONTRACTOR MAY ESTABLISH 1 SET OF BENCHMARKS FOR ALL CROSSINGS, PROVIDED THAT ALL MONITORING POINTS OF ALL GRIDS CAN BE SURVEYED FROM EACH BENCHMARK.
 - THE CONTRACTOR SHALL MAKE ALL ARRANGEMENTS FOR TRAFFIC SAFETY PRECAUTIONS AS REQUIRED BY THE MTO.
 - ALL MONITORING POINTS WITHIN THE ROW OF THE 401 SHALL BE EQUIPPED WITH SUITABLE TARGETS TO ALLOW SURVEYING WITHOUT ENTRY INTO THE ROW.
 - THE CONTRACTOR SHALL USE THE HIGHWAY 401 CROSSING REPORT (PROJECT 131-12884-00) TO BE PROVIDED BY GENAR INC. FOR ALL GEOTECHNICAL INFORMATION CONCERNING THIS CROSSING.
 - REMOVE ALL MONITORING POINTS ON COMPLETION OF SURVEY AS PER ITEM NO. 8 ABOVE, SUBJECT TO APPROVAL FROM THE ENGINEER.

- NOTE:
- THE ACTUAL SOIL STRATIFICATION HAS BEEN VERIFIED FROM DATA OBTAINED AT THE TEST LOCATIONS ONLY. THE INFERRED CONTACTS SHOWN ARE BASED ON GEOLOGICAL EVIDENCE AND THESE MAY VARY FROM THOSE SHOWN BETWEEN BORINGS.
- * LOCALIZED COMPACT ZONES POSSIBLE
- LEGEND:
- INGROUND DEEP SETTLEMENT MONITORING POINTS
 - INGROUND SHALLOW SETTLEMENT MONITORING POINTS
 - SURFACE MONITORING POINTS



SUBSURFACE PROFILE

GEOTECHNICAL INVESTIGATION
COURTICE TRUNK SANITARY SEWER
Courtice, Ontario
For The Regional Municipality of Durham

DATE: DECEMBER 2016	SCALE: AS SHOWN
PROJECT: 131-12884-00	FILE NO.: 131-12884-00-01
WSP	FIGURE 3

OF
CONSTRUCTION
CHAINAGE

NOTE: ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.

Appendix A

BOREHOLE EXPLANATION FORMS, BOREHOLE LOGS

BOREHOLE LOG EXPLANATION FORM

This explanatory section provides the background to assist in the use of the borehole logs. Each of the headings used on the borehole log, is briefly explained.

DEPTH

This column gives the depth of interpreted geologic contacts in metres below ground surface.

STRATIGRAPHIC DESCRIPTION

This column gives a description of the soil based on a tactile examination of the samples and/or laboratory test results. Each stratum is described according to the following classification and terminology.

<u>Soil Classification *</u>		<u>Terminology</u>	<u>Proportion</u>
Clay	<0.002 mm		
Silt	0.002 to 0.06 mm	"trace" (eg. trace sand)	<10%
Sand	0.06 to 2 mm	"some" (eg. some sand)	10% - 20%
Gravel	2 to 60 mm	adjective (eg. sandy)	20% - 35%
Cobbles	60 to 200 mm	"and" (eg. and sand)	35% - 50%
Boulders	>200 mm	noun (eg. sand)	>50%

* Extension of MIT Classification system unless otherwise noted.

The use of the geologic term "till" implies that both disseminated coarser grained (sand, gravel, cobbles or boulders) particles and finer grained (silt and clay) particles may occur within the described matrix.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

<u>COHESIONLESS SOIL</u>		<u>COHESIVE SOIL</u>	
Compactness	Standard Penetration Resistance "N", Blows / 0.3 m	Consistency	Standard Penetration Resistance "N", Blows / 0.3 m
Very Loose	0 to 4	Very Soft	0 to 2
Loose	4 to 10	Soft	2 to 4
Compact	10 to 30	Firm	4 to 8
Dense	30 to 50	Stiff	8 to 15
Very Dense	Over 50	Very Stiff	15 to 30
		Hard	Over 30

The moisture conditions of cohesionless and cohesive soils are defined as follows.









<u>COHESIONLESS SOILS</u>		<u>COHESIVE SOILS</u>	
Dry		DTPL	- Drier Than Plastic Limit
Moist		APL	- About Plastic Limit
Wet		WTPL	- Wetter Than Plastic Limit
Saturated		MWTPL	- Much Wetter Than Plastic Limit

STRATIGRAPHY

Symbols may be used to pictorially identify the interpreted stratigraphy of the soil and rock strata.

MONITOR DETAILS

This column shows the position and designation of standpipe and/or piezometer ground water monitors installed in the borehole. Also the water level may be shown for the date indicated.

	Standpipe and Designation		Cement Seal
	Piezometer and Designation		Granular Pack
	Gas Monitor and Designation		Granular Backfill
	Borehole Seal (Peltonite, Bentonite or Hole Plug)		Native Soil Backfill/Cave

Where monitors are placed in separate boreholes, these are shown individually in the "Monitor Details" column. Otherwise, monitors are in the same borehole. For further data regarding seals, screens, etc., the reader is referred to the summary of monitor details table.

SAMPLE

These columns describe the sample type and number, the "N" value, the water content, the percentage recovery, and Rock Quality Designation (RQD), of each sample obtained from the borehole where applicable. The information is recorded at the approximate depth at which the sample was obtained. The legend for sample type is explained below.

SS = Split Spoon	GS = Grab Sample
ST = Thin Walled Shelby Tube	CS = Channel Sample
AS = Auger Flight Sample	WS = Wash Sample
CC = Continuous Core	RC = Rock Core

$$\% \text{ Recovery} = \frac{\text{Length of Core Recovered Per Run}}{\text{Total Length of Run}} \times 100$$

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

ROD Classification

ROD (%)

Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

TEST DATA

The central section of the log provides graphs which are used to plot selected field and laboratory test results at the depth at which they were carried out. The plotting scales are shown at the head of the column.

Dynamic Penetration Resistance - The number of blows required to advance a 51 mm diameter, 60° steel cone fitted to the end of 45 mm OD drill rods, 0.3 m into the subsoil. The cone is driven with a 63.5 kg hammer over a fall of 750 mm.

Standard Penetration Resistance - Standard Penetration Test (SPT) "N" Value - The number of blows required to advance a 51 mm diameter standard split-spoon sampler 300 mm into the subsoil, driven by means of a 63.5 kg hammer falling freely a distance of 750 mm. In cases where the split spoon does not penetrate 300 mm, the number of blows over the distance of actual penetration in millimetres is shown as $\frac{x \text{ Blows}}{\text{mm}}$

Water Content - The ratio of the mass of water to the mass of oven-dry solids in the soil expressed as a percentage.

W_p - Plastic Limit of a fine-grained soil expressed as a percentage as determined from the Atterberg Limit Test.

W_L - Liquid Limit of a fine-grained soil expressed as a percentage as determined from the Atterberg Limit Test.

REMARKS

The last column describes pertinent drilling details, field observations and/or provides an indication of other field or laboratory tests that were performed.

RECORD OF BOREHOLE No BH13-6

1 OF 1

METRIC

LOCATION COURTICE ROAD TRUNK SEWER 131-12884-00

ORIGINATED BY VHG

BOREHOLE TYPE HOLLOW STEM AUGER / 50 mm OD SPLIT SPOON

COMPILED BY RDJ

DATUM GEODETIC DATE 7.26.13 - 7.26.13

CHECKED BY KZK

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								WATER CONTENT (%)		
101.7							20	40	60	80	100							
100.9	TOPSOIL:		1	SS	13													
0.3	BROWN SANDY SILT, TRACE ORGANICS, MOIST		2	SS	19											3 (97)		
	SILTY CLAY:		3	SS	27													
	BROWN SILTY CLAY, TRACE SAND, DTPL, STIFF TO VERY STIFF		4	SS	28													
98.8																		
2.9	SILT AND SAND TILL:		5	SS	96/ 300mm													
	BROWN SILT AND SAND, SOME CLAY, TRACE GRAVEL, MOIST, VERY DENSE		6	SS	100											9 39 (52)		
	- GREY		7	SS	97													
	- TRACE COBBLES (INFERRED)		8	SS	50/ 100mm													
			9	SS	50/ 75mm													
			10	SS	50/ 100mm											10 52 (38)		
	- TO SILTY SAND		11	SS	50/ 125mm													
88.3																		
13.9	GRAVEL:		12	SS	33													
	GREY GRAVEL, SOME SILT, SOME SAND, WET, DENSE																	
86.9	SAND:																	
14.8	GREY SAND, SOME SILT, TRACE TO SOME GRAVEL, WET, DENSE																	
86.2	GRAVEL:		13	SS	74													
15.5	GREY GRAVEL, SOME SAND, WET, VERY DENSE																	
	SILTY SAND:		14	SS	50/ 100mm											10 50 (40)		
	GREY SILTY SAND, TRACE GRAVEL, TRACE CLAY, WET, VERY DENSE																	
			15	SS	50/ 125mm													
82.5																		
19.2	SILT:		16	SS	50/ 125mm													
	GREY SILT, SOME SAND, WET, VERY DENSE																	
80.8																		
20.9	SANDY SILT:		17	SS	50/ 125mm													
79.9	GREY SANDY SILT, TRACE GRAVEL, WET, VERY DENSE																	
21.8	BOREHOLE TERMINATED AT 21.8 m BELOW GROUND SURFACE IN SANDY SILT																	
	BOREHOLE CAVED AT 12.9 mBGS, AND WATER AT 4.9 mBGS INSIDE HOLLOW STEM AUGERS UPON COMPLETION OF DRILLING																	

+ 3, X 3:

Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

ONTARIO MOT MTC COURTICE.GPJ ONTARIO MOT.GDT 4-17-14

RECORD OF BOREHOLE No BH13-7

1 OF 1

METRIC

LOCATION COURTICE ROAD TRUNK SEWER 131-12884-00

ORIGINATED BY VHG

BOREHOLE TYPE HOLLOW STEM AUGER / 50 mm OD SPLIT SPOON

COMPILED BY RDJ

DATUM GEODETIC DATE 7.29.13 - 7.29.13

CHECKED BY KZK

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									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
							WATER CONTENT (%)										
							20 40 60 80 100					10 20 30		GR SA SI CL			
102.9														21 (79)			
0.0	TOPSOIL: BROWN ORGANIC CLAYEY SANDY SILT, MOIST, LOOSE TO COMPACT		1	SS	8												
			2	SS	11												
101.5			3	SS	10												
1.4	SILT AND SAND TILL: BROWNISH GREY TO GREY SILT AND SAND, SOME CLAY, TRACE GRAVEL, TRACE COBBLES (INFERRED), MOIST, COMPACT TO VERY DENSE - SILTY SAND ABOVE 3.1 m		4	SS	10									10 41 (49)			
			5	SS	13												
			6	SS	15												
			7	SS	90												
			8	SS	92												
			9	SS	80									10 38 (52)			
			10	SS	58												
			11	SS	52												
	- SOME GRAVEL, TRACE CLAY, WET		12	SS	72									16 41 (43)			
			13	SS	61												
	- SOME CLAY, TRACE GRAVEL, COMPACT		14	SS	19												
			15	SS	50/ 125mm									1 40 (59)			
			16	SS	50/ 150mm												
			17	SS	50/ 100mm												
			18	SS	50/ 100mm												
89.3			19	SS	50/ 100mm												
89.6	SAND: BROWNISH GREY SAND, WET, VERY DENSE		20	SS	50/ 125mm									2 45 (53)			
88.8			21	SS	50/ 100mm												
88.6	SILT: BROWNISH GREY SILT, SOME SAND, WET, VERY DENSE		22	SS	50/ 100mm												
88.3	SILT AND SAND: BROWNISH GREY SILT AND SAND, SOME CLAY, TRACE GRAVEL, WET, VERY DENSE		23	SS	50/ 125mm									2 23 (75)			
14.6	SILT: GREY SILT, TRACE SAND, TRACE CLAY, WET, VERY DENSE		24	SS	50/ 100mm												
	SANDY SILT: GREY SANDY SILT, SOME CLAY, TRACE GRAVEL, MOIST, VERY DENSE		25	SS	50/ 100mm												
			26	SS	50/ 100mm												
83.1			27	SS	50/ 100mm									1 6 (83)			
19.8	SILTY CLAY: GREY SILTY CLAY, TRACE SAND, TRACE GRAVEL, APL, HARD		28	SS	50/ 125mm												
			29	SS	50/ 125mm												
81.1																	
21.8	BOREHOLE TERMINATED AT 21.8 m BELOW GROUND SURFACE IN SILTY CLAY BOREHOLE FLUSHED DURING DRILLING, WATER AND CAVE NOT MEASURED ON COMPLETION.																

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

ONTARIO MOT MTO COURTICE.GPJ ONTARIO MOT.GDT 4-17-14

RECORD OF BOREHOLE No BH13-8

1 OF 1

METRIC

LOCATION COURTICE ROAD TRUNK SEWER 131-12884-00

ORIGINATED BY VHJ

BOREHOLE TYPE HOLLOW STEM AUGER / 50 mm OD SPLIT SPOON

COMPILED BY RDJ

DATUM GEODETIC DATE 7.30.13 - 7.31.13

CHECKED BY KZK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								○ UNCONFINED	+ FIELD VANE					
								● QUICK TRIAXIAL	× LAB VANE					
103.0							20 40 60 80 100							
0.0	TOPSOIL: BROWN SANDY SILT, SOME TO TRACE CLAY, TRACE ORGANICS, MOIST, LOOSE		1	SS	6									
102.1			2	SS	5									
0.9	SILTY CLAY: BROWN SILTY CLAY, TRACE SAND, DTPL TO APL, FIRM TO VERY STIFF		3	SS	16									2 (98)
			4	SS	16									
100.1			5	SS	52									
2.9	SILTY SAND: BROWN GRAVELLY SILTY SAND, SOME CLAY, MOIST, VERY DENSE		6	SS	85									
			7	SS	50/ 125mm									26 34 (40)
97.8			8	SS	85									
5.2	SILT AND SAND TILL: GREY SILT AND SAND, SOME CLAY, TRACE TO SOME GRAVEL, TRACE COBBLE (INFERRED), WET, VERY DENSE		9	SS	72									
	- DENSE		10	SS	40									
	- COMPACT		11	SS	29									13 36 (51)
			12	SS	20									
	- VERY DENSE, TRACE COBBLES (INFERRED)		13	SS	75									
			14	SS	50/ 125mm									8 41 (51)
			15	SS	50/ 75mm									
			16	SS	50/ 75mm									
			17	SS	50/ 125mm									
			18	SS	50/ 125mm									6 36 (58)
			19	SS	50/ 100mm									
			20	SS	50/ 75mm									
			21	SS	50/ 75mm									
			22	SS	50/ 125mm									
			23	SS	50/ 125mm									6 34 (60)
85.6			24	SS	52									
17.4	SILTY CLAY: GREY SILTY CLAY, TRACE SAND, APL TO DTPL, HARD		25	SS	61									
			26	SS	53									
			27	SS	61									1 (99)
			28	SS	69									
81.2			29	SS	42									
21.8	BOREHOLE TERMINATED AT 21.8 m BELOW GROUND SURFACE IN SILTY CLAY BOREHOLE FLUSHED DURING DRILLING, WATER AND CAVE NOT MEASURED ON COMPLETION.													

+ 3, x 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

ONTARIO MOT MTO COURTICE.GPJ ONTARIO MOT.GDT 4-17-14

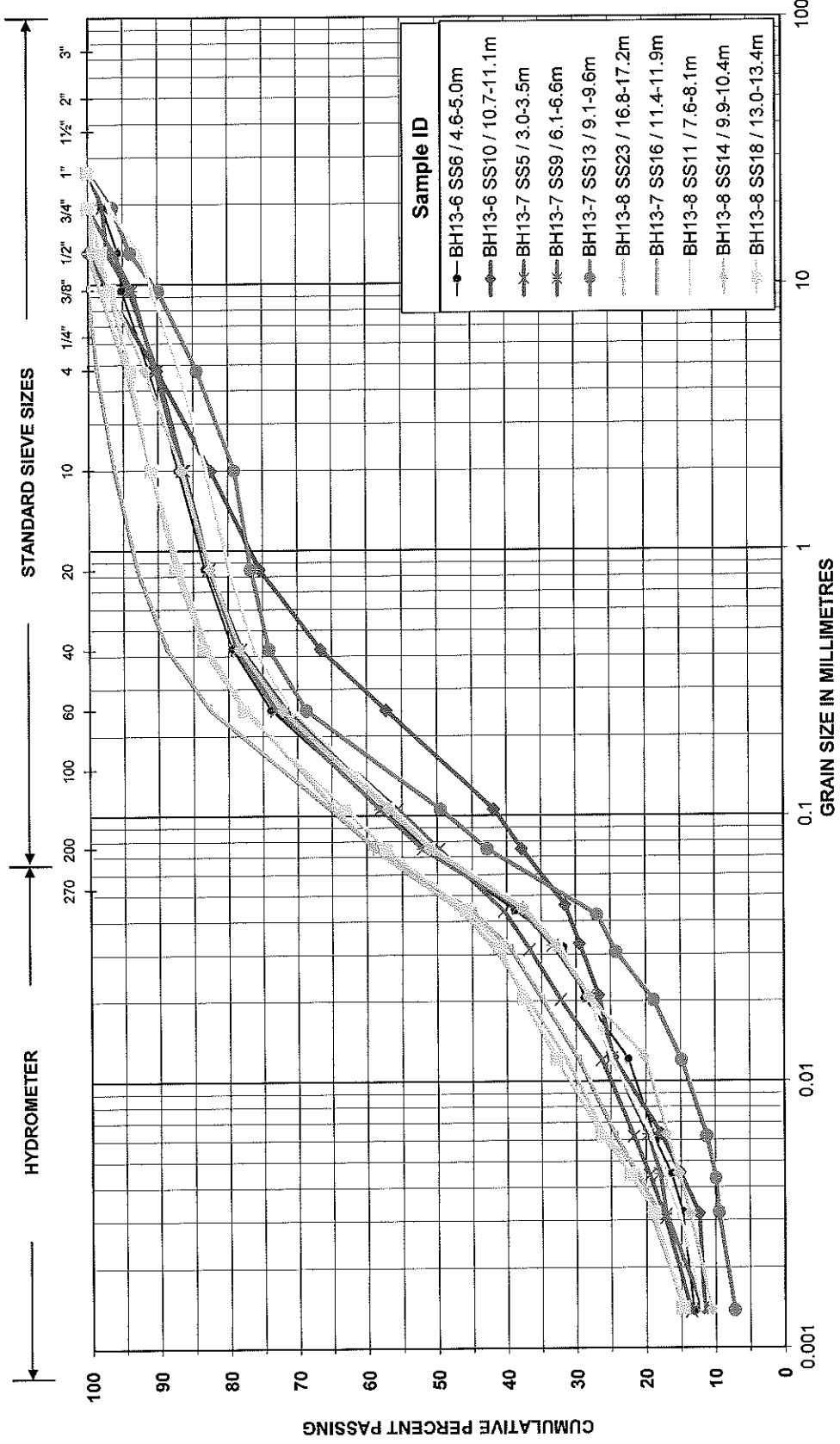
Appendix B

GEOTECHNICAL LABORATORY ANALYSIS



PARTICLE SIZE DISTRIBUTION

ASTM D422

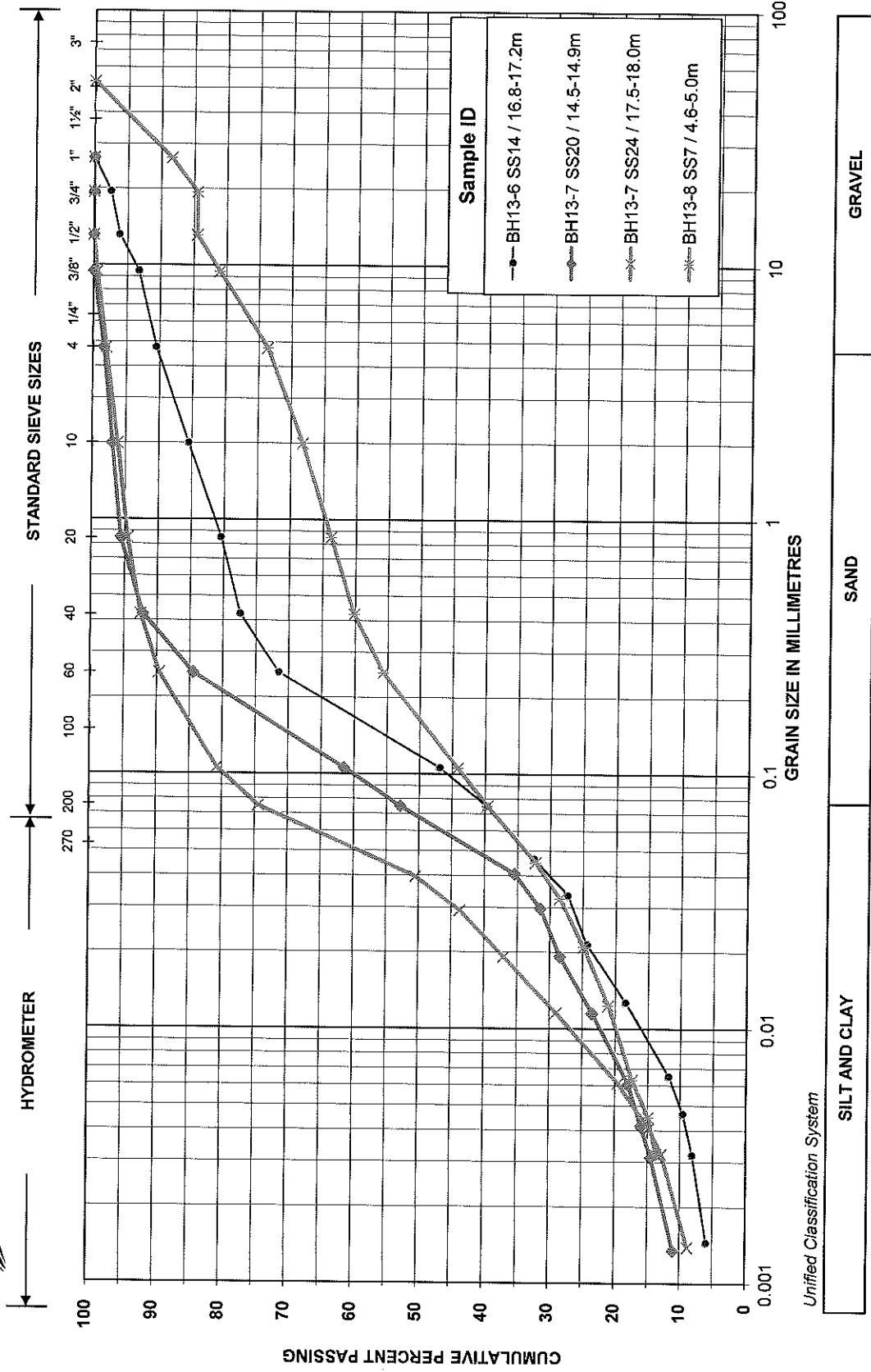


SILT AND CLAY		SAND	GRAVEL
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Project Name:	Courice Trunk Sewer - MTO	Project No.:	131-12884-00
Remarks:	Sand and Silt to Silty Sand Till		



PARTICLE SIZE DISTRIBUTION ASTM D422

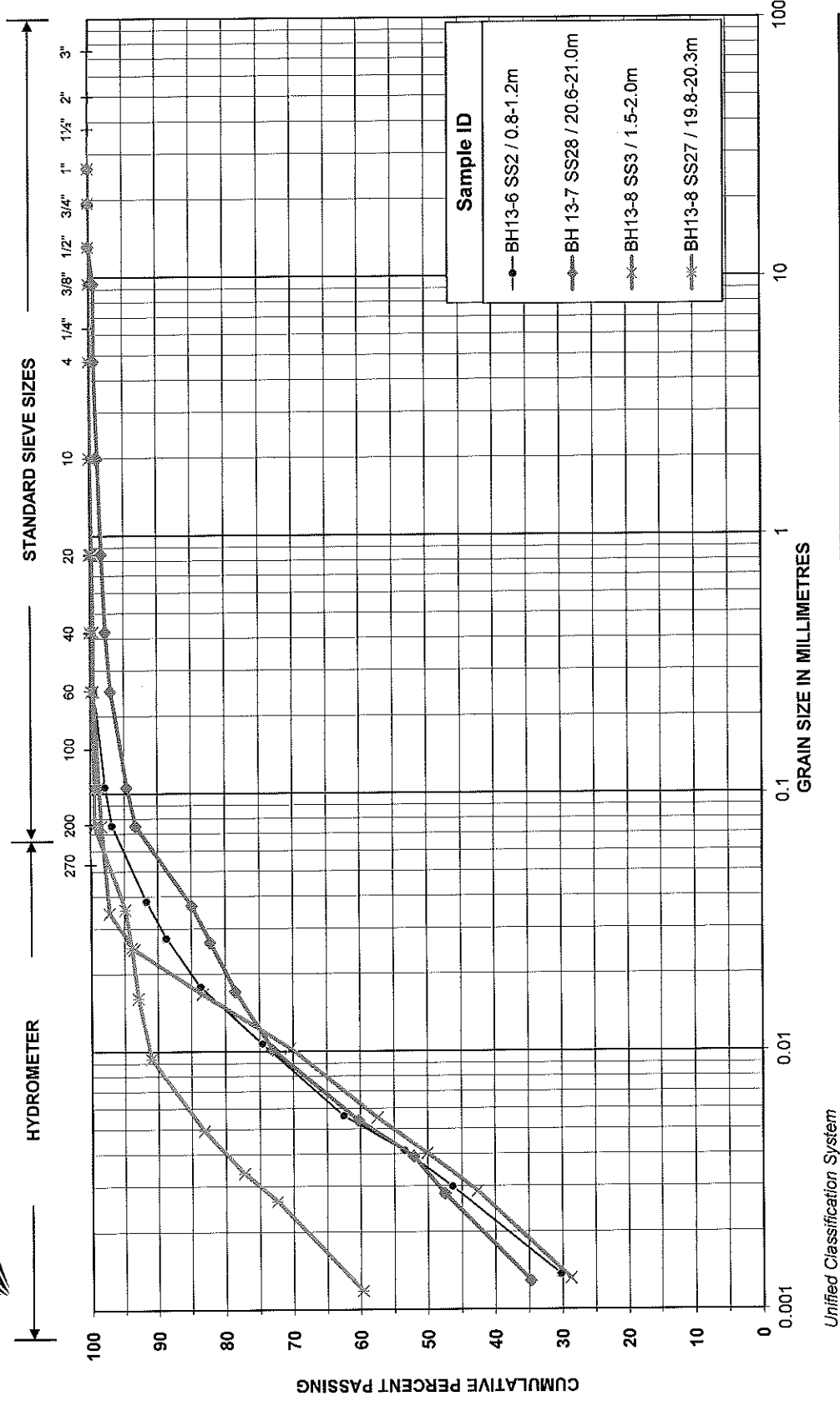


Project Name:	Courtice Trunk Sewer - MTO	Project No.:	131-12884-00
Remarks:	Sands, Silts and Gravels		



PARTICLE SIZE DISTRIBUTION

ASTM D422



Unified Classification System		
SILT AND CLAY	SAND	GRAVEL

Project Name: Courty Trunk Sewer - MTO

Project No.: 131-12884-00

Remarks: Silty Clay

WSP LABORATORY - ATTERBERG LIMITS			
Date:	15-Aug-13	Job No.:	131-12884-00
Project Name:	Courtice Trunk Sewer - MTO	Tech.:	KLK
Borehole/Sample No.: BH13-8 / SS27			

Liquid Limit Test

Number of Shocks	16	21	28
Tin No.	K20	45	23
Tin + Wet soil	27.46	29.48	27.11
Tin + Dry soil	25.44	27.05	25.36
Wt. of Water	2.02	2.43	1.75
Wt. of Tin	20.84	21.33	21.15
Wt. of Dry Soil	4.6	5.72	4.21
Water Content	44	42	42

Plastic Limit Test

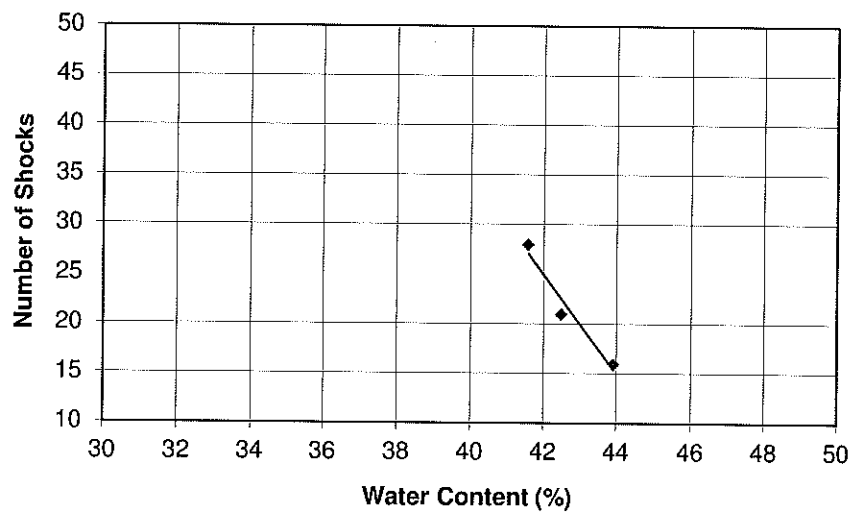
Tin No.	41	18
Tin + Wet soil	26	26.66
Tin + Dry soil	25	25.79
Wt. of Water	1	0.87
Wt. of Tin	20.16	21.61
Wt. of Dry Soil	4.84	4.18
Water Content	21	21

Liquid Limit, (W_L) 42
 Plastic Limit, (W_P) 21
 Plasticity Index ($I_P = W_L - W_P$) 21

Control Results

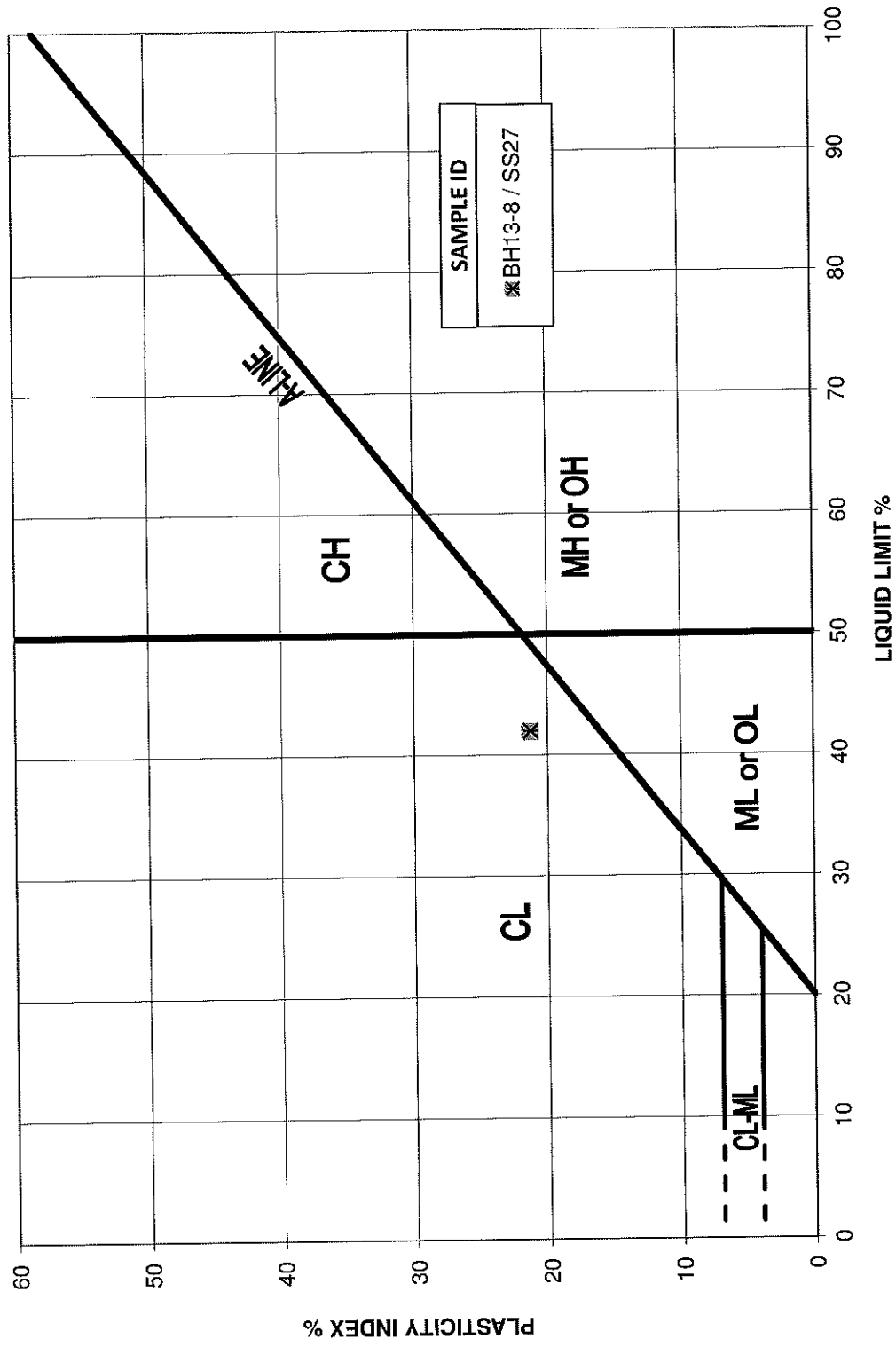
Liquid Limit, (W_L) 33
 Plastic Limit, (W_P) 18
 Plasticity Index ($I_P = W_L - W_P$) 15

Liquid Limit





Atterberg Limits Plasticity Chart



Appendix C

NON-STANDARD SPECIAL PROVISION (NSSP)

Date: April 17, 2014

Project No.: 131-12884-00 Phase 1 Section 2

Subject: Non-Standard Special Provision, Settlement Instrumentation and Monitoring

1. Instrumentation Monitoring

The work specified in this section includes furnishing and installing instruments for the monitoring of settlement and ground stability.

Ground stability and settlement shall be monitored by in-ground and surface monitoring points at the locations shown on the settlement monitoring plans. The equipment and procedures used for settlement monitoring during construction must be capable of surveying the settlement points to within ± 1 mm. Surface monitoring points installed on the pavement shall be hardened steel markers treated or coated to resist corrosion, with an exposed convex head having a minimum diameter of 12 mm and similar to surveyor's PK nails. Markers shall be rigidly affixed so as not to move relative to the surface to which it is attached.

In unpaved areas, settlement monitoring points shall be 19 mm rebar encased in a 75 mm SCH40 PVC pipe, as shown on the settlement monitoring drawings (Figure 3). The assembly shall be placed in a drill hole and backfilled with uniform sand as shown on the Contract Drawings.

Traffic control may be required to safely access settlement monitoring points. Traffic shall be managed by the Contractor in accordance with the Ontario Traffic Manual (OTM).

The Contractor shall install all surface settlement instruments a minimum of two weeks prior to the start of works.

A Pavement Condition survey shall be completed prior to the start of the job to identify distress manifestations in the roadway within the tunnelling area.

The surface settlement instruments shall be clearly labelled for easy identification.

The Settlement Monitoring Plan provided in Figure 3 indicates the approximate locations of monitoring instruments and provide typical instrument details. The monitoring point locations are approximate and must be confirmed by the Contractor in consultation with the Contract Administrator prior to installation and construction and may have to be adjusted in the field to suit local conditions/constraints.

The Contractor shall submit to the Contract Administrator a site plan showing the locations of the monitoring points, a geodetic survey of the settlement monitoring points including station, offset and elevation recorded at the following time intervals:

- Three (3) consecutive readings consisting of one (1) reading per day for three (3) days at least seven (7) days prior to commencement of the work (Baseline Reading);
- Two (2) times daily during tunnelling, including weekends, and
- One (1) time per day after completion of the work for a period of ten (10) days, and then once every three (3) days for fifteen (15) days, and then once per every ten (10) day period for thirty (30) days, or until such time at which all parties agree that further movement, if any, has stopped.

PETERBOROUGH 294 Rink Street, Suite 103, Peterborough ON K9J 2K2 Tel.: (705) 743-6850 Fax: (705) 743-6854

Preliminary readings should be made available to the Geotechnical Engineer within 2 hours after completion of any measurement. All finalized readings shall be submitted to the Contract Administrator, Geotechnical Engineer, and provided to the MTO within 24 hours during tunnelling operations. Each report shall include all survey data collected in tabular and graphical format as plots of time versus settlement in comparison to survey data collected prior to commencement of the work.

The Contractor shall avoid damaging instrumentation during construction. Instrumentation that is damaged as a result of the Contractor's operation shall be repaired or replaced by the Contractor within one business day. The costs for replacement/repair shall be borne by the Contractor.

At the completion of the job, the Contractor shall abandon all instrumentations installed during the course of the Work.

2. Criteria for Assessment of Subsidence/Heave

Based on the monitoring of ground movement as specified in Subsection 1 of this NSSP, the following represents trigger levels that define magnitude of movement and corresponding action:

- **Review Level:** If a maximum value of 10 mm relative to the baseline readings is reached, the Contractor shall review or modify the method, rate of sequence of construction or ground stabilization measures to mitigate further ground displacement.
 - If the Review Level is exceeded, the Contractor shall immediately notify the CA, MTO, and Geotechnical Engineer, and review and discuss response actions. The Contractor shall submit a plan of action to prevent Alert Levels from being reached. All construction work shall be continued such that the Alert Level is not reached.
- **Alert Level:** If a maximum value of 15 mm relative to the baseline readings is reached, the Contractor shall cease construction operations, inform the Contract Administrator, MTO, and Geotechnical Engineer, and execute pre-planned measures to secure the site, to mitigate further movements and to assure safety of public and maintain traffic.
- **No construction shall take place until all the following conditions are satisfied:**
 - The cause of the settlement has been identified.
 - The Contractor submits a corrective/preventive plan.
 - Any corrective and/or preventive measure deemed necessary by the Contractor is implemented.
 - The CA deems it is safe to proceed.

3. Basis of Payment

Payment at the contract price shall be full compensation for providing all labour, equipment, and materials required for the supply and installation of monitoring equipment and equipment removal, settlement monitoring, and submission of settlement data to the Contract Administrator, Geotechnical Engineer, and the land owner (MTO).

Appendix D

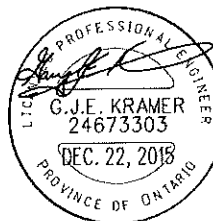
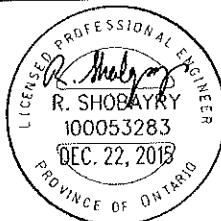
**HMM PEER REVIEW FOR PROPOSED TRUNK SANITARY SEWER AND
WATERMAIN CROSSING OF HIGHWAY 401, COURTICE, ONTARIO**



WSP Canada Inc
High-Complexity Tunneling Review

**Review of Proposed Trunk Sanitary Sewer and Watermain Crossing of
Highway 401 and Courtice, Ontario**

Issue and Revision Record					
Rev	Date	Originator (Print) (Signature)	Checker (Print) (Signature)	Approver (Print) (Signature)	Description
0	2015-07-17	C. Cosby	R. Shobayry	G. Kramer	Issue for Use
1	2015-12-22	C. Cosby	R. Shobayry	G. Kramer	Issued with Two Seals
	Signatures:				



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APPENDICES:

Appendix A - Project Drawings

Appendix B - Crossing Design Calculations



1. Introduction

1.1 Project Overview

The Regional Municipality of Durham (Region) proposes to undertake the construction of the Courtice Trunk Sanitary Sewer and Watermain near Clarington, Ontario. The Trunk Sanitary Sewer is to be installed via tunneling across the entire alignment presented in this report. The Watermain will be advanced by tunneling immediately below Highway 401 (approx. 100 m in length), and by open cut installation elsewhere.

WSP Canada Inc. has prepared a Foundation Investigation and Design Report (WSP, 2014). Hatch Mott MacDonald (HMM), as a qualified consultant under Ministry of Transportation Ontario (MTO) Registry Appraisal and Qualification System (RAQS) for High Complexity Tunneling has been retained by WSP Canada Inc. to complete the following tasks:

1. Review geotechnical investigations, plan and profile for planned crossing.
2. Confirm adequacy of investigations for MTO RAQS requirements.
3. Review recommendations for tunnelling and support systems.
4. Estimate settlement associated with proposed tunnelling and shaft methods.
5. Assess potential impacts due to these settlements.
6. Provide recommendations for monitoring and tunnelling methods to be built into contract documents.
7. Prepare and seal RAQS design report for submission to MTO.
8. Review completed specifications for compliance.

Documents provided by WSP for HMM review include the following:

- Foundation Investigation and Design Report, prepared by WSP and dated April, 2014.

1.2 Purpose

The purpose of this report is to provide a review and assessment of the Foundation Investigation and Design Report (WSP 2014) prepared for the Proposed Trunk Sanitary Sewer and Watermain Crossings for adequacy against MTO requirements for High Complexity Tunneling.

This report also provides recommendations, which in HMMs opinion as reviewers, must be incorporated into the Foundation Investigation and Design Report (WSP, 2014) and/or project specifications.



1.3 Project Constraints and Assumptions

The evaluations contained in this report are based, in part, on the following project constraints and assumptions:

1. Closure of any portion of Highway 401 will not be permitted, as such, only trenchless construction methods will be considered for the Trunk Sanitary Sewer and Watermain Crossings.

2. Review of Foundation Investigation Report

Details of the geotechnical investigation and encountered geotechnical and groundwater conditions for the proposed crossings of Highway 401 are provided in Sections 3 through 6 of the Foundation Investigation and Design Report (WSP, 2014), as summarized below.

The geotechnical investigation consisted of three (3) boreholes drilled to 21.8 m below existing grade. The boreholes terminated without meeting bedrock 8.1 m below anticipated tunnel depths, which is three (3) times the proposed tunnel diameter (2.7 m). The borehole location plan and subsurface section from the Foundation Investigation and Design Report (WSP 2014). is included as Appendix A.

The first borehole (BH13-6) is located 70 m south of South Service Road, in between the Sewer and Watermain alignments. It is approximately 60 m from the Proposed Watermain Southern Shaft, and 18 m from the Proposed Trunk Sanitary Sewer Southern Shaft.

The second borehole (BH13-7) is located 105 m north of BH13-6, in between South Service Road and Highway 401. This boring is located on the Proposed Trunk Sanitary Sewer Alignment.

The final borehole (BH13-8) is located 80 m north of BH13-7, and 35 m north of Highway 401. It is approximately 6 m from the Proposed Watermain Northern Shaft, and 39 m from the Proposed Trunk Sanitary Sewer Northern Shaft.

A standard penetration test was performed on samples taken at 0.75 m (overburden) and 1.5 m (below tunnel invert) intervals as required by MTO guidelines.

The laboratory testing consisted of a 'Routine Test' as defined in Guidelines For Foundation Engineering - Tunnelling Specialty For Corridor Encroachment. This test includes natural water content, Atterberg limits and grain size distribution.

In HMM's opinion, the geotechnical investigation completed was sufficient in its scope to verify design assumptions and provide the contractor with adequate subsurface information for design and construction planning.



3. Feasibility of Trenchless Construction Methods

Feasibility of Trenchless Crossing Methods was evaluated in the Foundation Investigation and Design Report (WSP, 2014). In that report, the feasibility of the following Trenchless Methods were evaluated:

- Jack and Bore.
- Tunnel Boring Machine (EPB).
- Tunnel Boring Machine (Open Face).
- Pipe Ramming.
- Microtunnelling.
- Horizontal Directional Drilling.

HMM concurs that the technologies evaluated by WSP are those which would be traditionally considered for crossings of this nature (i.e., diameter and length). However, given the tunnel zone subsurface conditions and anticipated flowing behaviour described in the Foundation Investigation Report (WSP, 2014), and the fact that dewatering of the tunnel alignment below Highway 401 will likely not be possible due to access requirements, it is HMM's opinion that the specifications should limit the contractor's choice of methods to those which can apply positive pressure to the excavation face, balancing earth and groundwater pressures. This limits the potential crossing methods to EPB tunnelling, Microtunnelling or Horizontal Directional Drilling (HDD).

3.1 Trunk Sanitary Sewer

Section 8.3 - Tunnelling Method of the Foundation Investigation and Design Report (WSP, 2014) recommends the 2100 mm diameter trunk sanitary sewer be installed within a 2.7 m diameter tunnel (two pass installation) constructed using an earth pressure balance Tunnel Boring Machine (TBM). It is understood this recommendation is due largely to the excess hydrostatic head (7 to 10 m above the trunk sanitary sewer invert) along the tunnel alignment, and the large diameter of the trunk sanitary sewer.

HMM recommends that use of jack and bore and pipe ramming should explicitly be excluded for construction of the trunk sanitary sewer tunnel, since those open-face methods are not considered viable options due to the risk of overmining in flowing ground conditions. It is further recommended that use of existing older EPB TBMs that are not equipped with screw conveyors be excluded for construction of the trunk sanitary sewer. In that view, use of an earth pressure balance tunnel boring machine in a two-pass installation would be the most suitable methods. HMM also notes that microtunnelling is also feasible for 2100 mm diameter installations (single pass). The minimum tunnel face pressure for either the EPB TBM or microtunnelling is 140 kPa, with a target face pressure of 190 kPa.



3.2 400 mm Watermain

Section 9.3 - Tunnelling Method of the Foundation Investigation and Design Report (WSP, 2014) recommends using Jack and Bore methods to construct the Watermain Tunnel

HMM does not agree that Jack and Bore is feasible for this application, due to the high risk of overmining and related ground surface settlement associated with the use of this open-face method in flowing ground conditions. HMM recommends that a closed-face method, such as microtunnelling be used instead. The minimum tunnel face pressure for microtunnelling is 100 kPa, with a target face pressure of 150 kPa.

HMM notes that in Section 9.3 - Tunnelling Method of the Foundation Investigation and Design Report (WSP, 2014), WSP states that HDD "Does not permit installation of steel liner". This is incorrect. HDD can be used to install a steel casing. However the setback distances required in a properly-designed HDD bore path (i.e., a bore path designed with entry and exit radii appropriate for the casing pipe diameter, and with a depth of cover appropriate to mitigate hydrofracture concerns) likely make HDD cost-ineffective. Additionally, HDD installations typically require curvatures on entry and exit, which is not likely conducive to the proposed CPP product pipe.

4. Trunk Sanitary Sewer

4.1 Geotechnical Conditions

Subsurface conditions reported in the Foundation Investigation and Design Report (WSP 2014) along the Proposed Trunk Sanitary Sewer Tunnel Alignment include very dense silt and sand glacial till. Occasional compact zones are reported to be possible within the tunnel zone, as they were observed in close proximity to the tunnel zone in borehole location BH13-7. A full description of geotechnical conditions encountered along with logs of project borings is provided in the Foundation Investigation and Design Report (WSP 2014).

4.2 Launch and Reception Shafts

It is understood that the trunk sanitary sewer tunnel will be launched and received from 6 m diameter circular shafts as detailed in the borehole location plan and subsurface section from the Foundation Investigation and Design Report (WSP 2014; see Appendix A).

The first shaft (MH 3) is located 9 m south of the Southern limit of Highway 401 widening and 94 m south of South Service Road with an invert elevation of 89.4 m (12.3 m below existing grade).

The second shaft (MH 4) is located 7 m north of the Northern limit of Highway 401 widening and 77 m north of Highway 401 with an invert elevation of 89.8 m (13.2 m below existing grade).

These shafts are approximately 12 m to 13 m deep, and are located 75 m to 90 m from the edge of pavement for Highway 401 and South Service Road respectively. It is not anticipated that shaft construction will impact MTO facilities. MTO facilities are located outside of the zone of influence for shaft construction, which is estimated as extending along a 1:1 (H:V) plane projected from the shaft bottoms.



4.3 Loading Conditions

It is understood that WSP intends to leave the design of the Trunk Sanitary Sewer Tunnel initial and final support to the Contractor. Section 9.4 - Temporary and Final Support of the Foundation Investigation and Design Report (WSP 2014) provides vertical ground stress and suggests the use of all-round pressure equal to vertical ground stress for the design of initial and permanent support for the trunk sanitary sewer tunnel. In HMM's opinion in-situ horizontal stresses should also be presented for the case where segmental lining is selected as the support system. Correspondingly, the specifications should require the designer to consider range of vertical and horizontal ground stresses in the design of tunnel lining.

4.4 Impacts Due to Tunnelling

The primary impacts anticipated for the proposed crossing of Highway 401 include the potential for ground loss and settlement-related damage to the highway surface. HMM used methodology from 'Ground movements resulting from urban tunnelling: predictions and effects' (Rankin, 1988) to determine potential settlement due to tunnelling, which are presented in Appendix B.

In the analysis presented in Appendix B, HMM assumed a mined diameter of 3.2 m (2.7 m inside diameter as indicated by WSP, along with an assumed 250 mm segment thickness) and considered the combined effects of Watermain and the Trunk Sanitary Sewer installations. With good tunnel construction workmanship, HMM anticipates that settlements, as measured above the trunk sanitary sewer, for closed-face tunnelling to be approximately 4 mm to 6 mm due to trunk sanitary sewer tunnelling and approximately 4 mm to 7 mm due to combined tunnelling of Watermain and Trunk Sanitary Sewer.

In the Foundation Investigation and Design Report (WSP 2014), WSP used a mined diameter of 2.7 m in their calculations and predicted the expected settlement for the Watermain and the trunk sanitary sewer separately. Using values for closed face methods, Section 8.5 - Anticipated Ground Deformation of the Foundation Investigation and Design Report (WSP, 2014) predicted 'less than 5 mm' of settlement. It is noted that WSP used a depth to springline of 11 m in their calculations, which differs from their profile drawing which shows 11 m as the depth to the top of the tunnel

5. Watermain

5.1 Geotechnical Conditions

Subsurface conditions reported in the Foundation Investigation and Design Report (WSP 2014) for the proposed 1200 mm diameter Watermain Tunnel Alignment include very dense silt and sand glacial till. Occasional compact zones are reported as possible within the tunnel zone, as they were observed in close proximity to the tunnel zones in borehole location BH13-7. Very dense silty sand with elevated percentage of gravel was encountered in borehole location BH13-8 less than 4 m above the proposed obvert. A full description of geotechnical conditions encountered along with logs of project borings is provided in the Foundation Investigation and Design Report (WSP 2014).



5.2 Launch and Reception Shafts

Two 4 x 8 m rectangular shafts are proposed to construct the 400 mm diameter Watermain Crossing.

The first shaft (401-S) is located 6 m south of South Service Road with an invert elevation of 92.5 m (10.4 m below existing grade).

The second shaft (401-N) is located 23 m north of Highway 401 with an invert elevation of 89.8 m (13.2 m below existing grade).

These shafts are approximately 10 m to 13 m deep, and are located 6 m to 23 m from the edge of pavement for South Service Road and Highway 401 respectively.

Settlement analysis was performed on Watermain shaft 401-S due to its proximity to South Service Road. At a distance of 6 m from the edge of excavation, South Service Road is expected to experience between 4 mm and 9 mm of vertical displacement. Such settlements is expected to be in addition to any settlement due to tunneling. If the shaft can be relocated so it has a minimum 10.5 m setback from the edge of pavement, shaft excavation is not anticipated to impact the pavement. Full calculations are presented in Appendix B.

It is not anticipated that Watermain shaft 401-N construction will impact MTO facilities. MTO facilities are located outside of the zone of influence for shaft construction, which is estimated as extending along a 1:1 (H:V) plane projected from the shaft bottom.

5.3 Casing Design

As it is currently proposed, the Watermain Tunnel will be advanced under approximately 9 m of overburden soil cover, through tills comprised of varying silt and sand concentrations, which are generally very dense with occasional compact zones.

Section 9.4 - Temporary and Final Support of the Foundation Investigation and Design Report (WSP 2014) indicates that a 19 mm steel casing will be used as initial support for the Watermain Tunnel. HMM has completed calculations to confirm the adequacy of the proposed casing thickness and concluded that 19 mm thickness should be sufficient against buckling under anticipated earth loading. HMM's calculations are included in Appendix B. Casing calculations assumes intimate and continuous contact between the casing and ground, therefore it is recommended that the overcut annulus between the jacked casing and surrounding ground be grouted on completion of tunnelling.

5.4 Impacts Due to Tunnelling

The primary impacts anticipated for the proposed crossing of Highway 401 include the potential for settlement-related damage to the highway surface. We used methodology from 'Ground movements resulting from urban tunnelling: predictions and effects' (Rankin, 1988) to determine estimates of potential settlement, which are presented in Appendix B.

WSP predicted the expected settlement for the Watermain and the Trunk Sanitary Sewer separately. Calculations for the combined effects of both tunnels are included in Appendix B.



With good tunnel construction workmanship, it is anticipated that settlement impact, as measured above the Watermain, for closed-face tunnelling to be approximately 1mm due to Watermain Tunnelling and approximately 3 mm due to combined tunnelling of Watermain and Trunk Sanitary Sewer.

Using values for closed face methods, Section 9.5 - Anticipated Ground Deformation of the Foundation Investigation and Design Report (WSP, 2014) predicted 'less than 2 mm' of settlement.

6. Settlement Monitoring

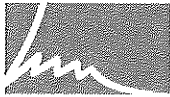
In the Foundation Investigation and Design Report (WSP, 2014), WSP has specified the location of surface monitoring points. Although the spacing of settlement monitoring points does not meet the maximum 5.0 m spacing required by MTO guidelines, it is HMM's opinion that proposed arrangement of settlement monitoring points should provide reasonable indication of achieved settlement. The proposed frequency of readings meets the requirement of MTO guidelines.

7. Conclusions and Recommendations

HMM reviewed Foundation Investigation and Design Report (WSP, 2014) and proposes the following modifications to the report or specifications:

7.1 Trunk Sanitary Sewer

1. In HMM's opinion the specifications should limit crossing methods to closed face tunnelling methods only, such as EPB tunnelling or Microtunnelling.
2. The minimum tunnel face pressure of 140 kPa, with a target face pressure of 190 kPa is recommended to be incorporated in the specifications. Contractor shall adjust face pressure to maintain subsidence or heave below the thresholds as specified in the Foundation Investigation and Design Report (WSP, 2014).
3. Use of precast concrete segmental lining as the support for the 2100 mm sewer crossing is recommended. It should be clarified in the report that design of segmental lining will be the responsibility of the contractor and the design shall be performed for anticipated range ground in-situ horizontal and vertical stresses. WSP should provide vertical and horizontal ground stresses for the design of segmental lining.
4. The combined effects of both Watermain and Trunk Sanitary Sewer Tunnel should be considered in the settlement impact assessment. Based on HMM's calculations under anticipated ground conditions and using good tunnel construction workmanship, a settlement impact of 5 mm to 6 mm is predicted for trunk sanitary sewer tunneling and a total settlement impact of 5 mm to 7 mm is predicted for combined effects of both the Watermain and trunk sanitary sewer tunnels. These settlements are measured above the trunk sanitary sewer, assuming closed-face tunnelling/microtunnelling.



7.2 Watermain

1. In HMM's opinion the specifications should limit crossing methods to closed face tunnelling methods only, such as Microtunnelling.
2. The minimum tunnel face pressure of 100 kPa, with a target face pressure of 150 kPa is recommended to be incorporated in the specifications. Contractor shall adjust face pressure level to maintain subsidence or heave below the thresholds as specified in the Foundation Investigation and Design Report (WSP, 2014).
3. External annulus grouting is recommended for the steel casing. It is further recommended that the internal annulus (the annulus between the intrados of the steel casing and the extrados of the Watermain) be grouted as well.
4. With current location of shaft 401-S, South Service Road is expected to experience a settlement between 4 mm and 9 mm of vertical displacement due to excavation of Watermain Launch Shaft which is at a distance of 6 meters from the edge of excavation. Such settlement is anticipated to be in addition to any predicted settlement due to tunnelling. If the shaft 401-S can be relocated so it has a minimum 10.5 m setback from the edge of pavement, shaft excavation is not anticipated to impact the pavement.
5. The combined effects of both Watermain and Trunk Sanitary Sewer Tunnel should be considered in the settlement impact assessment. Based on HMM's calculations under anticipated ground conditions and using good tunnel construction workmanship, a settlement impact of 1 mm is predicted for Watermain tunneling and a total settlement impact of 3 mm to 4 mm is predicted for combined effects of both the Watermain and trunk sanitary sewer tunnels. These settlements are measured above the Watermain, assuming closed-face tunnelling/microtunnelling.

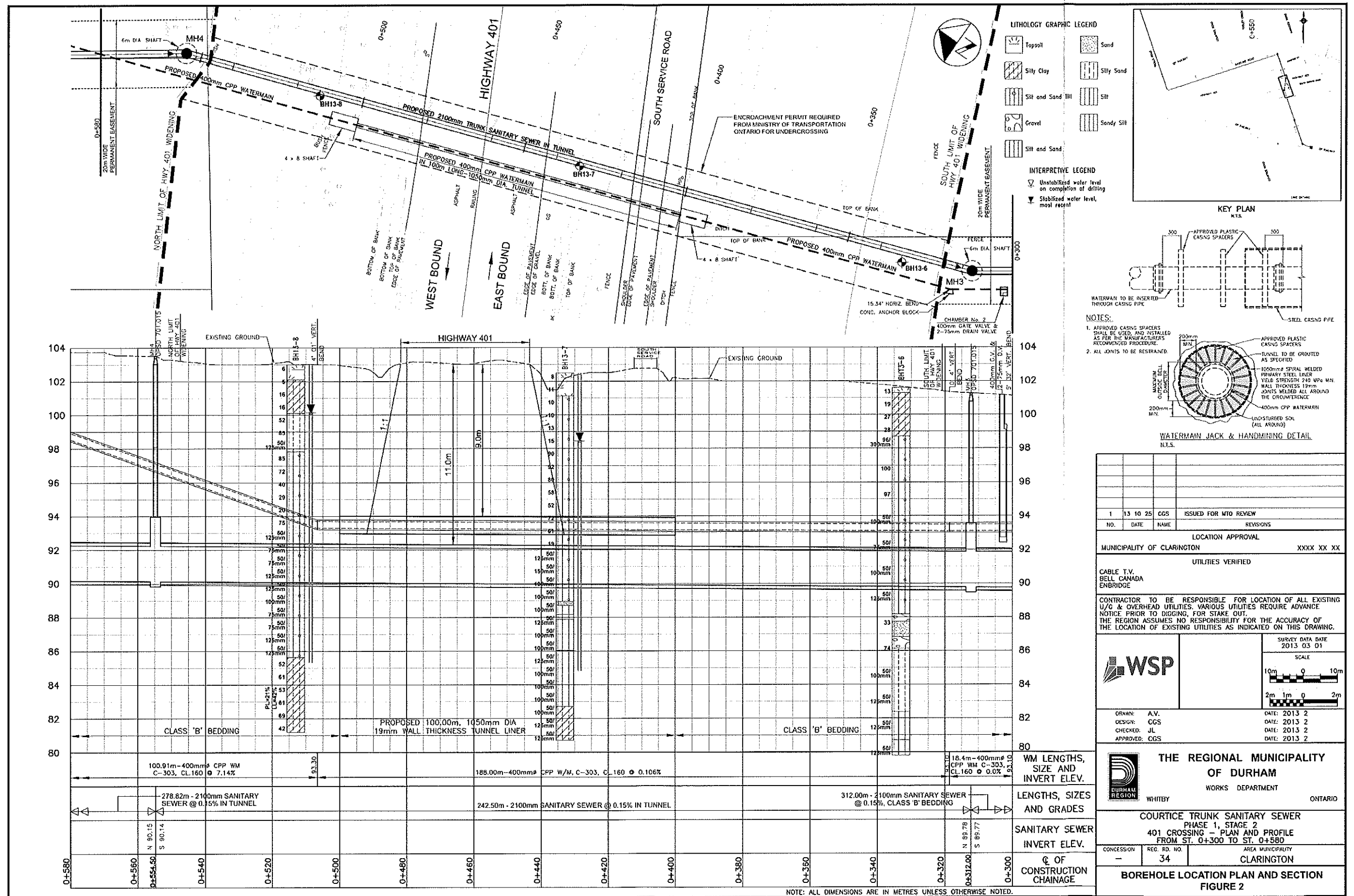


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WSP Canada Inc - High-Complexity Tunneling Review
Review of Proposed Trunk Sanitary Sewer and Watermain Crossing of Highway 401 and Courtice, Ontario -
December 22, 2015

Appendix A

Project Drawings



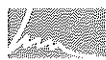


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WSP Canada Inc - High-Complexity Tunneling Review
Review of Proposed Trunk Sanitary Sewer and Watermain Crossing of Highway 401 and Courtice, Ontario -
December 22, 2015

Appendix B

Crossing Design Calculations



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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Settlement Estimate - Watermain**

PROJECT NO.

358397

MADE BY

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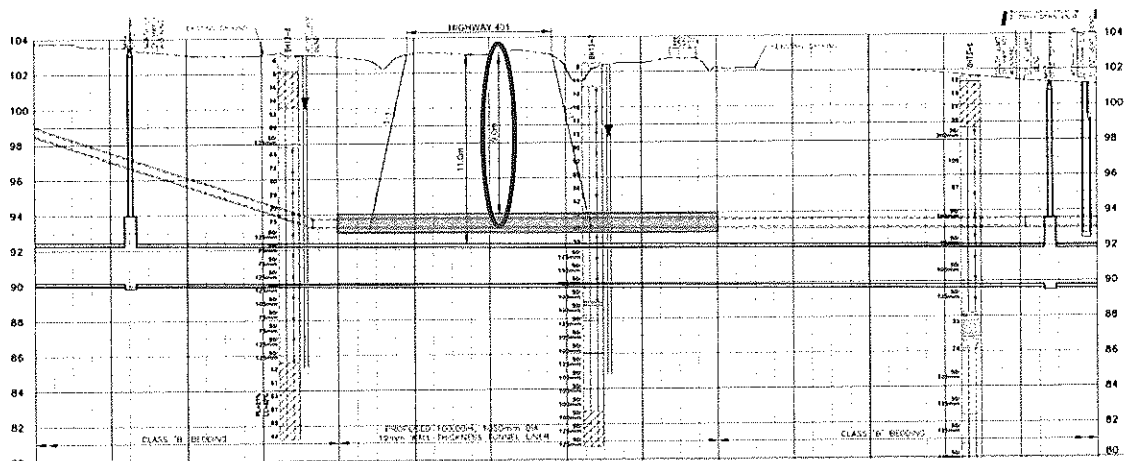
CHECKED BY

DATE 21-Dec-2015

DATE

1.0 Calculation Purpose

The purpose of this calculation sheet is to estimate the ground surface settlement due to the proposed 1050mm Watermain tunnel crossing Ontario Highway 401. The critical location for settlement is estimated to be at the centre of the highway.



2.0 Engineering Assumptions and Tunnel Parameters

Description	Symbol	Value	Units	Notes
Tunnel Springline Depth	Z_0	9.6	m	9.0m cover, plus half tunnel diameter
Tunnel Outside Diameter	D_0	1.20	m	1200mm diameter steel liner
Ground Characterization, min	K_{min}	0.40		WSP Foundation Investigation
Ground Characterization, max	K_{max}	0.60		O'Reilly, New (1982), Glacial Deposits
Volume Loss, min	V_{Lmin}	1.00	%	WSP Foundation Investigation
Volume Loss, max	V_{Lmax}	4.00	%	O'Reilly, New (1982), Glacial Deposits

3.0 Equations

Trough Width Parameter: $i_y = K \times Z_0$

Maximum Settlement: $w_{max} = \frac{0.125 \times V_L \times [D_0/2]^4}{i_y}$

Vertical Settlement Profile: $w = w_{max} \times \exp\left(-\frac{y^2}{2i_y^2}\right)$

4.0 Trough Width Parameter

Settlement and ground movements associated with a single tunnel appear as a Gaussian curve shaped trough at the ground surface centred above the tunnel centreline. The shape of the settlement trough depends on the trough width parameter.

Case 1 and 3: K_{min}

Case 2 and 4: K_{min}

$$K = 0.40$$
$$i_y = 3.84$$

$$K = 0.60$$
$$i_y = 5.76$$



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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Settlement Estimate - Watermain**

PROJECT NO.

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5.0 Estimated Settlement

Maximum vertical displacements:

where V_L = Volume Loss
 K = Ground characterization
 D_0 = Tunnel Springline Depth
 i_y = Trough width parameter ($K \times z_0$)
 w_{max} = maximum settlement

Case 1: K_{min} , V_{Lmin}

$K = 0.40$
 $V_L = 1.00$
 $i_y = 3.84$

$w_{max} = 1.2 \text{ mm}$

Case 2: K_{max} , V_{Lmin}

$K = 0.60$
 $V_L = 1.00$
 $i_y = 5.76$

$w_{max} = 0.8 \text{ mm}$

Case 3: K_{min} , V_{Lmax}

$K = 0.40$
 $V_L = 4.00$
 $i_y = 3.84$

$w_{max} = 4.7 \text{ mm}$

Case 4: K_{max} , V_{Lmax}

$K = 0.60$
 $V_L = 4.00$
 $i_y = 5.76$

$w_{max} = 3.1 \text{ mm}$

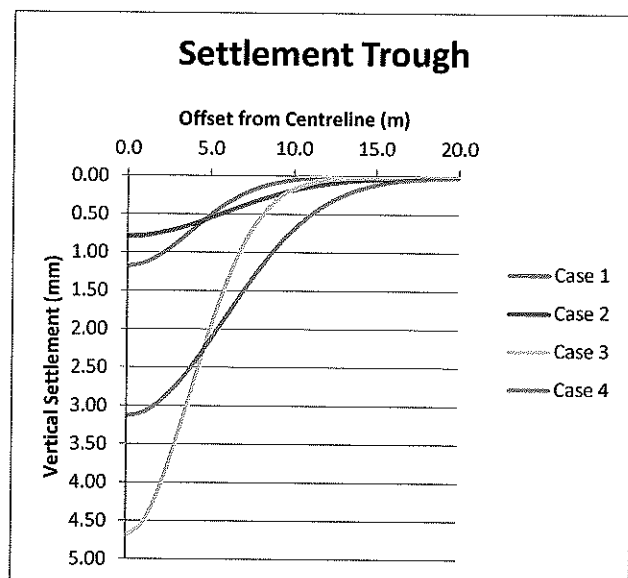
6.0 Settlement Trough

The profile of the vertical settlement can be predicted using the following equation:

Vertical Settlement Profile: $w = w_{max} \times \exp\left(-\frac{y^2}{2i_y^2}\right)$

where w_{max} = maximum settlement
 y = Horizontal Offset perpendicular to alignment
 i_y = Trough width parameter ($K \times z_0$)

Offset (m)	Case 1	Case 2	Case 3	Case 4
0.0	1.17	0.78	4.69	3.13
1.0	1.13	0.77	4.53	3.08
2.0	1.02	0.74	4.09	2.94
3.0	0.86	0.68	3.45	2.73
4.0	0.68	0.61	2.72	2.46
5.0	0.50	0.54	2.01	2.14
6.0	0.35	0.45	1.38	1.82
7.0	0.22	0.37	0.89	1.49
8.0	0.13	0.30	0.54	1.19
9.0	0.08	0.23	0.30	0.92
10.0	0.04	0.17	0.16	0.69
11.0	0.02	0.13	0.08	0.50
12.0	0.01	0.09	0.04	0.36
13.0	0.00	0.06	0.02	0.24
14.0	0.00	0.04	0.01	0.16
15.0	0.00	0.03	0.00	0.11
16.0	0.00	0.02	0.00	0.07
17.0	0.00	0.01	0.00	0.04
18.0	0.00	0.01	0.00	0.02
19.0	0.00	0.00	0.00	0.01
20.0	0.00	0.00	0.00	0.01





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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Settlement Estimate - Watermain**

PROJECT NO.

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7.0 Damage Assessment

The impact to Ontario Highway 401 is assessed based on the maximum ground surface settlement over the centreline of the tunnel.

Maximum allowable ground surface settlement: 15 mm

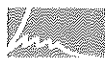
Based on WSP Technical Memo 'Non-Standard Special Provision, Settlement Instrumentation and Monitoring' at ground surface movement of 10mm or greater triggers 'Alert Level'.

			Maximum Settlement (mm)	Percent Allowable	Damage Assessment
Case 1	K _{min}	V _{Lmin}	1.2	8%	Acceptable I
Case 2	K _{max}	V _{Lmin}	0.8	5%	Acceptable I
Case 3	K _{min}	V _{Lmax}	4.7	31%	Acceptable II
Case 4	K _{max}	V _{Lmax}	3.1	21%	Acceptable II

Potential Impact Categories	
Acceptable I	<10%
Acceptable II	10-50%
Acceptable III	50-100%
Not Acceptable A	100-150%
Not Acceptable B	>150%

8.0 References

1. Lake L.M., Rankin W.J. and Hawley J. "Prediction and Effects of Ground Movements Caused by Tunnelling in Soft Ground Beneath Urban Areas" (CIRIA Funders Report/CP/5, September 1992).
2. O'Reilly, M.P. and New, B.M. (1982). "Settlements above tunnels in the United Kingdom-their magnitude and prediction", Tunnelling, pp173-181. London:IMM.



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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Settlement Estimate - Sewer**

PROJECT NO.

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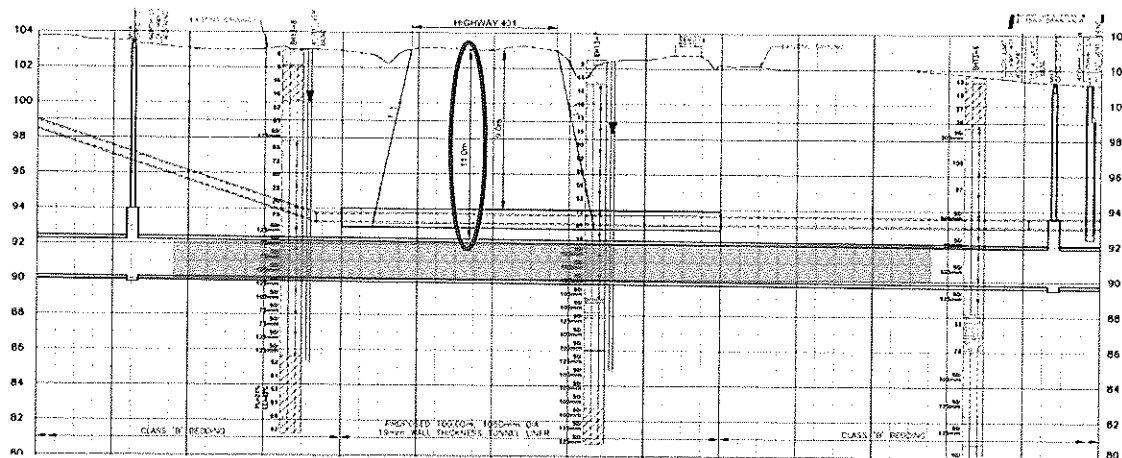
DATE 9-Jun-2015

CHECKED BY

DATE

1.0 Calculation Purpose

The purpose of this calculation sheet is to estimate the ground surface settlement due to the proposed 2.7m Trunk Sewer tunnel crossing Ontario Highway 401. The critical location for settlement is estimated to be at the centre of the highway.



2.0 Engineering Assumptions and Tunnel Parameters

Description	Symbol	Value	Units	Notes
Tunnel Springline Depth	Z_0	12.6	m	11.0m cover, plus half tunnel diameter
Tunnel Outside Diameter	D_0	3.20	m	2.7m diameter steel liner
Ground Characterization, min	K_{min}	0.40		WSP Foundation Investigation
Ground Characterization, max	K_{max}	0.60		O'Reilly, New (1982), Glacial Deposits
Volume Loss, min	V_{Lmin}	1.00	%	WSP Foundation Investigation
Volume Loss, max	V_{Lmax}	4.00	%	O'Reilly, New (1982), Glacial Deposits

3.0 Equations

Trough Width Parameter: $i_y = K \times z_0$

Maximum Settlement: $w_{max} = \frac{0.125 \times V_L \times [D_0/2]^4}{i_y}$

Vertical Settlement Profile: $w = w_{max} \times \exp\left(-\frac{y^2}{2i_y^2}\right)$

4.0 Trough Width Parameter

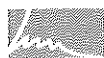
Settlement and ground movements associated with a single tunnel appear as a Gaussian curve shaped trough at the ground surface centred above the tunnel centreline. The shape of the settlement trough depends on the trough width parameter.

Case 1 and 3: K_{min}

Case 2 and 4: K_{min}

$K = 0.40$
 $i_y = 5.04$

$K = 0.60$
 $i_y = 7.56$



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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Settlement Estimate - Sewer**

PROJECT NO.

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5.0 Estimated Settlement

Maximum vertical displacements:

where V_L = Volume Loss
 K = Ground characterization
 D_0 = Tunnel Springline Depth
 i_y = Trough width parameter ($K \times z_0$)
 w_{max} = maximum settlement

Case 1: K_{min} , V_{Lmin}

$K = 0.40$
 $V_L = 1.00$
 $i_y = 5.04$

$w_{max} = 6.3 \text{ mm}$

Case 2: K_{max} , V_{Lmin}

$K = 0.60$
 $V_L = 1.00$
 $i_y = 7.56$

$w_{max} = 4.2 \text{ mm}$

Case 3: K_{min} , V_{Lmax}

$K = 0.40$
 $V_L = 4.00$
 $i_y = 5.04$

$w_{max} = 25.4 \text{ mm}$

Case 4: K_{max} , V_{Lmax}

$K = 0.60$
 $V_L = 4.00$
 $i_y = 7.56$

$w_{max} = 16.9 \text{ mm}$

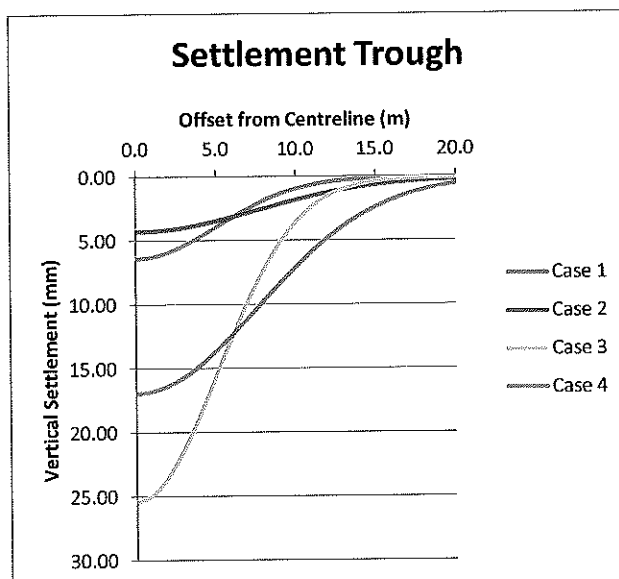
6.0 Settlement Trough

The profile of the vertical settlement can be predicted using the following equation:

Vertical Settlement Profile: $w = w_{max} \times \exp\left(-\frac{y^2}{2i_y^2}\right)$

where w_{max} = maximum settlement
 y = Horizontal Offset perpendicular to alignment
 i_y = Trough width parameter ($K \times z_0$)

Offset (m)	Case 1	Case 2	Case 3	Case 4
0.0	6.35	4.23	25.40	16.93
1.0	6.23	4.20	24.90	16.78
2.0	5.87	4.09	23.47	16.35
3.0	5.32	3.91	21.27	15.65
4.0	4.63	3.68	18.54	14.72
5.0	3.88	3.40	15.53	13.61
6.0	3.13	3.09	12.50	12.36
7.0	2.42	2.76	9.68	11.03
8.0	1.80	2.42	7.21	9.67
9.0	1.29	2.08	5.16	8.34
10.0	0.89	1.76	3.55	7.06
11.0	0.59	1.47	2.35	5.87
12.0	0.37	1.20	1.49	4.80
13.0	0.23	0.97	0.91	3.86
14.0	0.13	0.76	0.54	3.05
15.0	0.08	0.59	0.30	2.37
16.0	0.04	0.45	0.16	1.80
17.0	0.02	0.34	0.09	1.35
18.0	0.01	0.25	0.04	0.99
19.0	0.01	0.18	0.02	0.72
20.0	0.00	0.13	0.01	0.51





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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Settlement Estimate - Sewer**

PROJECT NO.

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DATE 9-Jun-2015

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7.0 Damage Assessment

The impact to Ontario Highway 401 is assessed based on the maximum ground surface settlement over the centreline of the tunnel.

Maximum allowable ground surface settlement: 15 mm

Based on WSP Technical Memo 'Non-Standard Special Provision, Settlement Instrumentation and Monitoring' at ground surface movement of 10mm or greater triggers 'Alert Level'.

			Maximum Settlement (mm)	Percent Allowable	Damage Assessment
Case 1	K_{min}	V_{Lmin}	6.3	42%	Acceptable II
Case 2	K_{max}	V_{Lmin}	4.2	28%	Acceptable II
Case 3	K_{min}	V_{Lmax}	25.4	169%	Not Acceptable B
Case 4	K_{max}	V_{Lmax}	16.9	113%	Not Acceptable A

Potential Impact Categories	
Acceptable I	<10%
Acceptable II	10-50%
Acceptable III	50-100%
Not Acceptable A	100-150%
Not Acceptable B	>150%

8.0 References

1. Lake L.M., Rankin W.J. and Hawley J. "Prediction and Effects of Ground Movements Caused by Tunnelling in Soft Ground Beneath Urban Areas" (CIRIA Funders Report/CP/5, September 1992).
2. O'Reilly, M.P. and New, B.M. (1982). "Settlements above tunnels in the United Kingdom-their magnitude and prediction", Tunnelling, pp173-181. London:IMM.



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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Settlement Estimate - Combined**

PROJECT NO.

358397

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DATE 21-Dec-2015

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DATE

1.0 Combined Watermain and Sewer Damage Assessment

The cumulative ground surface movement due to both the watermain and the sewer can be found by adding the effects of the individual tunnels, taking into account the lateral distance between the tunnels (approximately 7 meters).

Maximum allowable ground surface settlement: 15 mm

Based on WSP Technical Memo 'Non-Standard Special Provision, Settlement Instrumentation and Monitoring' at ground surface movement of 10mm or greater triggers 'Alert Level'.

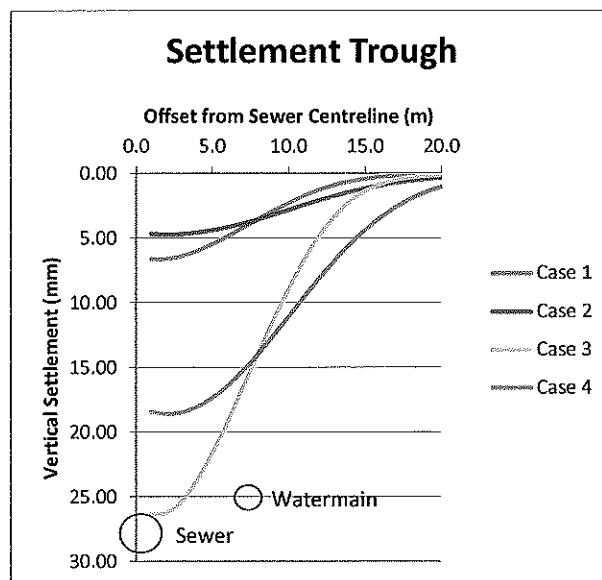
			Settlement due to Watermain	Settlement due to Trunk Sewer	Cumulative Settlement	Percent Allowable	Damage Assessment
Case 1	K_{min}	V_{Lmin}	1.2	6.3	6.6	44%	Acceptable II
Case 2	K_{max}	V_{Lmin}	0.8	4.2	4.6	31%	Acceptable II
Case 3	K_{min}	V_{Lmax}	4.7	25.4	26.3	175%	Not Acceptable B
Case 4	K_{max}	V_{Lmax}	3.1	16.9	18.6	124%	Not Acceptable A

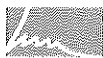
Potential Impact Categories

Acceptable I	<10%
Acceptable II	10-50%
Acceptable III	50-100%
Not Acceptable A	100-150%
Not Acceptable B	>150%

Horizontal Distance between Watermain and Trunk Sewer: 7 m

Offset (m)	Case 1	Case 2	Case 3	Case 4
0.0	6.56	4.60	26.26	18.40
1.0	6.56	4.64	26.23	18.57
2.0	6.35	4.61	25.39	18.46
3.0	5.96	4.51	23.85	18.05
4.0	5.44	4.34	21.78	17.37
5.0	4.84	4.11	19.36	16.45
6.0	4.19	3.83	16.75	15.31
7.0	3.52	3.50	14.07	14.00
8.0	2.86	3.14	11.44	12.58
9.0	2.24	2.77	8.98	11.08
10.0	1.69	2.40	6.76	9.58
11.0	1.22	2.03	4.87	8.12
12.0	0.83	1.68	3.33	6.74
13.0	0.54	1.37	2.17	5.47
14.0	0.33	1.09	1.33	4.35
15.0	0.19	0.85	0.77	3.39
16.0	0.11	0.64	0.42	2.58
17.0	0.05	0.48	0.22	1.92
18.0	0.03	0.35	0.11	1.39
19.0	0.01	0.25	0.05	0.99
20.0	0.01	0.17	0.02	0.69





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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Watermain - Steel Casing Design**

PROJECT NO.

358397

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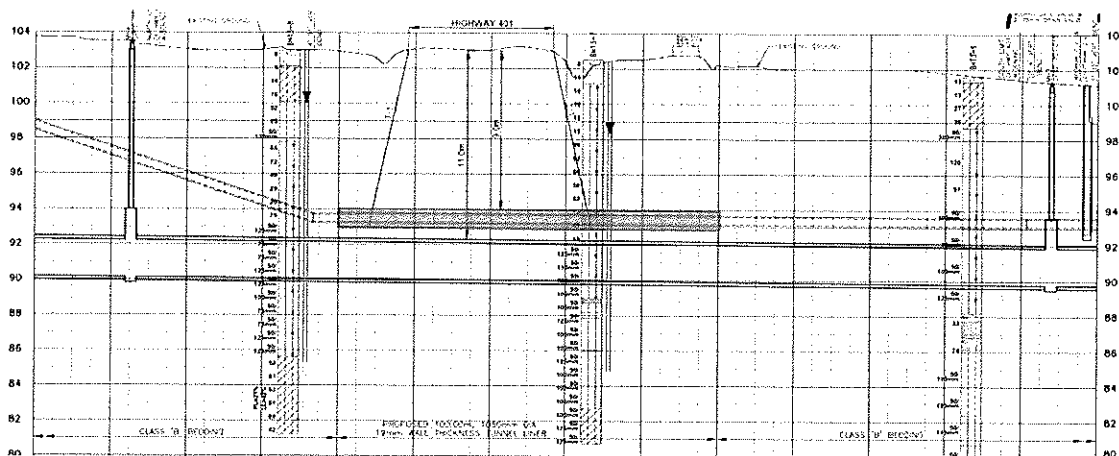
DATE 9-Jun-2015

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DATE

1.0 Calculation Purpose

The purpose of this calculation sheet is to justify the selection of a 19mm steel casing for the watermain tunnel crossing Ontario Highway 401 at Courtice using methods from "Guideline for the Design of Buried Steel Pipe".



2.0 Engineering Assumptions and Tunnel Parameters

Description	Symbol	Value	Units	Notes
Height of Earth above Pipe	C	9.0	m	9.0m cover to top of pipe
Height of Water above Pipe	h_w	5.0	m	Approximate height
Soil Bulk Unit Weight	γ	20	kN/m ³	From WSP Foundation Investigation
Soil Moisture Content	w	10	%	From WSP Foundation Investigation
Soil Dry Unit Weight	γ_d	18.2	kN/m ³	From WSP Foundation Investigation
Water Unit Weight	γ_w	9.8	kN/m ³	
Young's Modulus for Pipe	E	200	GPa	Steel Pipe Stiffness
Soil Reaction Modulus	E'	6895	kN/m ²	Bureau of Reclamation' values of E'
Pipe I.D.	ID	1.05	m	WSP Foundation Investigation
Pipe Thickness	T	0.019	m	

3.0 Vertical Earth Load

Vertical Earth Load
(Below water table)

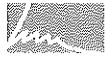
$$P_v = h_w \times \gamma_w + (1 - 0.3 \times (h_w / C)) \times \gamma_d \times C$$

$$P_v = 185.4 \text{ kN/m}^2$$

4.0 Live Load on Pipe

From Table 4.1 of Guideline for the Design of Buried Steel Pipe:

$$\text{Live Load Applied to Pipe } P_p = 0$$



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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Watermain - Steel Casing Design**

PROJECT NO.

358397

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DATE 9-Jun-2015

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DATE

5.0 Ovality

$$\frac{dy}{D} = \frac{D_1 \times K \times P}{\left(\frac{EI_{eq}}{R^3} \right) + 0.061E'}$$

$$EI_{eq} = E \times \frac{t^3}{12}$$

Where dy = vertical pipe deflection

D = Pipe outside diameter = 1.09 m

K = Bedding constant = 0.1

P = Pressure on pipe ($P_v + P_p$) = 185 kN/m²

EI_{eq} = Pipe wall stiffness per pipe length = 114.3 kNm

R = Pipe Radius = 0.525 m

E' = Soil Reaction Modulus = 6895 kN/m²

t = Pipe thickness = 0.019 m

$$\frac{dy}{D} = 0.015$$

6.0 Wall Bending Stress

$$\begin{aligned} wbs &= 4 \times E \times \frac{dy}{D} \times \frac{t}{D} \\ &= 214 \text{ MPa} \end{aligned}$$

7.0 Buckling Limit

$$\frac{1}{FS} \sqrt{32 R_w B' E' \frac{EI_{eq}}{D^3}}$$

$$R_w = 1 - 0.3 \times \left(\frac{h_w}{C} \right)$$

$$B' = \frac{1}{1 + 4 e^{(-0.065 C/D)}}$$

Where FS = Factor of Safety = 3 for $C/D < 2$

h_w = Height of water above pipe = 5 m

EI_{eq} = Pipe wall stiffness per pipe length = 114.3 kNm

D = Pipe outside diameter = 1.09 m

C = Height of soil above pipe = 9 m

$$\text{Buckling limit} = 730 \text{ kN/m}^2$$

Vertical loading of 185.4 kN/m² is less than buckling limit

Buckling okay

19mm casing is okay



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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Settlement Estimate - WM Shaft 401-S**

PROJECT NO.

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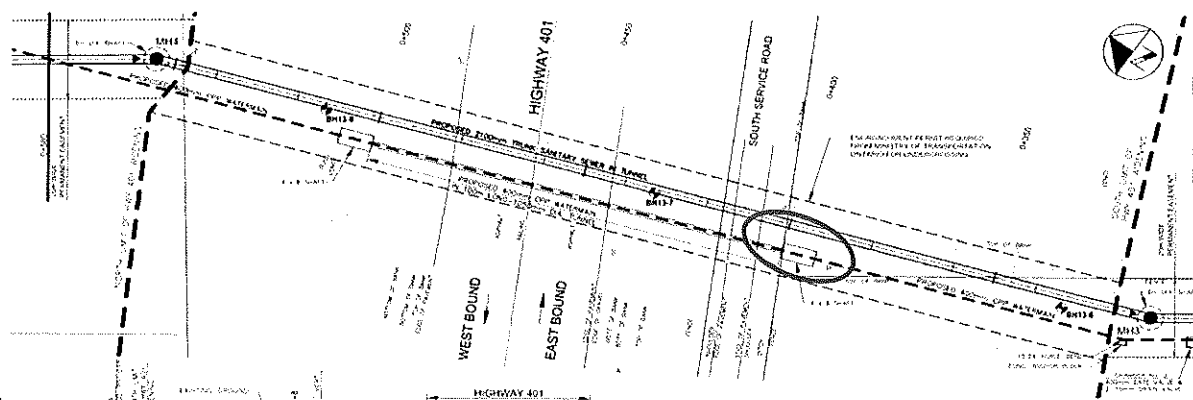
DATE 9-Jun-2015

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DATE

1.0 Calculation Purpose

The purpose of this calculation sheet is to estimate the ground surface settlement due to the proposed 4 x 8 meter watermain shaft construction near Ontario Highway 401. This shaft is of concern due to its proximity (<10m) from South Service Road.



2.0 Engineering Assumptions and Tunnel Parameters

Description	Symbol	Value	Units	Notes
Excavation Depth	d_{shaft}	10.4	m	
Tunnel Depth	d_{tunnel}	9.5	m	
Zone of Influence Factor	K	2		
Parabolic Expected	$W_{p,\text{exp}}$	0.10	%	Percent of excavated depth
Parabolic Design	$W_{p,\text{des}}$	0.20	%	Percent of excavated depth
Concave Expected	$W_{c,\text{exp}}$	0.00	%	Percent of excavated depth
Concave Design	$W_{c,\text{des}}$	0.00	%	Percent of excavated depth

3.0 Tunnel Settlement Trough

The pattern of ground surface movement behind the support of excavation can take two general forms; 1) a parabolic shape for walls that are considered to be cantilevered and flexible, having little friction behind the wall and unrestrained rotation about the toe; and 2) a concave shape where there is significant friction or where the wall is stiff enough to resist cantilevered bending, producing an outward bulging of the wall at depth.

The estimated zone of influence for both settlement profiles is found below:

$$\begin{aligned}y_{\text{max}} &= \text{zone of influence} \\&= K \times d_{\text{tunnel}} \\y_{\text{max}} &= 19.0 \text{ m}\end{aligned}$$



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CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
Settlement Estimate - WM Shaft 401-S**

PROJECT NO.

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DATE 9-Jun-2015

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3.0 Tunnel Settlement Trough

w_{max} = maximum vertical settlement
= $W \times Z$

where: W = Percent of excavation depth
 Z = effective depth

Case 1: Parabolic Expected

$W_{p,exp} = 0.1 \%$

$w_{p,max} = 9.50 \text{ mm}$

Case 2: Parabolic Design

$W_{p,des} = 0.2 \%$

$w_{p,max} = 19.00 \text{ mm}$

Case 3: Concave Expected

$W_{c,exp} = 0.0 \%$

$w_{c,max} = 0.00 \text{ mm}$

Case 4: Concave Design

$W_{c,des} = 0.0 \%$

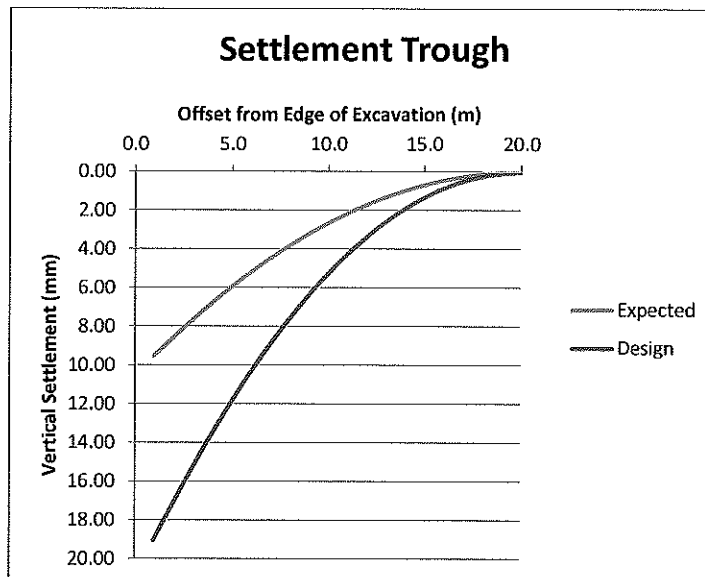
$w_{c,max} = 0.00 \text{ mm}$

w = vertical settlement

$w = [w_{max} \times ((y_{max} - y) / y_{max})^2] + [w_{max} \times 6 \times (y / y_{max}) \times \exp(-6 \times (y / y_{max})^2)]$

where: w_{max} = maximum vertical settlement
 y_{max} = zone of influence = 19.0
 y = horizontal offset perpendicular to edge of excavation


Offset (m)	Expected	Design
0.0	9.50	19.00
1.0	8.53	17.05
2.0	7.61	15.21
3.0	6.74	13.47
4.0	5.92	11.84
5.0	5.16	10.32
6.0	4.45	8.89
7.0	3.79	7.58
8.0	3.18	6.37
9.0	2.63	5.26
10.0	2.13	4.26
11.0	1.68	3.37
12.0	1.29	2.58
13.0	0.95	1.89
14.0	0.66	1.32
15.0	0.42	0.84
16.0	0.24	0.47
17.0	0.11	0.21
18.0	0.03	0.05
19.0	0.00	0.00
20.0	0.00	0.00



4.0 Damage Assessment

The edge of pavement for South Service Road is approximately 6 meters from the proposed excavation. Based on the above calculations, this pavement is expected to experience between 4 and 9mm of settlement

These values are below the 'Review Level' of 10mm as described in WSP Technical Memo 'Non-Standard Special Provision, Settler

 Hatch Mott MacDonald	<h2 style="margin: 0;">CALCULATION SHEET</h2>	SHEET NO.	
DESCRIPTION High Complexity Tunnelling RAQ EPB Pressures	PROJECT NO. 358397	MADE BY CC DATE 9-Jun-2015	CHECKED BY DATE

1.0 Calculation Purpose

The purpose of this calculation sheet is to determine operating pressures for a Earth Pressure Balance TBM for the construction of the trunk sewer and watermain.

2.0 Engineering Assumptions and Tunnel Parameters

Description	Symbol	Value	Units	Notes
Average Soil Unit Weight	γ_{soil}	21.3	kN/m ³	
Average Friction Angle	ϕ'	33.3	°	
Water Unit Weight	γ_{water}	9.81	kN/m ³	
Depth to Water Table	d_{water}	4.0	m	
Depth to watermain springline	d_{wm}	9.5	m	
Depth to sewer springline	d_{swr}	12.6	m	
Watermain Diameter	ϕ_{wm}	1.05	m	
Sewer Diameter	ϕ_{swr}	3.2	m	

3.0 Earth Pressure Calculation at Tunnel Face

Watermain

Depth of water table with respect to springline:

$$d_{\text{water/wm}} = 5.5 \text{ m}$$

Stress due to soil:

$$\sigma = d_{\text{wm}} \times \gamma_{\text{soil}} = 202.7 \text{ kPa}$$

Hydrostatic Pressure:

$$U_0 = d_{\text{water/wm}} \times \gamma_{\text{water}} = 54.0 \text{ kPa}$$

Effective Stress:

$$\sigma'_v = \sigma - U_0 = 148.7 \text{ kPa}$$

Active Pressure:

$$K_A = (1 - \sin \phi') / (1 + \sin \phi') = 0.29$$

Trunk Sewer

Depth of water table with respect to springline:

$$d_{\text{water/wm}} = 8.6 \text{ m}$$

Stress due to soil:

$$\sigma = d_{\text{wm}} \times \gamma_{\text{soil}} = 268.8 \text{ kPa}$$

Hydrostatic Pressure:

$$U_0 = d_{\text{water/wm}} \times \gamma_{\text{water}} = 84.4 \text{ kPa}$$

Effective Stress:

$$\sigma'_v = \sigma - U_0 = 184.4 \text{ kPa}$$

Active Pressure:

$$K_A = (1 - \sin \phi') / (1 + \sin \phi') = 0.29$$

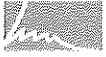
4.0 Stability Number

$$P_{\text{min, full chamber}} = (\text{Tunnel Diameter}/2) \times \gamma = 11.2 \text{ kPa}$$

The minium pressure at tunnel springline to maintain a full excavation chamber is 11.2 kPa. The minimum calculated press pressure.

$$P_{\text{min, full chamber}} = (\text{Tunnel Diameter}/2) \times \gamma = 34.1 \text{ kPa}$$

The minium pressure at tunnel springline to maintain a full excavation chamber is 11.2 kPa. The minimum calculated press pressure.



Hatch Mott
MacDonald

CALCULATION SHEET

SHEET NO.

DESCRIPTION

**High Complexity Tunnelling RAQ
EPB Pressures**

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4.0 Minimum Tunnel Face Pressures

The minimum tunnel face pressure for granular soils (kPa) is calculated from the hydrostatic pressure (kPa), the effective vertical stress (kPa) and the active pressure (unitless). Target pressure is assumed to be 0.5 Bar above minimum pressure.

$$P_{i,min,gran} = U_0 + (\sigma'_v \times K_A)$$

Watermain

$$P_{i,min,gran} = 97.2 \text{ kPa}$$

$$P_{target} = 147.2 \text{ kPa}$$

Trunk Sewer

$$P_{i,min,gran} = 138.0 \text{ kPa}$$

$$P_{target} = 188.0 \text{ kPa}$$