



# **GEOTECHNICAL BASELINE REPORT VICTORIA STREET UTILITIES RELOCATION CITY OF KITCHENER, ON**

**Prepared for:  
MMM GROUP LIMITED**

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## **1.0 INTRODUCTION**

### **1.1 Purpose of Report**

This is the Geotechnical Baseline Report (GBR) for the Victoria Street Utilities Relocation project in the City of Kitchener, Ontario. This GBR was prepared by SPL Consultants Limited (SPL) with input from MMM Group Limited.

The GBR has been developed on the basis of the Geotechnical Report for the project, and provides baseline surface conditions and geotechnical engineering parameters of the soil and groundwater for use during bidding, and forms the basis on which to judge whether or not the conditions encountered during construction are materially different from those anticipated at the time of bidding. Methods of testing and the interpretation of the test results are described as background to the presentation of baseline geotechnical and geo-environmental parameters. Both GBR and the geotechnical report form part of the Contract Documents. The GBR is to be read in conjunction with the Contract Documents.

The GBR includes figures and tables that summarize the data and present baseline conditions and parameters. The chainages (stationing) described in the GBR are approximate. The Contractor shall refer to the Contract Documents for the exact stationing and coordinates.

The GBR is intended to:

- Identify important geotechnical considerations that will need to be addressed during bid preparation and construction of the project;
- Assist in evaluating the requirements for excavating and supporting the ground, controlling groundwater and ground movement;
- Provide a geotechnical baseline for bidding the work;
- Assist the Owner in reviewing the Contractor's submittals, and
- Establish a geotechnical baseline that will be used to resolve potential disagreements, disputes, and claims related to subsurface conditions.

Statements regarding the baseline conditions are presented in this GBR. These are intended to summarize the baseline conditions.

The designs, means, methods, sequences, timing and workmanship required to construct the project in accordance with the proponent's design may influence the behaviour of the subsurface materials during construction. As such, in its submission, the proponent is required to address specific elements of their design and to discuss how their design and construction approach may address to minimize and eliminate any construction impacts on the prevailing subsurface conditions.

The provision of baseline conditions in the Contract is not a warranty that the baseline conditions will be encountered; however the baseline conditions represent a contractual basis for the Owner and the

Contractor will agree to use. The Contractor will rely on this report for bidding and construction planning purposes related to anticipated ground, subsoils and groundwater conditions and behaviour. The Contractor is to plan construction and select equipment and construction methods to address the expected baseline conditions identified in this report.

The GBR summarizes the soil and groundwater conditions expected to be encountered in the surface and subsurface excavations. While the actual conditions encountered in the field are expected to be within the given ranges, the distribution of geologic conditions encountered will likely vary from those presented in this GBR.

## **1.2 Project Background and Description**

The Victoria Street Bridge is located at the interchange of Hwy 85 and Victoria Street North in the City of Kitchener. The bridge will be reconstructed and this reconstruction of the bridge involves some widening of the approach east and west embankments as well as proposed new ramps to accommodate the new bridge length and widened structure. As a result of this reconstruction the existing utilities present in the area of Victoria Street and Hwy 85 are required to be relocated. A wide range of utilities exist within the project area as shown in Figures 1 to 11 comprising of stormwater sewers, watermains, sanitary gravity sewers, sanitary forcemains, gasmains etc. of various sizes ranging from 150mm to over 750mm in diameter and will be relocated. In addition, other utilities such as underground Bell, hydro and Roger cables are also present in the area.

A West sanitary sewer of dia. 300mm to 450mm from Sta. 0+000 to Sta. 0+0+235 will be installed by open cut. From Sta. 0+235 to the entry shaft at Sta. 0+275 the PVC sewer will be installed inside a reinforced concrete primary liner and grouted in place. The section of the PVC sewer primary liner between the two shafts will be installed by microtunnelling and the sewer grouted in place. A short section of this primary liner and sewer lies to the east of the exit shaft and will be installed by open cut inside a primary liner and grouted in place to connect with MH 7A. The 750mm sewer down stream of MH 7A to the downstream connection at existing MH 4 will be installed by open cut. The existing 750mm diameter sanitary sewer and the 500mm sanitary forcemain is located on the east side and parallel to Hwy. 85. This sewer and forcemain will be relocated at the same size.

A 200mm dia. gas main pipe from Sta. 0+020 to Sta. 0+130 will be installed by open cut technique and will be about 1.4m± to 11.5m± below ground surface. Between Sta. 0+130 and Sta. 0+170 twin 200mm dia. HDPE gasmains will be installed inside a welded steel casing installed by open cut. Between Sta. 0+170 and 0+270 the steel casing will be installed by microtunnelling and the gasmains secured in a common spacer. From Sta. 0+270 to Sta. 0+280 the steel casing will be installed by open cut with the gasmains secured in a common spacer. The balance of the gasmains length on the east side of Hwy. 85 to the connection point will be installed in open cut.

A range of watermain sizes and materials (PVC and CPP) will be installed on the west and east of Hwy. 85 using open cut. A 600mm dia. CPP watermain from Sta. 0+035 to Sta. 0+120 and from Sta. 0+260 to Sta. 0+340 will be installed using open cut technique, including inside a steel casing primary liner and the

Watermain at these locations will be 2.2 to 8.3m below ground surface, corresponding to the Elev. 314.3 to 321.0m. At Sta. 0+160 to Sta. 0+260 the 600mm watermain will be installed in a 1200mm dia. RC

microtunnel pipe with 900mm dia. steel casing will cross Hwy 85 and will be installed using microtunnelling. The watermain at this location will be 8.6 to 12.5m deep corresponding to Elev. 310m±.

Other utilities including storm and sanitary sewers and sanitary CPP and PVC forcemain will be installed using an open cut technique. These utilities will be about 2.8 to 8.1m below ground surface, corresponding to the Elev. 314.8 to 321m. Corresponding maintenance holes and chambers will be installed and constructed as part of the new system.

## **2.0 SOURCES OF INFORMATION**

The documents listed in this section have been used in developing this GBR; however, these documents are not included in this document. Several reports listed below are interpretive reports prepared from field and laboratory investigations and testing for this or other related projects. Also, other publications that were used in developing this GBR are referenced here for information purposes.

The Subsurface data reports referenced below were used as the primary source of data for development of this GBR. The subsurface materials as characterized at specific sample locations within the boreholes can be relied upon and the Contractor is expected to review the specific subsurface data available in the geotechnical factual data reports. However, the interpretation of engineering properties and parameters for the deposits and the stratigraphy as interpreted between samples provided in this GBR are the baselines for this project. In the event of conflict between the geotechnical reports and the GBR, the GBR shall be given precedence for the purpose of tendering and evaluating claims related to ground conditions.

### **2.1 Subsurface Data Reports**

Subsurface data gathered from multiple sources have been referenced in development of this report. The principal sources of data are these geotechnical factual data reports and geotechnical investigations:

- SPL Consultants Limited, Report on Geotechnical Investigation, Victoria Street Utilities Relocation, Town of Kitchener, Ontario, Project No. 10001862, dated October 30, 2015.
- Thurber Engineering Ltd., Draft Foundation Investigation and Design report Victoria Street Under pass Replacement Highway 85, Regional Municipality of Waterloo, dated March 11, 2015.
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### **2.2 Geological and Other Publications**

The publications referenced in this document are listed below for information purposes only:

- ASCE (2007). Geotechnical Baselines for Construction: Suggested Guidelines. The Technical Committee on Geotechnical Reports of the Underground Technology Research Council, R.J. Essex, Chairman, ASCE, Reston, VA.

- Canadian Geotechnical Society (2006). Canadian Foundation Engineering Manual, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publishers Ltd. Richmond, British Columbia.
- Chapman, L.J. and Putnam, D.F (1984). The Physiography of Southern Ontario, Ontario Geological Survey, Volume 2.
- Thewes, M. Burger, W., 2005: Clogging of MTBM drives in Clay – identification and mitigation of risks. Proceedings ITA-AITES World Tunnel Congress, Istanbul, Turkey, pp. 737-742.
- Bedrock Topography of the Guelph Area, 1:50,000 (P. Karrow et al., 1979).

### 3.0 PROJECT GEOLOGIC AND ENVIRONMENTAL CONDITIONS

#### 3.1 Physiographic and Geologic Setting

The area is part of the physiographic region known as the Waterloo Sand Hills or Waterloo Moraine (Chapman & Putnam, 1984). The regional geology of the Victoria Street and Highway 85 area in Kitchener, Ontario is a result of the interaction of several glacial lobes. The oldest deposits (mapped in the area to the northeast and another to the west of the intersection of Victoria Street and Highway 85) is a sandy silt till to silty sand textured till. This till was reworked and overlain by ice contact stratified sand and gravel, with minor silt, clay and till that were deposited as the ice lobes receded and re-advanced. There are also glaciofluvial major river and delta deposits of sand located to the south of the Victoria Street and Highway 85 intersection, which were laid down as the glacial ice melted and retreated.

The bedrock subsurface is relatively flat, with an average relief of about 30 m (Karrow 1976a, 1976b; Karrow et al. 1979; Miller et al. 1979). In places it is incised by pre-glacial valleys that are filled with glacial drift. The bedrock surface in North Dumfries Township is flat to gently rolling and its elevation varies from more than 274 m above sea level near Orrs Lake to 193 m above sea level north of Ayr (Karrow 1963).

#### 3.2 Summary of Geotechnical Exploration and Testing Programs

The subsurface investigations for this project were completed in sections as follows:

The field investigation consisted of putting down ten (10) boreholes (BH15-1 to BH15-10) to depths ranging from 6.7 to 17.4m below existing ground surface at the approximate locations shown on the attached **Drawing No. 3.2.1**. Boreholes BH15-1 through BH15-3, BH15-5 and BH15-8 through BH15-10 were drilled at the four corners of the interchange of Highway 85 and Victoria Street Bridge for the construction of proposed utilities to be installed by means of an open cut technique. Boreholes BH15-2 through BH15-5 and BH15-7 through BH15-9 were drilled on the private properties namely Kitchener Glass and Factory shoe which are located on the northeast and northwest corners of the interchange. Boreholes BH15-1 and BH15-10 were moved and drilled at lower elevations because of the access issues with the private property owners at these locations. Boreholes BH15-4, BH15-6 and BH15-7 were drilled for the trenchless installation of the utilities beneath Hwy 85.



The locations of boreholes are shown on Figures 3.2.1 through 3.2.11.

Standard Penetration Tests (ASTM D 1586) were carried out in overburden deposits at regular intervals of depth.

The laboratory soil testing program consisted of the measurement of the natural water content of all samples, grain size analyses on twenty six (26) selected samples and consistency (Atterberg) limits for eighteen (18) plastic soil samples.

Five (5) soil samples were analysed for metal and inorganic parameters as set out in Ontario Regulation 153 of the Environmental Protection Act. There may be further testing required by receivers during excavation. For the purpose of determining the requirements for off-site disposal of excess soil and the fact that the location of the ultimate receptor of excess soil is unknown at this time, the MOE Table 1 & 2 Standards for residential/parkland/institutional (RPI) and industrial/commercial/community (ICC) property uses were selected to compare the analytical results.

### **3.3 Topographic Setting and Land Use**

Along the proposed route of the utilities to be located on the four corners of the interchange of Hwy 85 and Victoria Street, the topography of the site slopes down towards southeast corner of the interchange and elevations ranging from 324.0 to 322.5m.

The topography of the site slopes down on Hwy 85 from east and west sides of the interchange and elevations ranging from 322.5m± to 318.6m±.

The land around the four corners of the interchange is occupied by private properties and there is Hwy 85 at the middle portion of the site.

### **3.4 Seismicity**

The earthquake ground motion parameters for the project site are estimated using the Natural Resources Canada Webpage:

<http://earthquakescanada.nrcan.gc.ca/hazard-alea/interpolate/index-eng.php>

Which provides interpolation of the 2010 National Building Code of Canada Seismic Hazard values based on the latitude and longitude of the site. Estimated latitude and longitude of the site used for this purpose are 43.4624° North and 80.4696° West. The hazard level for Site Class C corresponding to an earthquake having 2 percent probability of exceedence in a 50 years (return period of 2475 years) are given by damped reference spectral response acceleration values of  $S_a(0.2) = 0.160g$ ,  $S_a(0.5) = 0.096g$ ,  $S_a(1.0) = 0.058g$  and  $S_a(2.0) = 0.019g$  and the reference peak ground acceleration (PGA) = 0.057g (g = acceleration due to gravity).

For the manhole structures founded on compact/stiff to very stiff soils and proposed utilities founded on compact sandy and very stiff cohesive soils, the Site Classification can be taken as “Class D” for seismic site response.

### **3.5 Climatic Conditions**

The climatic information in the project area is obtained from Environment Canada Webpage:

<http://climate.weatheroffice.gc.ca/climateData/canada.html>

The average rainfall, snowfall and precipitation value in each month from 1971 to 2000 are summarized in Figure 3.5 of this GBR. The highest rainfall and precipitation of 80mm per month occurs in August. The highest snowfall of 43.5 cm per month occurs in January.

### **4.0 PREVIOUS LOCAL TUNNELLING EXPERIENCE**

The following sections provide a summary of the subsurface conditions encountered during a similar microtunnelling project. These summaries are for information only and are intended to aid the reader in gaining a general understanding of how the ground might behave during excavation.

#### **4.1 CN Rail crossings for Storm and Sanitary Sewer System (Bronte Street South Town of Milton)**

Installation of two no. microtunnels using reinforced concrete jacking pipe. The first tunnel, with a 1500mm id acted as the storm sewer in a single pass installation system. The first tunnel was 78m long constructed in silty clay with occasional cobbles and boulders. The drive passed under a CN Rail Line with 3.4m cover. The closed face pressurized slurry excavation system tunnelled under the rail line without any problems and the tunnel was completed in 8 days (9 hour shift). Average advance rate of 10m/shift.

The second tunnel, with a 600mm id, acted as a liner for a 250mm sanitary sewer. This second microtunnel was 92m long and again constructed in silty clay with occasional cobbles and boulders. It passed under the same CN rail line with 4.7m cover. The jacking pipe was installed to an exact invert level and grade so that the sanitary pipe, when installed, would tie into an existing downstream pipe. The tunnel was complete in 7 days with an average advance rate of 13m/day. A 250mm id PVC pipe was installed inside the jacking pipe using a specifically designed pipe support on rollers. The entire 92m of 250mm id PVC pipe was installed into the tunnel by hand from the launch shaft.

Both microtunnels were installed under the rail line with no settlement recorded at any stage of construction. The entire project from the commencement of mobilization to the end of demobilization took 7 weeks with no problems encountered at any stage.

### **5.0 GROUND CHARACTERIZATION**

#### **5.1 General**

The subsurface soil and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil samples, are provided in the geotechnical report (see **Section 3.1**).

The interpreted stratigraphic profiles shown on Figures 3.2.1 through 3.2.11 along the proposed sanitary sewer, watermain and gasmain primary liners alignment are the baseline stratigraphy; these figures are a simplification of the subsurface conditions encountered at the boreholes locations. Variations in the stratigraphic boundaries between boreholes will exist and should be expected. Subsurface conditions will vary between and beyond the borehole locations. When precise determination of deposit boundaries and subsurface water levels are critical for the safety and stability of the works, the Contractor shall verify these boundaries and water levels using appropriate methods during construction.

This section of the GBR provides baseline geotechnical engineering parameters to be used for design of temporary works and for selection of equipment and construction methods. Within this section, baseline values are provided consistent with 10<sup>th</sup>, 50<sup>th</sup>, and 90<sup>th</sup> percentiles. The baseline 10<sup>th</sup>, 50<sup>th</sup>, and 90<sup>th</sup> percentile values are provided as means of quantitatively describing the statistical distribution of the parameter values. In some cases the percentiles are based directly on statistical evaluation of available data and in other cases these values are supplemented by judgement based on local and regional experience with these soil types. While the 50<sup>th</sup> percentile value can be used for soil design purposes, the range represented by 10<sup>th</sup> to 90<sup>th</sup> percentiles must also be considered as variability in physical properties is intrinsic to the nature of earth materials and is to be taken into account for estimating qualities, selection of equipment, and selection of construction means and method.

## 5.2 Soil Characterization

The boreholes revealed the presence of a variety of soil types ranging in texture from pavement structure, fill material, cohesive silty clay and silty clay till to non-cohesive sand, silt, sandy silt and sand and gravel.

No bedrock was encountered during drilling and the boreholes were terminated within overburden.

### 5.2.1 Pavement Structure

All boreholes except BH15-1 were drilled on the roadway and parking lots of private properties encountered 40 to 250mm of asphalt overlying 200mm to 450mm of granular base and sub-base.

The baseline thickness of the pavement structure is summarized in Table 5.2.1.

**Table 5.2.1 Baseline Thickness of Pavement Structure**

Pavement Structure (mm)			Thickness of Base and Subbase Combined (mm)		
10 <sup>th</sup> Percentile	50 <sup>th</sup> Percentile	90 <sup>th</sup> Percentile	10 <sup>th</sup> Percentile	50 <sup>th</sup> Percentile	90 <sup>th</sup> Percentile
40	225	350	280	300	370

### 5.2.2 Topsoil and Fill

BH15-1 was drilled on grass and encountered 150mm of surficial topsoil layer.

A heterogeneous fill consisting of silty sand and sand was found. The fill extends to the depths ranging from 2.0 to 5.3m below ground surface. Trace to some organics, asphalt pieces and brick fragments were also observed in fill material. The thickness of the encountered fill was found to vary from 1.0 to 4.9m.

The baseline thickness of the fill material is summarized in Table 5.2.2.

**Table 5.2.2 Baseline Thickness of Fill Material**

Thickness of Fill (m)		
10 <sup>th</sup> Percentile	50 <sup>th</sup> Percentile	90 <sup>th</sup> Percentile
1.3	2.7	4.0

The SPT carried out within the fill material recorded 'N' values ranging from 3 to more than 50 blows per 300mm penetration indicating a very loose to very dense state of fill material. High SPT 'N' values infer that boulders/cobbles/buried concrete pieces exist within the fill material.

The baseline water content values measured on the samples of the fill material are 8%, 10% and 18% for the 10%, 50% and 90% percentiles, respectively.

### 5.2.3 Cohesionless Deposits of Sand/Silty Sand/Sand and Gravel and Silt/Sandy Silt

#### ***Sand/Silty Sand/Sand and Gravel***

Coarse-grained deposits of sand to silty sand and sand and gravel were encountered below the fill in Boreholes BH15-2 through BH15-5 and B15-7 through BH15-10. SPT 'N' values in these coarse-grained cohesionless deposits were in the range of 5 to 78 blows per 0.3m penetration, corresponding to a loose to very dense state.

The baseline water content values measured on the samples of the sand, silty sand and sand and gravel are 6%, 17% and 20% for 10%, 50% and 90% percentiles, respectively.

Seven (7) grain size analyses of sand, silty sand and gravelly sand revealed 27% gravel, 57 to 94% sand; 4 to 36% silt and 2 to 6% clay size particles and are presented in Figure 5.2.3 (I).

#### ***Silt/Sandy Silt***

Fine-grained deposit of silt to sandy silt deposits were locally encountered in BH15-4, BH15-8 and BH15-9. SPT 'N' values in these deposits were in the range of 22 to 39 blows per 0.3m penetration corresponding to a compact to dense state.

The baseline water content values measured on the samples of the silt to sandy silt are 12%, 12% and 14% for 10%, 50% and 90% percentiles, respectively.

One grain size analysis of silt revealed 15% sand; 75% silt and 10% clay size particles as presented in Figure 5.2.3 (II).

**Table 5.2.3a: Baseline Thickness of Cohesionless Deposits**

Thickness of Cohesionless Deposits (m)		
10 <sup>th</sup> Percentile	50 <sup>th</sup> Percentile	90 <sup>th</sup> Percentile
0.7	1.8	3.1

Based on SPT N-values and local experience, the baseline geotechnical parameters including unit weight ( $\gamma$ ), effective angle of internal friction ( $\phi'$ ), elastic modulus (E), and Poisson's ratio ( $\mu$ ) for the cohesionless deposits are summarized in Table 5.2.3b.

**Table 5.2.3b: Baseline Unfactored Geotechnical Parameters for Cohesionless Soils**

	SPT N-value	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (°)	E (MPa)	$\mu$
10 <sup>th</sup> percentile	10	20	30	10	0.2
50 <sup>th</sup> percentile	18	21	32	18	0.3
90 <sup>th</sup> percentile	33	21.5	34	33	0.35

## 5.2.4 Silty Clay Till/Clayey Silt Till and Silty Clay

### *Silty Clayey Till/Clayey Silt Till*

The silty clay till/clayey silt till deposits were encountered in all boreholes at various depths. Boreholes BH15-4 through BH15-6 and BH15-9 were terminated in this deposit.

SPT 'N' values of 9 to more than 50 blows per 300mm penetration indicated the cohesive soils in a stiff to hard consistency.

The baseline water content values measured on the samples of silty clay till to clayey silt till are 10%, 18% and 33% for 10%, 50% and 90% percentiles, respectively.

Eleven (11) grain size analyses revealed 1 to 13% gravel; 11 to 33% sand; 38 to 59% silt and 16 to 27% clay size particles as presented in Figure 5.2.4 (I).

### **Silty Clay**

The silty clay was encountered in boreholes BH15-1 through BH15-4 and BH15-6 through BH15-8 below/interbedded the silty clay/clayey silt till deposits. Boreholes BH15-1 through BH15-3, BH15-7 and BH15-8 were terminated in this deposit.

The SPT 'N values 12 to more than 50 blows per 300mm penetration, indicate a stiff to hard consistency.

The baseline water content values measured on the samples of silty clay till to clayey silt till are 18%, 21% and 24% for 10%, 50% and 90% percentiles, respectively.

Seven (7) grain size analyses revealed 1% gravel, 1 to 5% sand; 38 to 61% silt and 45 to 59% clay size particles. Baseline envelopes of grain size distributions for cohesive deposits are presented in Figure 5.2.4.

The baseline thickness of the cohesive deposits is summarized in Table 5.2.4a.

**Table 5.2.4a Baseline Thickness of Cohesive Deposits**

Thickness of Cohesive Deposits (m)		
10 <sup>th</sup> Percentile	50 <sup>th</sup> Percentile	90 <sup>th</sup> Percentile
1.0	3.4	6.0

Baseline water content and Atterberg Limits for the cohesive soils are summarized Table 5.2.4b.

**Table 5.2.4b Baseline Water Contents and Atterberg Limits of Cohesive Soils**

Engineering Property	Atterberg Limits of Cohesive Soils		
	10 <sup>th</sup> Percentile	50 <sup>th</sup> Percentile	90 <sup>th</sup> Percentile
Water Content (%)	18	21	24
Plastic Limit (PL)	12	14	19
Liquid Limit (LL)	22	29	46
Plasticity Index (PI)	9	15	27

Based on SPT N-values and local experience, the baseline geotechnical parameters including unit weight ( $\gamma$ ), effective angle of internal friction ( $\phi'$ ), effective cohesion ( $c'$ ), undrained shear strength ( $s_u$ ), elastic modulus ( $E$ ), Poisson's ratio ( $\mu$ ) for the cohesive soils are provided in Table 5.2.4c.

**Table 5.2.4c Baseline Unfactored Geotechnical Parameters for Cohesive Soils**

	SPT N-value	$\gamma$ (kN/m <sup>3</sup> )	$\phi'$ (°)	$c'$ (kPa)	$s_u$ (kPa)	E (MPa)	$\mu$
10 <sup>th</sup> percentile	13	20	30	5	80	13	0.4
50 <sup>th</sup> percentile	22	21.5	32	5	130	22	0.3
90 <sup>th</sup> percentile	41	22	34	10	250	41	0.2

### 5.2.5 Boulders and Cobbles

The glacial till deposits will contain cobbles and boulders even if specific obstacles were not encountered during drilling operations. Cobbles are defined as rock fragments that cannot pass through a screen with 75mm square openings and are less than 200mm in maximum dimension. Boulders are defined as rock fragments with their maximum dimension being equal to or greater than 200mm. Removal of cobbles during shaft excavation and construction is considered part of routine construction and these materials will not be considered as obstructions for this project.

Boulders and other obstructions including but not limited to construction debris will be randomly distributed within the fill. Considering that the fill materials extend from the ground surface to relatively shallow depths, it is not considered necessary, and it is not feasible, to estimate the frequency of obstruction within the fill.

During the construction of the Kennedy Road Watermain project in Markham, the percentage of cobbles and boulders within sandy silt till was determined from a TBM rescue shaft excavation located at the Hwy 7/Kennedy Road in Markham, in which 1543 cobbles (including 254 cobbles with diameter of 75mm to 100mm and 1289 cobbles with diameter of 100 to 300mm) and 38 boulders (maximum dimension equal to or greater than 300mm) in 100 cubic metre of the glacial till were counted.

At the time when this report was prepared there was no specific experience with deep excavations along the sewer and watermain alignments where boulder occurrences have been qualified. For construction baseline purposes, the average Boulder Volume Ratio (BVR) defined as the ratio of the cumulative volume of all boulders to the total volume of ground, is recommended in Table 5.2.5.

**Table 5.2.5 Baseline BVR Data**

Parameter	Dimension	Granular Deposits	Clayey Silt to Silty Clay Till
Boulder Volume Ratio (BVR)	Max. dimension equal to or greater than 200mm	0.25%	0.25%
Boulder Maximum Size		3m <sup>3</sup>	

For baseline purposes, 99% of the boulders will not exceed 1m in maximum dimension and 1% of the boulders will range from 1m to 3m in maximum dimension.

For baseline purposes, cobbles and boulders shall be assumed to be comprised of Canadian Shield-derived igneous or metamorphic rock of 'extremely high' Cerchar abrasiveness and 'very strong to extremely strong' unconfined strength (100 MPa to 250 MPa), as defined by ISRM.

### 5.2.6 Bedrock

For baseline purposes, bedrock will not be encountered within the excavation depth for the proposed utilities to be installed.

## 5.3 Groundwater Conditions

For baseline purposes, the groundwater levels in the monitoring wells installed at the site and the expected fluctuation in groundwater table elevation is given in Table 5.3.

**Table 5.3 – Baseline Groundwater Conditions in Monitoring Wells**

BH No.	Ground Surface Elev. (m)	Soil Type at Screen Location (Depth, m)	Baseline Groundwater Table Elev. (m)	Baseline Groundwater Table Fluctuation (m)
BH15-1	318.7	Fill/Silty Clay Till/Silty Clay (1.5 – 4.6)	318.3	±1
BH15-2	323.9	Silty Clay Till (6.1 – 7.6)	318.6	±1
BH15-4	322.5	Clayey Silt Till/Silt (7.6 – 10.7)	317.8	±1
BH15-7	322.6	Sand/Sandy Silt/Silty Clay Till (6.1 – 9.1)	318.7	±1
BH15-8	322.9	Sandy Silt/Sand	318.9	±1
BH15-9	323.2	Fill/Sandy Silt/Sand and Gravel/Sand (3.1 – 9.1)	320.0	±1
BH15-10	320.5	Sand/Silty Clay Till (3.1 – 6.1)	312.9	±1



## 6.0 DESIGN CONSIDERATION FOR TEMPORARY WORKS

This section addresses the design consideration for temporary works which will be the responsibility of the Contractor. The Contract also specifies that the Contractor locate and protect all existing underground and above ground services and facilities.

### 6.1 Open-Cut and Shaft Support Design Consideration

All temporary excavation must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA) for Construction Projects. The soil types, as defined in the OHSA, for the overburden soils and conditions at the shaft locations at the site are outlined below:

- Fill – Type 3 Soils above groundwater table and Type 4 Soil below groundwater and in perched water;
- Cohesive deposits – Type 2 Soils (very stiff to hard), Type 3 soils (firm to stiff)
- Cohesionless deposits – Type 4 soil below groundwater

Unsupported excavations would be temporarily stable at a slope of 1.5H:1V in the fill, stiff silty clay and cohesionless soils above ground water table, and a slope of 1H:1V in the very stiff to hard silty clay/ clayey silt (till). Below the water table, unsupported excavations in the cohesionless soils cannot safely proceed until such time as a lowering of the groundwater table to a minimum depth of 1.0m below the base of the excavation has been achieved.

Due to the typically limited space at the proposed shaft and open cut locations and the presence of existing underground utilities/services, it is anticipated that temporary excavations will generally have to be made in vertical cut and as such temporary support systems will be required at each shaft location.

Contiguous caisson walls or continuous interlocked sheet piles can be considered to avoid the need of dewatering for the construction of shafts and for the installation of all primary liners.

The design of all temporary support system for open-cut and shaft construction will be the responsibility of the Contractor.

Where trench depths exceed 6m and in Type 4 Soils of any trench depth, Engineered Support Systems specifically for those project locations are required under the OHSA as defined in the Regulation.

The design and construction considerations for shaft excavation in this report generally relate to the use of contiguous caisson walls or continuous interlocked steel sheet piles can be considered to avoid the need of dewatering for the construction of shafts and for the installation of primary liners. The contiguous caisson walls should be supported by tie-back anchors or struts. A circular shaft, however, is more structurally efficient and could be supported with walers. All shoring designs should be in accordance with the 4<sup>th</sup> Edition of the Canadian Foundation Engineering Manual and must be reviewed by this office or the Owner designated geotechnical engineer. Allowable bond stress for pressure-grouted tie-back soil anchors installed within very stiff to hard /compact to design dense soil can be

taken as 80 kPa (minimum bond length is 4m). An allowable (SLS) bearing capacity of 300 kPa can be used for caissons founded on very stiff to hard/compact to dense soils.

If shoring is to be carried out over the winter months or if the excavation is to be left open for any period during below zero temperature, shored walls must be protected against frost penetration by means of insulation or heated hoarding.

The design of the temporary support systems at each shaft/manhole i.e. at each project specific location shall take into account:

- Groundwater level;
- Compactness condition/consistency of the soils;

It should not be construed that the Contractor is limited to the retaining systems discussed in this report. The design and installation considerations presented are for baseline purposes only. The Contract Documents address the specific requirements of any limitations on selection of excavation support systems.

## **6.2 Tunnel Support Design Consideration**

For microtunnelling sections, the microtunnel boring machine (MTBM) shall be capable to pressurize the face to prevent flowing ground conditions at the face. Slurry-type MTBM is recommended for this project. The MTBM head must be designed to accommodate steel casing and occasional cobbles and boulders. The reinforced concrete jacking pipe (RCP) must be designed for jacking pressure as well as overburden pressure. The RCP segments must be precast by a manufacturer with previous experience in jacking pipes. Close quality control is needed in the production of this product.

The selection of MTBM, design of the RC microtunnel pipe, design of the steel casings and drilling mud shall consider the following:

- Groundwater level;
- Compactness condition/consistency of the soils;
- Grain size distribution of soils;
- Boulders and cobbles.

## **7.0 ANTICIPATED GROUND BEHAVIOUR RELATED TO CONSTRUCTION**

### **7.1 Open-Cut and Shaft Excavation and Support**

The majority of the soil deposits encountered at the open cut and shaft locations are a combination of both cohesive and cohesionless soils;

Cohesionless deposits will be saturated below the groundwater table and will behave as “flowing ground” if left unsupported or without some advance stabilizing measure. “Active” dewatering using closely spaced eductors/well points are required at the locations where the saturated cohesionless deposits are encountered.

Where permissible under the OHSA and where its use is considered to be a safe alternative for shoring and bracing, contractors may elect to utilize trench boxes for temporary trench wall support for trenches less than 6m deep in Type 2 Soils (cohesive soils) only. While the use of trench boxes is an effective and economical trench-support method, its use can cause increased loss of ground relative to properly braced shoring, especially when working close to granular base courses below existing pavements or along existing utility trenches backfilled with granular materials. Trench boxes also reduce the contractor’s ability to compact backfill materials placed between the trench wall and the outer trench box shell, thereby increasing the likelihood of post-construction settlements along the trench walls.

When trench boxes are used along existing roadways, settlements frequently occur along the trench wall, which may manifest months after completion of backfilling. In such cases, following the backfilling of the trench, road reconstruction should include a provision for saw-cutting the asphalt at least 1.5m back from the trench walls, recompacting the upper trench backfill, and then repaving.

It is expected that Contiguous caisson walls or Continuous interlocked steel sheet piles can be considered to avoid the need of dewatering as specified for the construction of shafts and for the installation of all primary liners. The contiguous caisson wall or sheet pile wall must be installed at least one meter below the excavation level and use mass concrete tremie plugs for the shaft installed within saturated sandy soils.

## **7.2 Tunnel Excavation and Support**

A West sanitary sewer of dia. 300mm to 450mm from Sta. 0+000 to Sta. 0+0+235 will be installed by open cut. From Sta. 0+235 to the entry shaft at Sta. 0+275 the PVC sewer will be installed inside a reinforced concrete primary liner and grouted in place. The section of the PVC sewer primary liner between the two shafts will be installed by microtunnelling and the sewer grouted in place. A short section of this primary liner and sewer lies to the east of the exit shaft and will be installed by open cut inside a primary liner and grouted in place to connect with MH 7A. The 750mm sewer down stream of MH 7A to the downstream connection at existing MH 4 will be installed by open cut. The existing 750mm diameter sanitary sewer is located on the east side and parallel to Hwy. 85. This sewer will be relocated at the same size.

A 200mm dia. gas main pipe from Sta. 0+020 to Sta. 0+130 will be installed by open cut technique and will be about 1.4m± to 11.5m± below ground surface. Between Sta. 0+130 and Sta. 0+170 twin 200mm dia. HDPE gas mains will be installed inside a welded steel casing installed by open cut. Between Sta. 0+170 and 0+270 the steel casing will be installed by microtunnelling and the gas mains secured in a common spacer. From Sta. 0+270 to Sta. 0+280 the steel casing will be installed by open cut with the gas mains secured in a common spacer. The balance of the gas mains length on the east side of Hwy. 85 to the connection point will be installed in open cut.

A range of watermain sizes and materials (PVC and CPP) will be installed on the west and east of Hwy. 85 using open cut. A 600mm dia. CPP watermain from Sta. 0+035 to Sta. 0+120 and from Sta. 0+260 to Sta. 0+340 will be installed using open cut technique, including inside a steel casing primary liner and the watermain at these locations will be 2.2 to 8.3m below ground surface, corresponding to the Elev. 314.3 to 321.0m. At Sta. 0+160 to Sta. 0+260 the 600mm watermain will be installed in a 1200mm dia. RC microtunnel pipe with 900mm dia. steel casing will cross Hwy 85 and will be installed using microtunnelling. The watermain at this location will be 8.6 to 12.5m deep corresponding to Elev. 310m±. The stabilization of the granular soils by dewatering could possibly be achieved by eductors/well points installed along the tunnel alignment. This approach, however, is not recommended due to the settlement risks associated with dewatering at this site. Slurry-type MTBM is recommended to avoid construction dewatering.

Contiguous caisson walls or continuous interlocked sheet piles can be considered to avoid the need of dewatering for the construction of shafts and for the installation of all primary liners.

### 7.2.1 Tunnel Ground Behaviour

The behaviour of cohesionless (non-cohesive) soils (sand) for the purpose of tunnelling can be categorized as “fast ravelling” to “flowing” and potentially “bouldery” ground in accordance with the behaviouristic ground classification system established by Terzaghi in 1950. Tunneling in these soil types will require immediate and full support at the full perimeter and the face of the tunnel.

The silty clay and clayey silt till for the purpose of tunnelling can be categorized as “slowly ravelling”, “hard” and potentially “bouldery” ground in accordance with the behaviouristic ground classification system established by Terzaghi in 1950. The initially “slowly ravelling” ground will change into “fast ravelling” ground where, less cohesive or fissured zones are encountered, requiring immediate and full support at the crown, the perimeter and the face.

### 7.2.2 Tunnel Excavation

The alignment of the tunnel is defined by the Contract.

At the micro-tunnelling sections, it is expected that slurry-type MTBM will be used to excavate saturated cohesionless deposit of sand, interbedded locally with cohesive soils of clayey silt (till). Reinforced concrete jacking pipes (RCP) and welded steel casing are pushed into the bore mined remotely by the MTBM fixed to the lead pipe segment.

Microtunneling is a method whereby reinforced concrete microtunnel pipe (RCP) is pushed into a bore mined remotely using a microtunnel boring machine head fixed to the lead pipe segment. Operation of the MTBM (microtunnel boring machine) is done from the launching shaft. Spoils removal, using the slurry shield MTBM method is performed by mixing the excavated soil in the front chamber of the MTBM head to the consistency of a slurry with conditioning using bentonite, water and other soil conditioning agents and then pumping this slurry back to the manhole where it is de-sanded and thickened. The rate of slurry removal from the chamber is carefully controlled and matched to the advance rate of pipe jacking such that the predicted lateral earth pressures are balanced with the slurry pressure. The MTBM head must be designed to accommodate occasional cobbles and boulders. The MTBM must also be fitted with appropriate high pressure water jets to deal with the cohesive silty clay

and appreciate separation plant to remove fine clay and silt sized particles from the slurry. Based on Thewes et. al. (2014), the silty clay till is classified as low to medium potential for clogging.

A large laydown area is needed for the subsoils separation support plant. The RCP segments must be precast by a manufacturer with previous experience in microtunnel pipe. Close quality control is needed in the production of this product. Driving length typically range from 75 to 200m. Additional interjack stations may be required for driving length greater than 75m. Bentonite and/or polymer lubrication of the annular spaces will also be needed to reduce frictional resistance to jacking. The determination of maximum drive length should be made by the specialist pipe jacking contractor.

## 8.0 GROUND MOVEMENTS AND GEOTECHNICAL INSTRUMENTATION

The process of tunneling and deep excavation relieves the existing stresses in the ground around the excavation and allows convergence of the ground mass prior to installation of initial support. This will lead to ground displacement around deep excavation and tunnels. Controlling ground displacement is essential to avoid damage to existing facilities (e.g., structures, buildings, existing pipelines and other underground utilities) above, adjacent to and crossing the alignments of sewer, watermain and gasmain.

### 8.1 Ground Movements

#### 8.1.1 Settlement due to Tunnelling

With good workmanship, loss of ground and soil relaxation can be minimized. The theoretical ground settlement induced by tunnelling can be estimated using a semi-theoretical method originally proposed by Peck (1969). In the method, the ground settlement induced by tunnelling is described by a Gaussian distribution curve as follows

$$S_v = S_{\max} \exp\left(\frac{-x^2}{2i^2}\right)$$

where

$S_v$  is the vertical settlement.

$S_{\max}$  is the maximum ground settlement on the tunnel center line and

$$S_{\max} = \frac{0.313V_L D^2}{Kz_o}$$

$V_L$  is the ground loss (ratio of ground loss volume/tunnel volume per meter length).

$D$  is the diameter of tunnel.

K is the trough width parameter and taken as 0.5 and 0.25 for tunneling in clay and sands or gravels respectively. In this analysis, K of 0.5 is used for tunneling in clayey silt to clayey silt (till); K of 0.375 is used for tunneling in silty sand till to sandy silt till; K of 0.25 is used for tunneling in silt to sandy gravel.

$Z_o$  is the depth of the tunnel axis.

X is the horizontal distance from the tunnel center line.

i is the horizontal distance from the tunnel center line to the point of inflexion on the settlement trough

$$i = Kz_o$$

Assuming that the volume loss due to tunnelling is limited to 1% to 2% (which represents good practises) and the outside diameter of jacking pipe is 1.5m, the maximum ground settlement induced by tunneling is estimated to be 10 mm above the center line of the tunnel. . The ground settlement at a lateral offset distance of 10 m from the tunnel center line is expected to be less than 1 mm. This assumes good slurry control and maintenance of appropriate slurry processes and jacking advance rates. Table 8.1.1 shows the estimated ground settlements along the tunnel alignment for a range of volume losses.

**Table 8.1.1 Estimated Ground Settlements along Tunnel Alignment for 600mm Ø Sanitary Sewer, 200mm Ø Gasmain and 600mm Ø Watermain**

BH No.	Soil Type at Tunnel Axis	Tunnel Axis Depth (m)	Theoretical Settlement at Tunnel Center Line (mm)		Settlement at 10m from Tunnel Center Line (mm)	
			$V_L = 1\%$	$V_L = 2\%$	$V_L = 1\%$	$V_L = 2\%$
BH13-4	Silty Clay Till	6.5	<1	<1	<1	<1
BH13-6	Silty Clay Till	2.5	<1	2	<1	<1
BH13-7	Sand	7	<1	1	<1	<1

As the nearby buildings are located more than 10 m away from the tunnel center line, the influence of tunnelling on the existing building can be deemed to be insignificant. The minimum clear distance between the Shoe Factory, Kitchener Glass and CN Railway Bridge and tunnel is about 10, 20m and 35m respectively. The ground settlement due to tunnelling at this separation is negligible. However, other construction factors, such as shaft sinking and vibrations will likely pose greater risk to building damage or perceived damage. Therefore, recommendations for pre-construction condition surveys of nearby buildings (within 50m of the trenchless excavation) and bridge are given in **Sections 8.2** and **for**

settlement monitoring of the existing nearby buildings, bridge, utilities and railway during tunnelling are given in **Section 8.3**.

### 8.1.2 Settlement due to Shaft Excavation

Excavation for shafts will result in ground displacement adjacent to the excavation with maximum values occurring adjacent to or near the excavation perimeter, reducing with increasing distance from the excavation edge. Based on experience with other deep excavation in the Toronto area, maximum settlements at or near the excavation edge of deep excavation with properly engineered and constructed support systems are typically less than a value equal to 0.2% times the depth of excavation (i.e.  $\delta_{vmax} = 0.002H$ ) with maximum lateral displacement of an equal magnitude. Ground displacements equal to these values were used to assess the effects of ground displacements on nearby facilities. The assumed width of the zone of influence for the settlements extends horizontally from the excavation outward to a distance equal to twice the depth of excavation (2H)

The anticipated maximum depth of the shaft excavation is 13.5m. Thus the maximum vertical/lateral displacement next to the shaft is expected to be 27mm and the maximum zone of influence of the shaft excavation is about 27m. Pre-construction condition surveys and settlement monitoring of the existing structures during shaft excavation are recommended in **Sections 8.2 and 8.3**.

## 8.2 Pre-Construction Surveys

Preconstruction condition surveys of existing buildings, CN Bridge, structures, utilities, roadway surfaces, curbs, light standards, etc. should be carried out as required by the project specification. During construction, the monitoring programs must include checks on these items.

## 8.3 Ground Surface and Structural Settlement Monitoring

In order to check that ground and structure/utility settlements are limited to acceptable values, the movements of the structures/utilities near the construction site and the roadway along the tunnel should be monitored during the entire tunnelling operation as required by the project specification. This might consist of a series of settlement points installed on the structures/utilities and on the road surface, and settlement rods grouted into the soil below the frost line.

The monitoring frequency should follow the project specification.

## 9.0 MANAGEMENT OF SOIL, GROUNDWATER AND SURFACE GASES

### 9.1 Management of Excess material

#### 9.1.1 Cut and Cover Excavation

Based on the environmental analytical results obtained for the selected tested soil samples, there are electrical conductivity (EC) and sodium adsorption ratio (SAR) impacts that are greater than the MOE Table 2 and Table 3 standards for residential/parkland/institutional (RPI) property uses and for industrial/commercial/community (ICC) property uses at the site. As the project site is a roadway, EC

and SAR impacts are expected to be present in the subsurface soils to varying degrees. Municipal roadways are exempt for all impacts related to de-icing activities (i.e. EC and SAR). Based on the available test results and in absence of other aesthetic indicators of impact, such as staining or odours, the site fill and native soils are considered to be suitable for reuse on site if kept within the municipal right of way (where geotechnically acceptable) but at a distance greater than 30m from surface watercourses or, alternatively, for off-site disposal at an approved receiver of commercial landuse fill who will accept the elevated SAR and EC values. Cost premiums will apply for the disposal of this SAR/EC impacted soil.

The topsoil and organic soils can be assumed to be suitable for transfer to land-based sites requiring topsoil fill. The environmental quality of the excavated topsoil and organic soils will need to be verified by the Contractor during construction. More topsoil testing required to be carried out by the contractor. For baseline purposes the Contractor is expected to assume that organic or nutrient amendment will be required for materials classified in this report as organic soils to support specific vegetation growth.

### **9.1.2 Tunnelling Spoils**

Materials excavated from the MTBM operation will consist of a blend of soil types described in this report mixed with groundwater and soil conditioning agents. The environmental quality of the in-situ soils, which are to be removed through MTBM operation, is suitable for transfer to a land-based receiving site pending receiving site acceptance (i.e. EC and SAR). However, tunneling operations will alter the in-situ state resulting in spoils that will consist of a mixture of materials. Given the anticipated consistency of the tunnel spoils and the potential effects of the Contractor's operations, the tunneling spoils will be physically and chemically unsuitable for engineering or construction uses without additional management and modification.

Depending on the soil type, conditioning agents added to the soil during construction are expected to alter the consistency of the tunnelling spoils. The consistency of the tunnelling spoil will affect handling and transportation methods and availability of disposal sites for the tunnelling spoil. The Contractor is expected to establish a drying area for treatment of the soils (e.g., by mixing and drying) prior to transportation to the Contractor's selected site. The drying area is to be designed and constructed with an impervious base that is sloped to a collection area for containment of water in accordance with Contract Documents. Collected water will need to be treated and tested by the Contractor prior to discharge.

The environmental quality of material excavated and discharged by the MTBM will also be influenced by the use of oil, grease, grouts and conditioning agents chosen by the Contractors. Therefore, the tunnel spoil will have to be sampled and tested by the Contractors to assess whether containment concentrations and physical characteristics are suitable for the selected receiving site. The Contractor is responsible for any change to disposal requirements that result from tunnelling operations.

The soil environmental test results obtained from the investigation for this baseline report indicate that there are elevated concentrations of electrical conductivity (EC) and sodium adsorption ratio (SAR). Blending of the excavated materials will inherently occur during tunnelling, and therefore, these elevated parameter concentrations are not expected to affect the transfer of material off-site should these materials not otherwise be affected and modified by tunnelling operations.



If environmentally-impacted material (i.e., visually stained odorous or chemically impacted materials) are suspected in the pumped slurry, the tunnel spoils in that slurry are to be separately stockpiled and are to be covered until the spoils are tested and characterized to determine appropriate disposal options. In order to verify the approximate disposal options, the suspect materials will need to be sampled by the Contractor and analysed in accordance with the requirement of O. Reg. 153/04 as amended.

### **9.1.3 Imported Fill**

Where imported fill is required at the site for construction, the Contractor is expected to provide granular materials and earth borrow soils that are free of organic matter, debris and deleterious materials that do not exceed the MOE's 2011 Table 1 Standards and that are in compliance with the requirements outlined in the Contract Documents.

## **9.2 Groundwater Control**

### **9.2.1 Groundwater Control in Open Cut and Shafts**

The majority of the excavation for the open cut section and the shafts will extend through predominantly cohesive and cohesionless soils and groundwater inflow into the excavations are expected. Dewatering systems consisting of closely spaced eductors/well points plus sumps and pumping will be required to control the groundwater seepage inflows into excavations for open cut excavation. If the shafts are constructed using contiguous caisson walls and "sealed" bases, "active" dewatering using eductors/well points will not be necessarily be required.

### **9.2.2 Groundwater Control in Tunnelling**

For the microtunnelling sections, the connection between the jacking pipe and the MTBM must be designed to resist groundwater seepage forces. Any groundwater leakage into the tunnel space between jacking pipes can be handled using sumps and pumping from the shafts.

The baseline groundwater table shown in **Section 5.3** must be used for the groundwater control design.

## **9.3 Surface Gases**

The deep soils in Southern Ontario are known to contain pockets of methane and hydrogen sulphide gas. Methane has been recorded in the Toronto Area, typically in granular layers capped by cohesive soils (Coleman, 1932). Methane can form an explosive mixture with air while hydrogen sulphide is toxic. Methane and hydrogen sulphide are also soluble in water. Change in groundwater pressure or flow (such as dewatering or seepage into underground spaces) can lead to migration of these gases, either as a gas or entrained in the groundwater.

There is a potential for an explosion occurring when methane vapours accumulate in a space to an explosive concentration (generally when methane gas in the air is 5% to 15% of the air volume). Methane concentrations of 5% of the air volume (v/v) (50,000 parts per million, ppm) is the lower explosive limit (LET) for this gas (i.e., the lowest concentration at which an explosion could occur).

The Contractor is required to prepare and implement his own procedure and Health and Safety Plans that address all hazards, monitoring requirements, including potential hazards from methane and other subsurface gases that will be encountered during tunnelling and excavation activities.

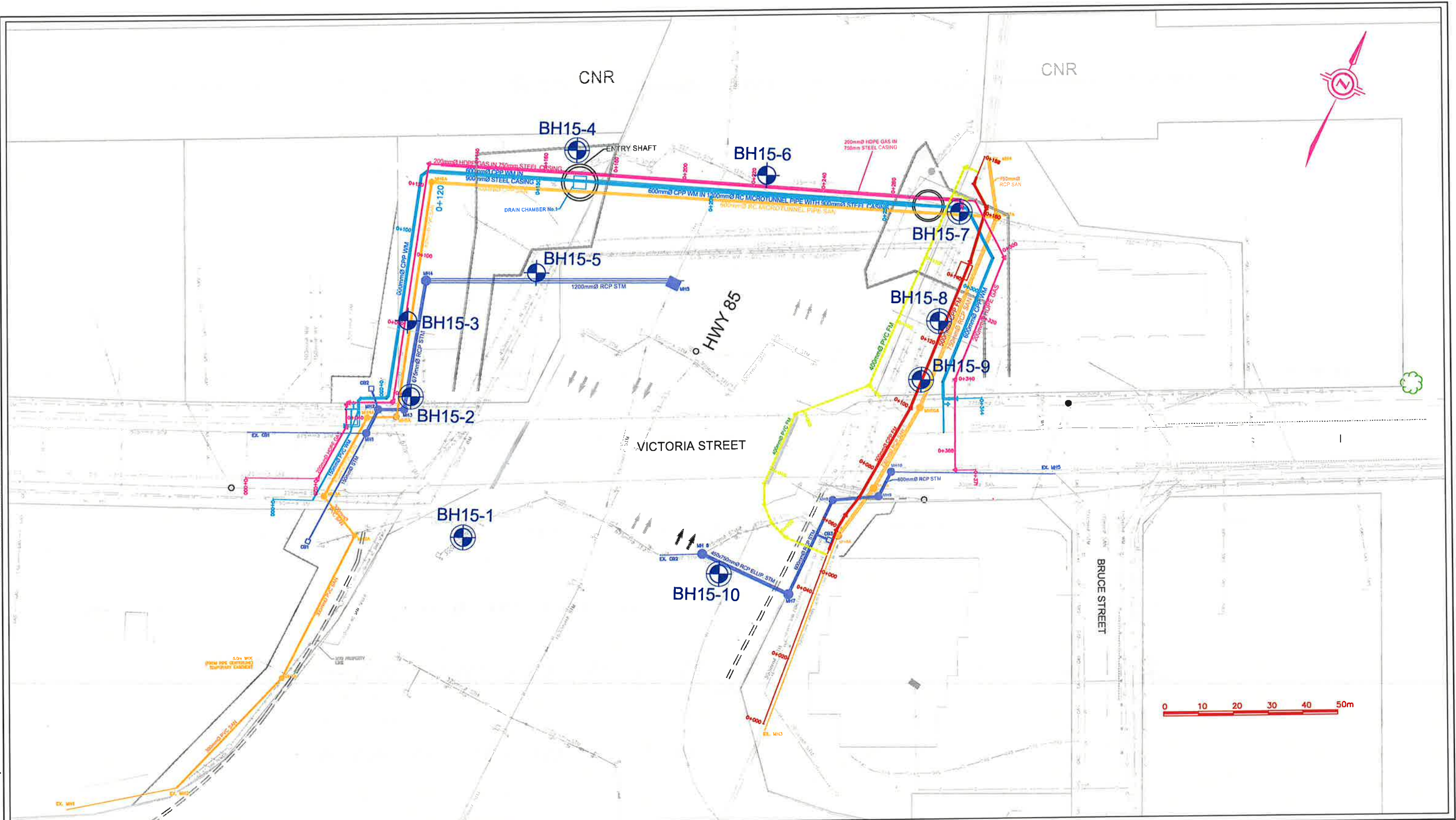
## **10.0 CLOSURE**

This Geotechnical Baseline Report was prepared by SPL Consultants Limited, with input and consultation by the project engineer, MMM Group Ltd. It is intended for use by bidders for construction of the Victoria Street Utilities Relocation project in the Town of Kitchener, Ontario.

Should there be any questions or need for further clarification on any aspect of this report, please contact this office.

# Figures

SERVER\Geotech\10 GeoProjects\1001-2000 Projects\1001862 - Victoria St. Kitchen\Drawings\1001862-Profile-Victoria-Aug27-2015.dwg




Key Map



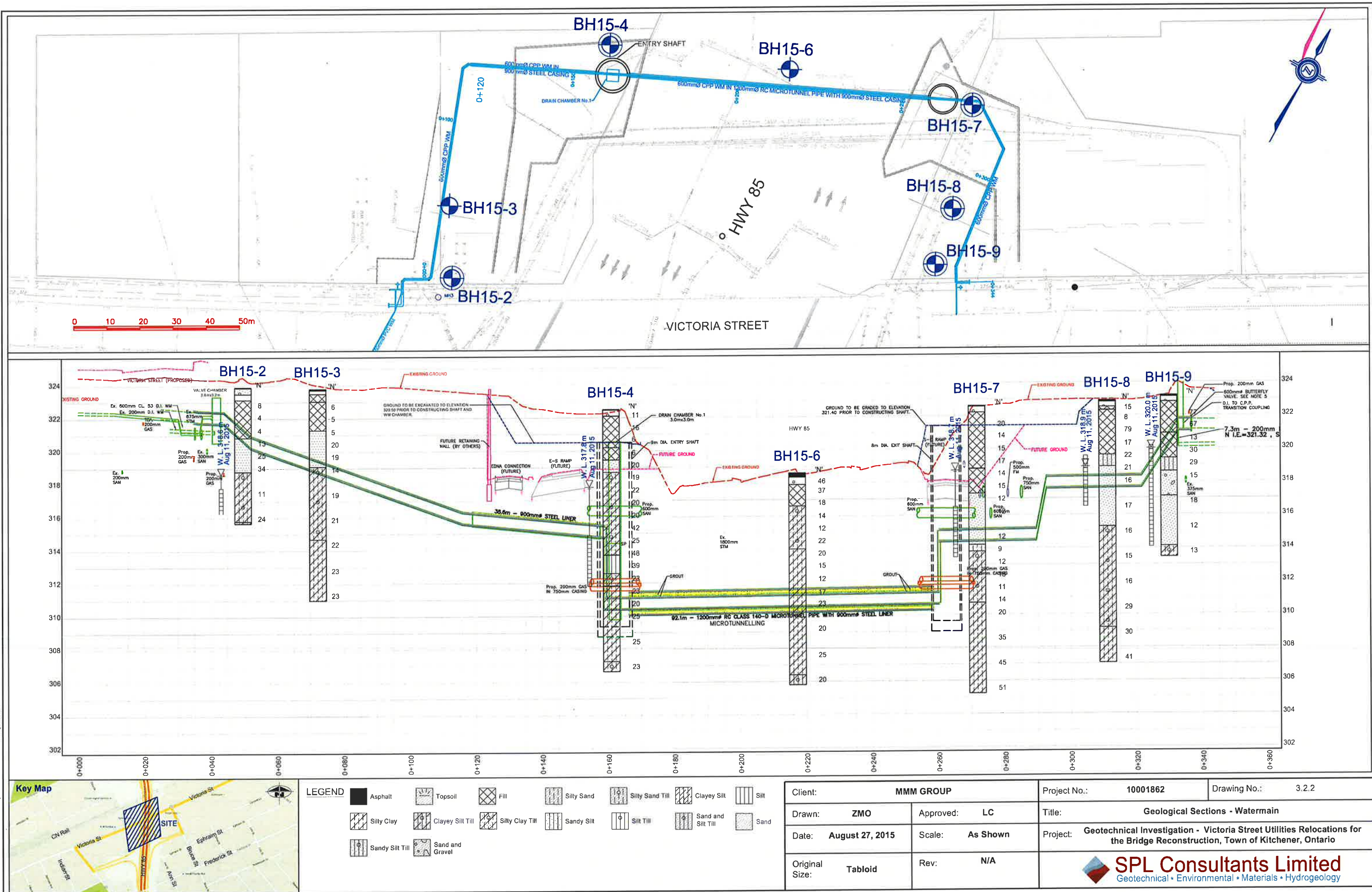
LEGEND

- Borehole Location
- Borehole with Monitoring Well Location

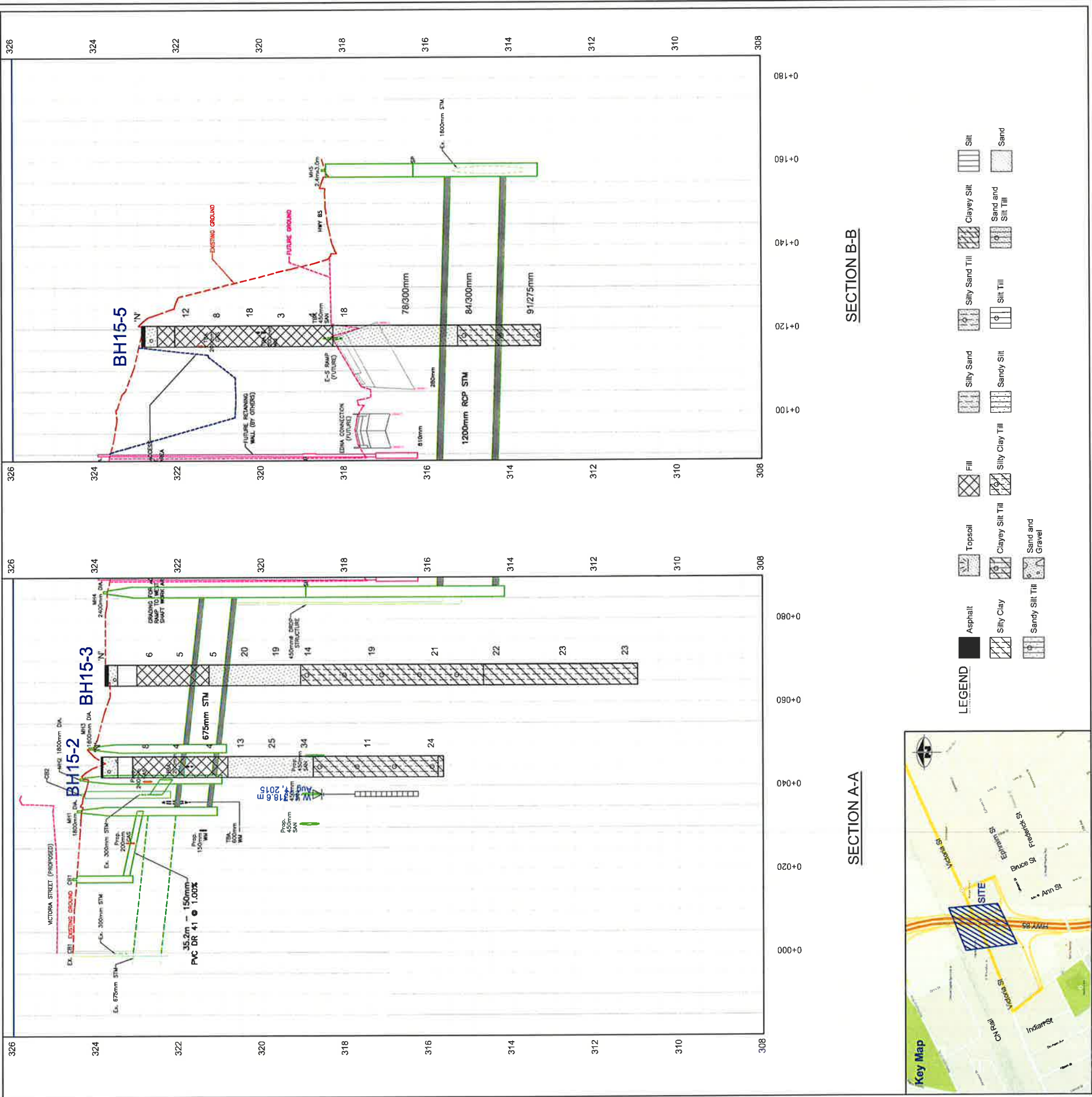
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Date: <b>August 27, 2015</b>	Scale: <b>As Shown</b>	Project: <b>Geotechnical Investigation - Victoria Street Utilities Relocations for the Bridge Reconstruction, Town of Kitchener, Ontario</b>	
Original Size: <b>Tabloid</b>	Rev: <b>N/A</b>	 <b>SPL Consultants Limited</b> Geotechnical • Environmental • Materials • Hydrogeology	



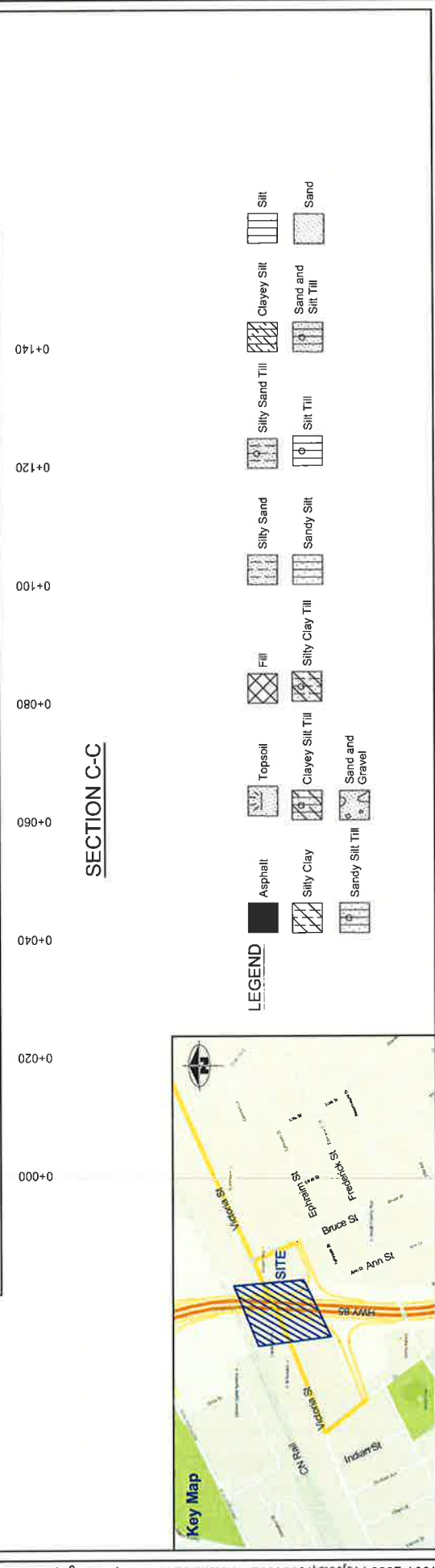
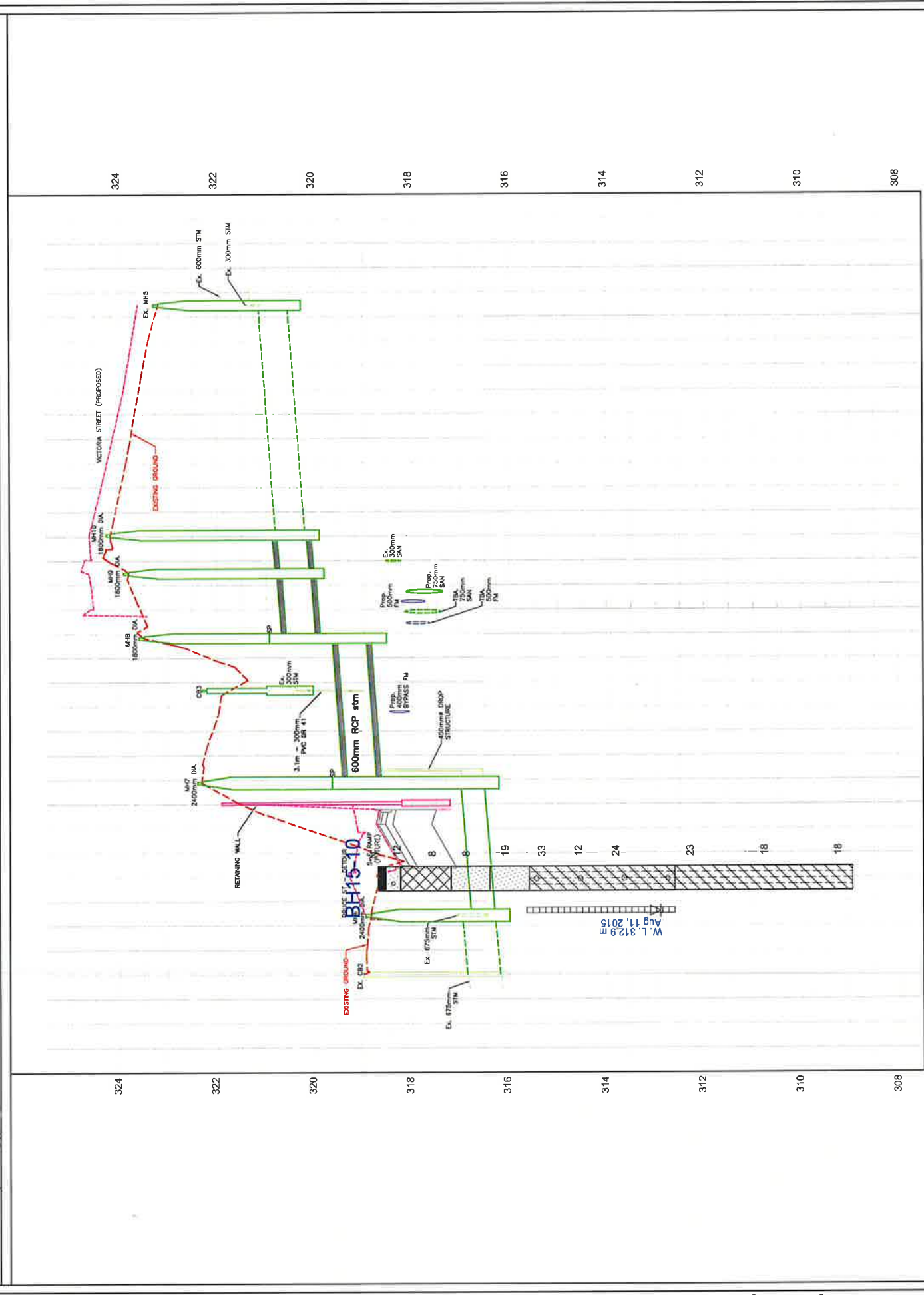
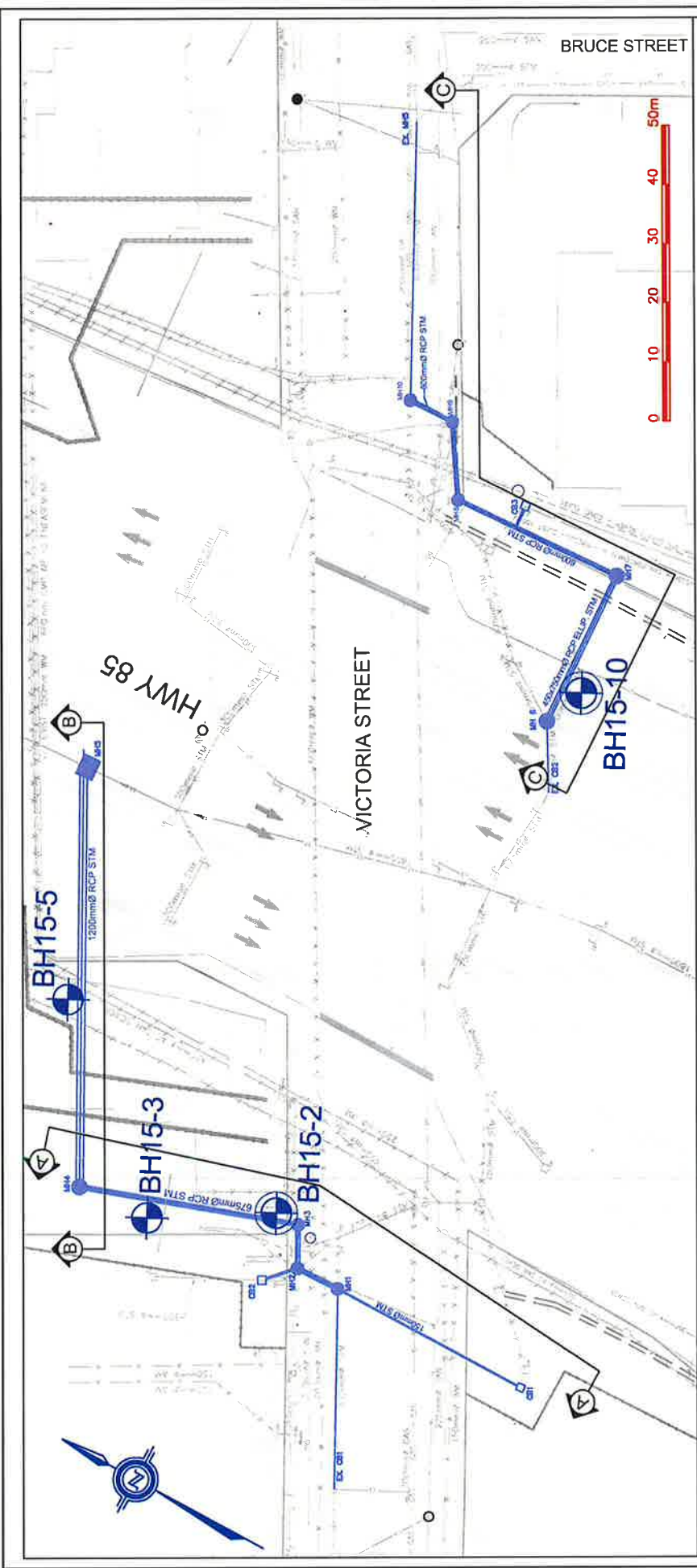
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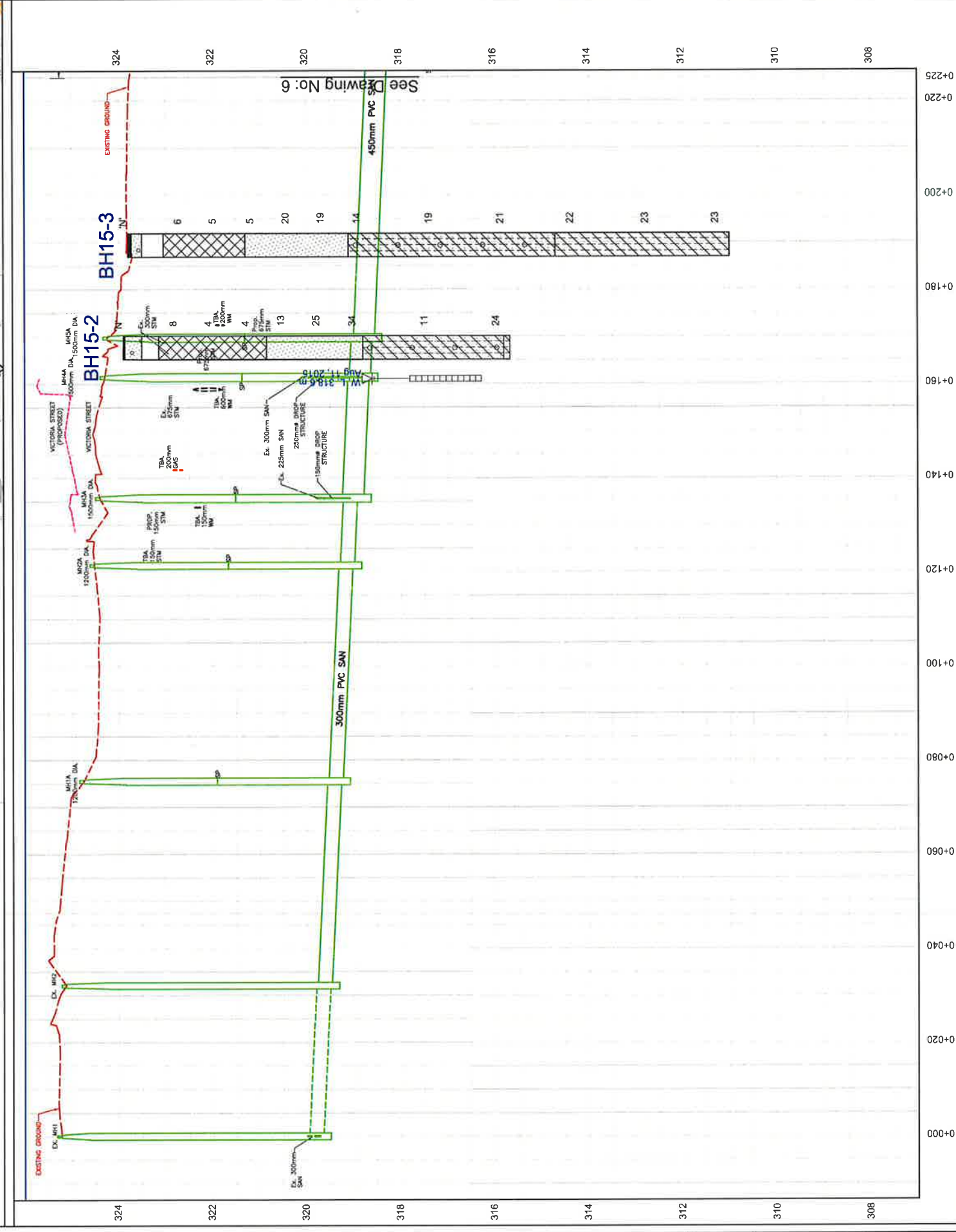
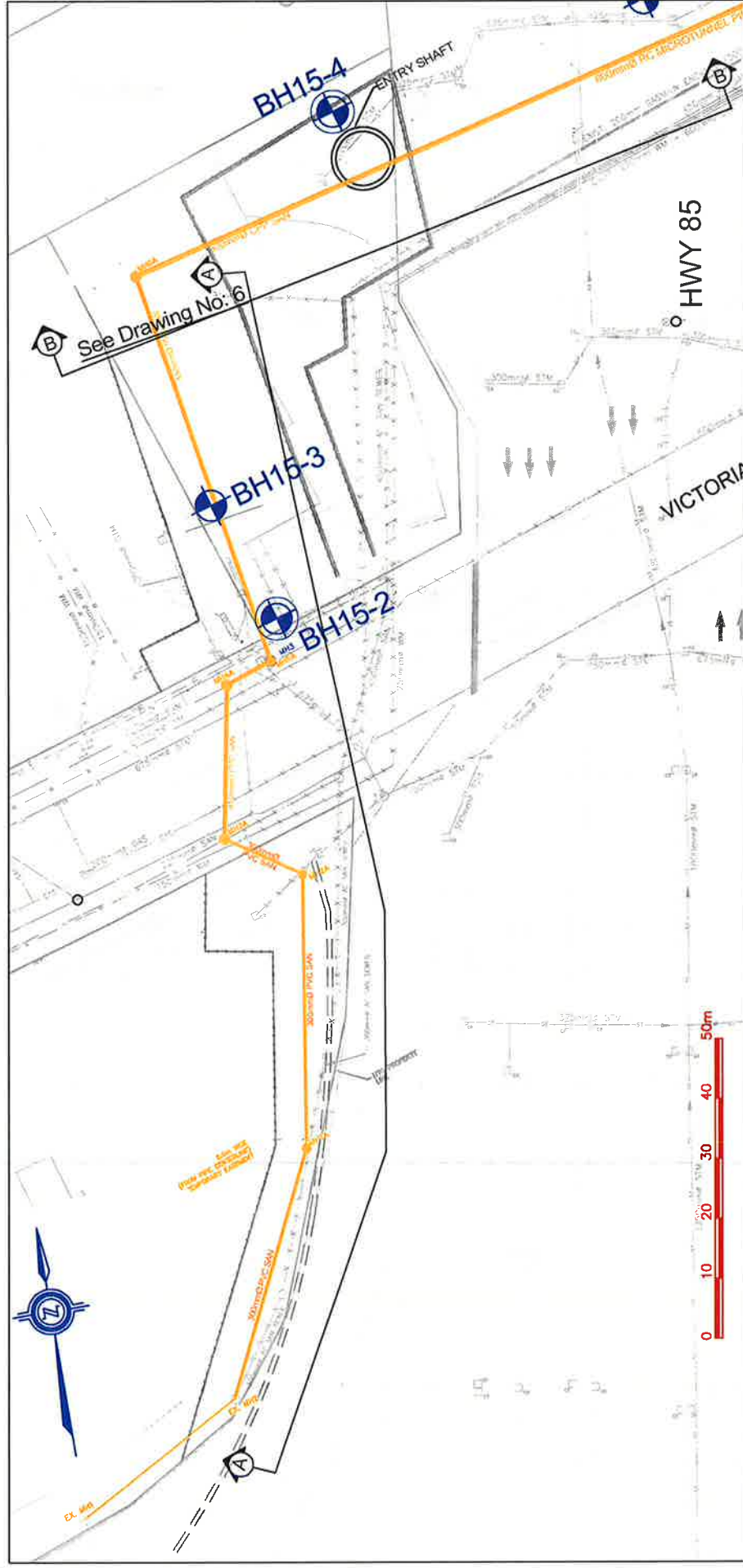







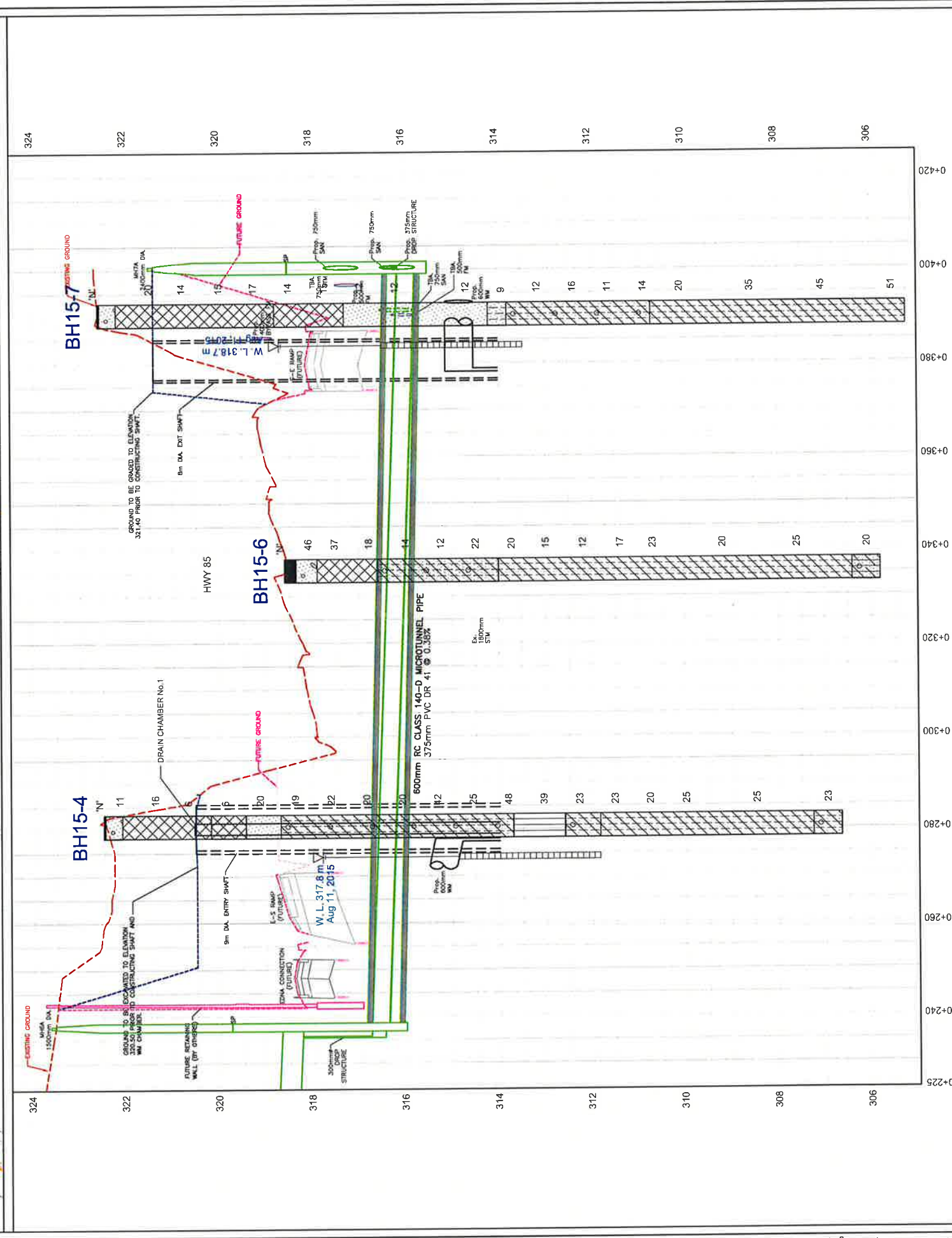
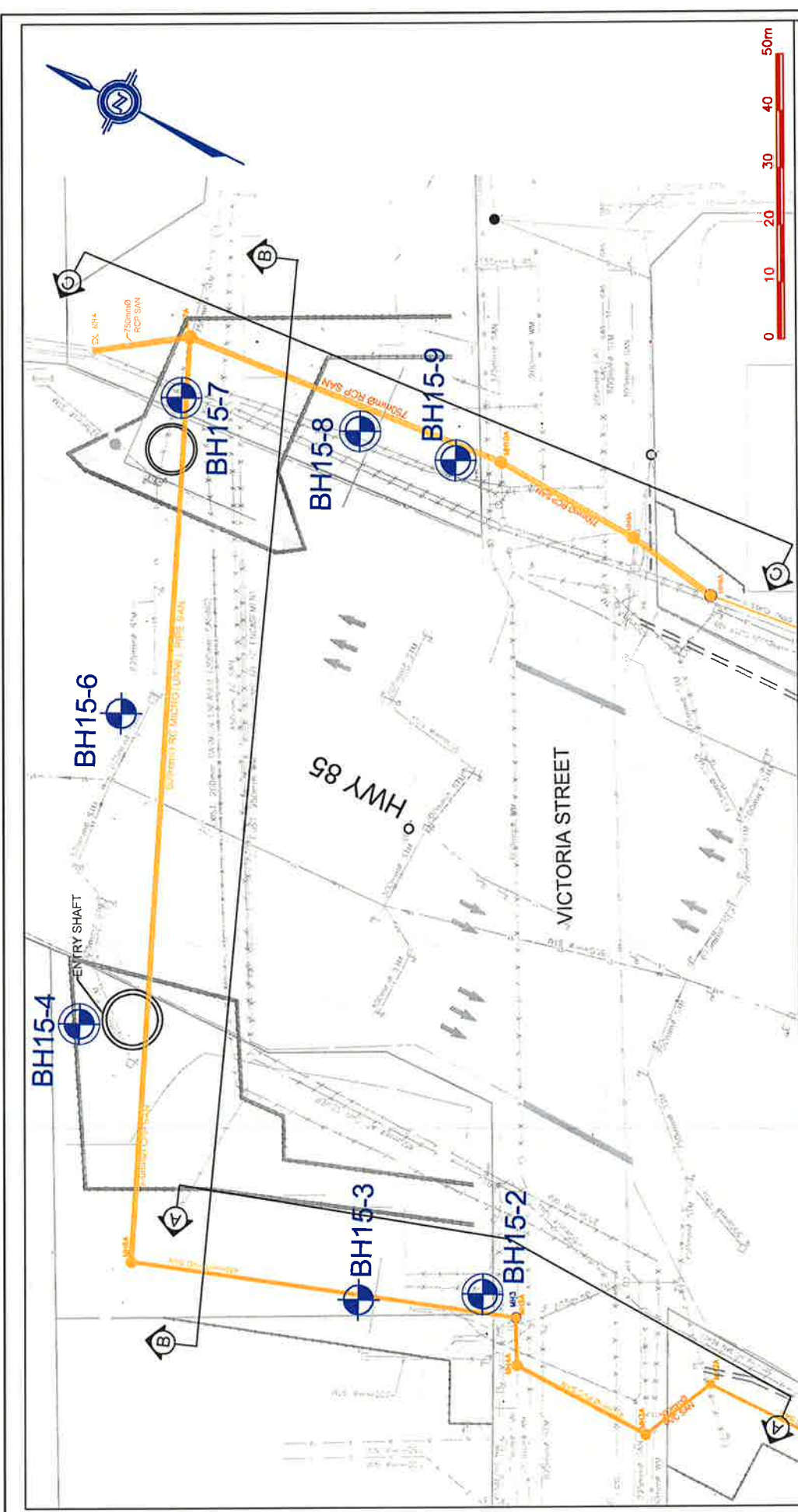


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Date:	August 27, 2015	Scale:	As Shown	Project: Geotechnical Investigation - Victoria Street Utilities Relocations for the Bridge Reconstruction, Town of Kitchener, Ontario			
Original Size:	Tablet	Rev:	N/A	SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology			



Client:	MMM GROUP		Project No.:	10001862	Drawing No.:	3.2.5
Drawn:	ZMO	Approved:	LC	Title: Geological Sections - West Sanitary Sewer		
Date:	August 27, 2015	Scale:	As Shown	Project: Geotechnical Investigation - Victoria Street Utilities Relocations for the Bridge Reconstruction, Town of Kitchener, Ontario		
Original Size:	Tabloid	Rev:	N/A	 <b>SPL Consultants Limited</b> Geotechnical • Environmental • Materials • Hydrogeology		





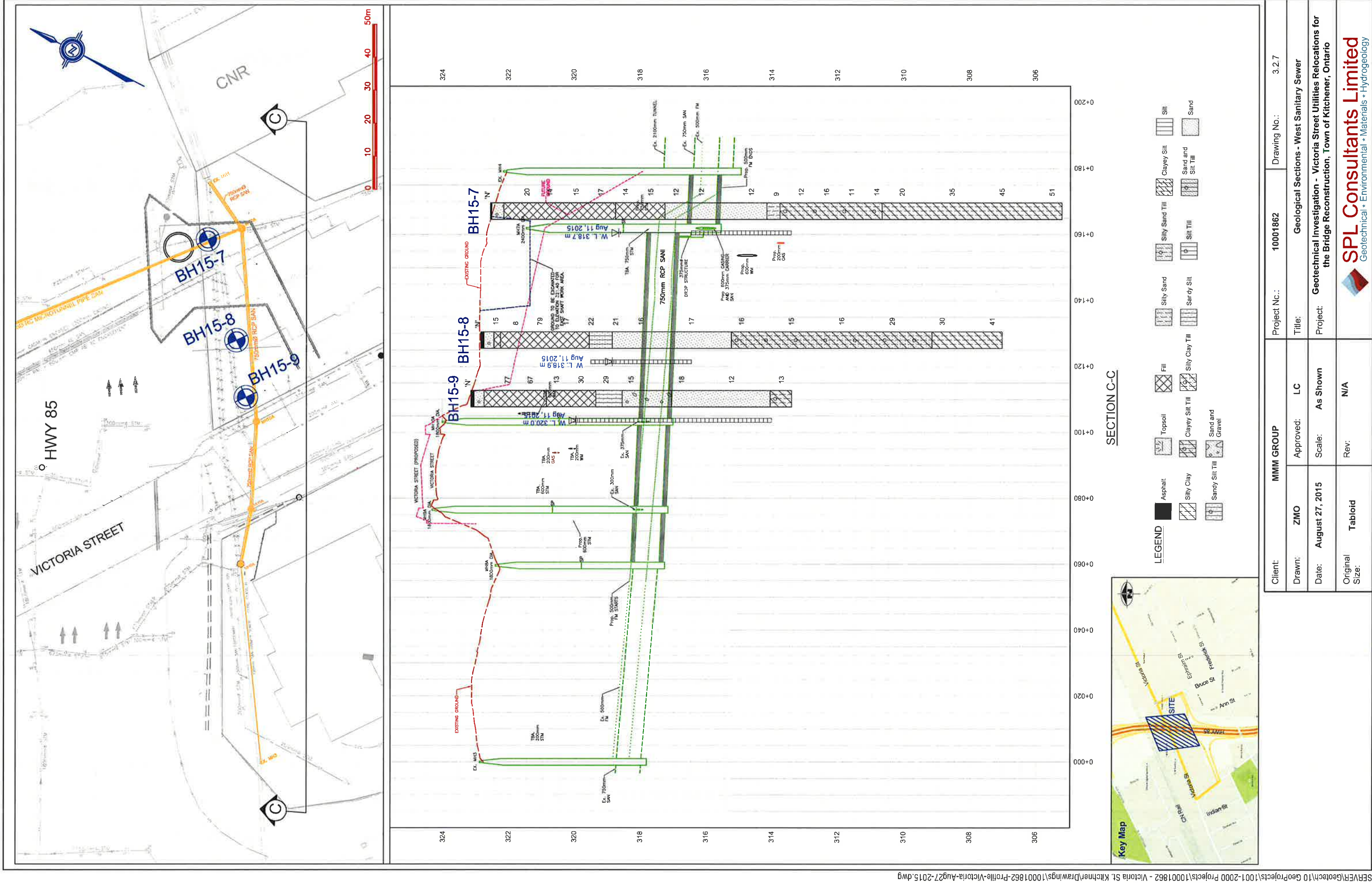
SECTION B-B

LEGEND	
Asphalt	Fill
Silty Clay	Topsoil
Sandy Silt Till	Clayey Silt Till
Silty Silt Till	Silty Sand
Sandy Silt Till	Silty Sand Till
Sand and Gravel	Silt
	Clayey Silt
	Sand and Silt Till
	Sand



Client:	MMM GROUP	Project No.:	10001862	Drawing No.:	3.2.6
Drawn:	ZMO	Approved:	LC	Title:	Geological Sections - West Sanitary Sewer
Date:	August 27, 2015	Scale:	As Shown	Project:	Geotechnical Investigation - Victoria Street Utilities Relocations for the Bridge Reconstruction, Town of Kitchener, Ontario
Original Size:	Tabloid	Rev:	N/A		<b>SPL Consultants Limited</b> Geotechnical • Environmental • Materials • Hydrogeology

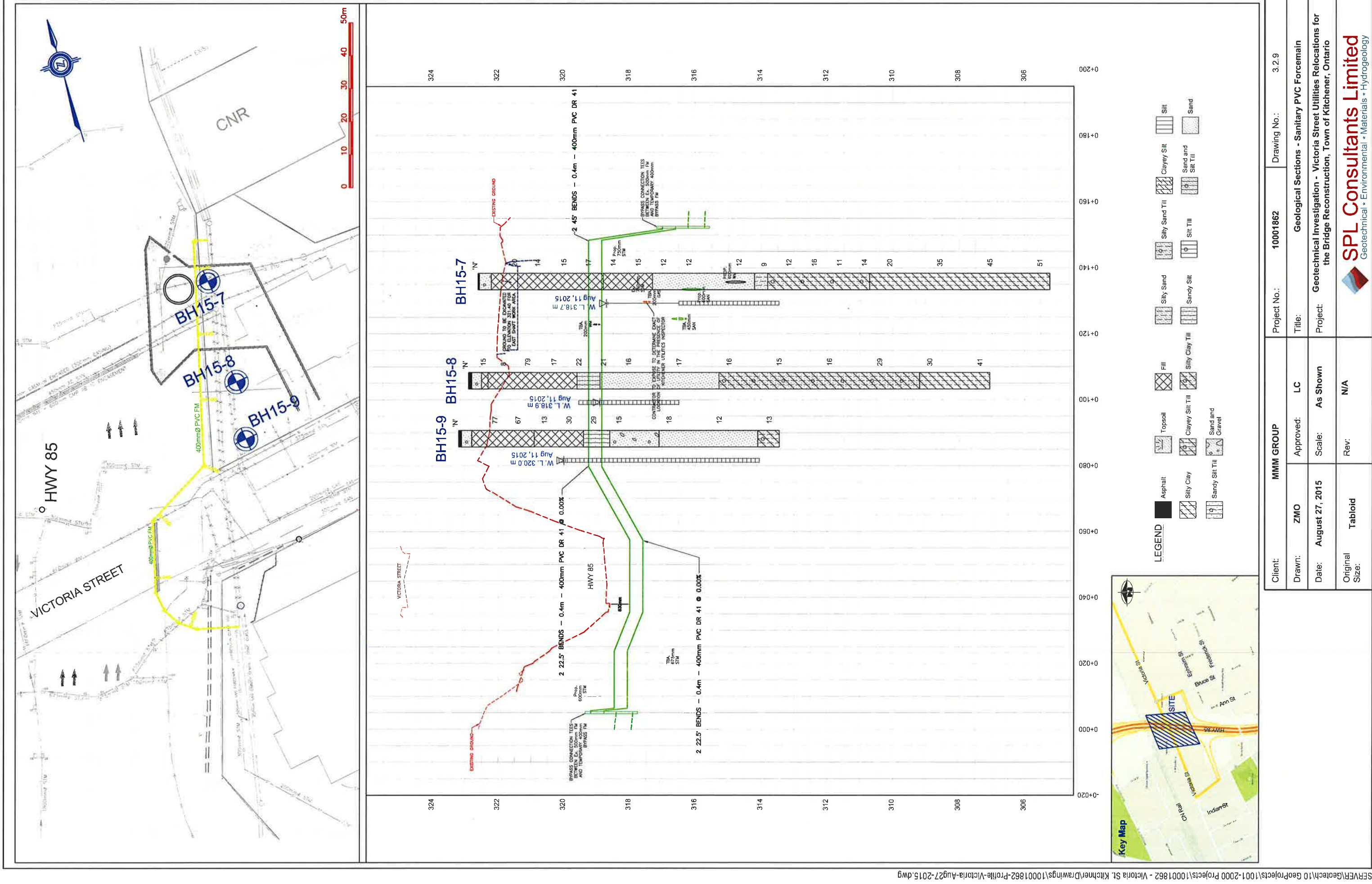


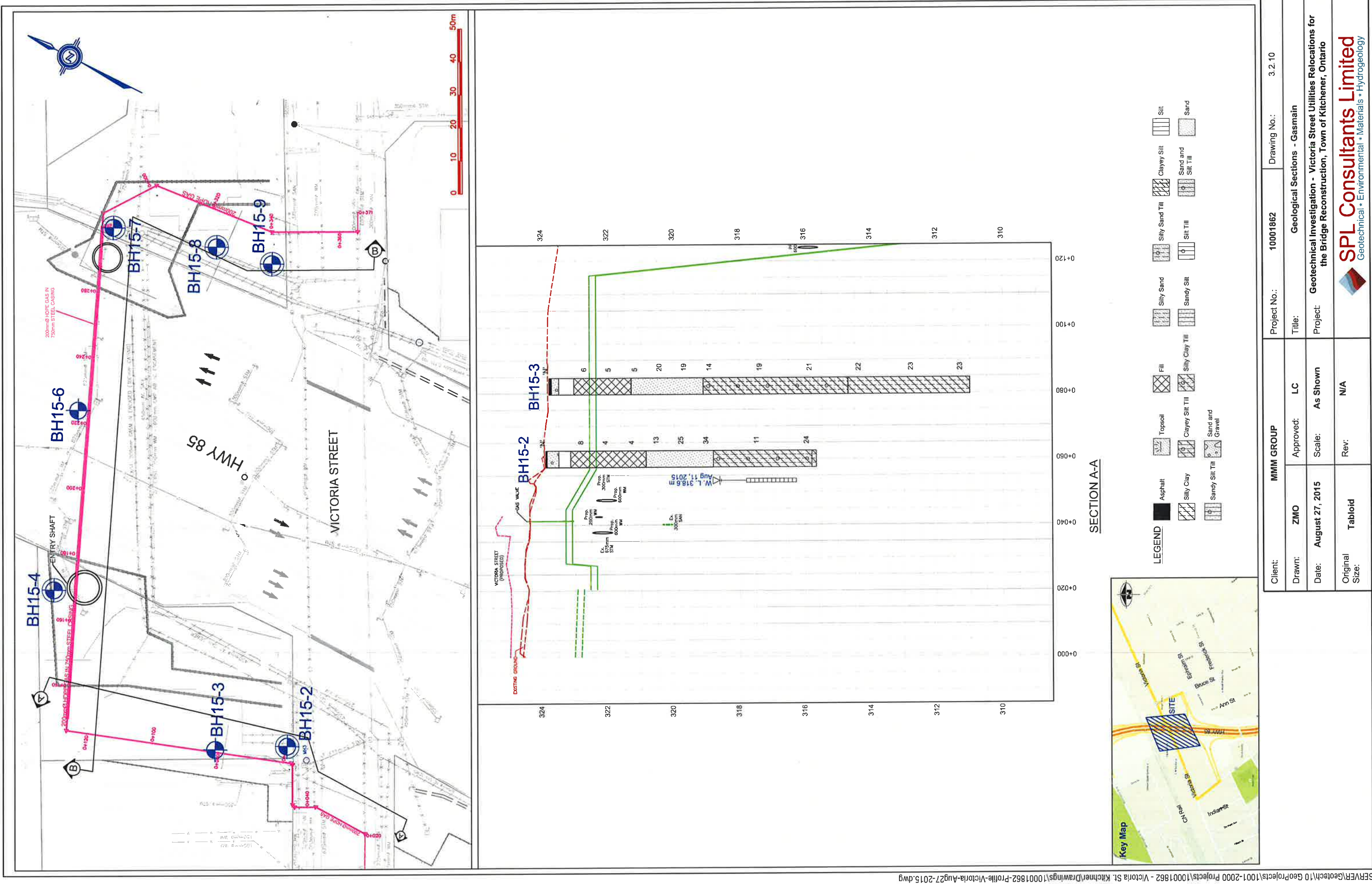




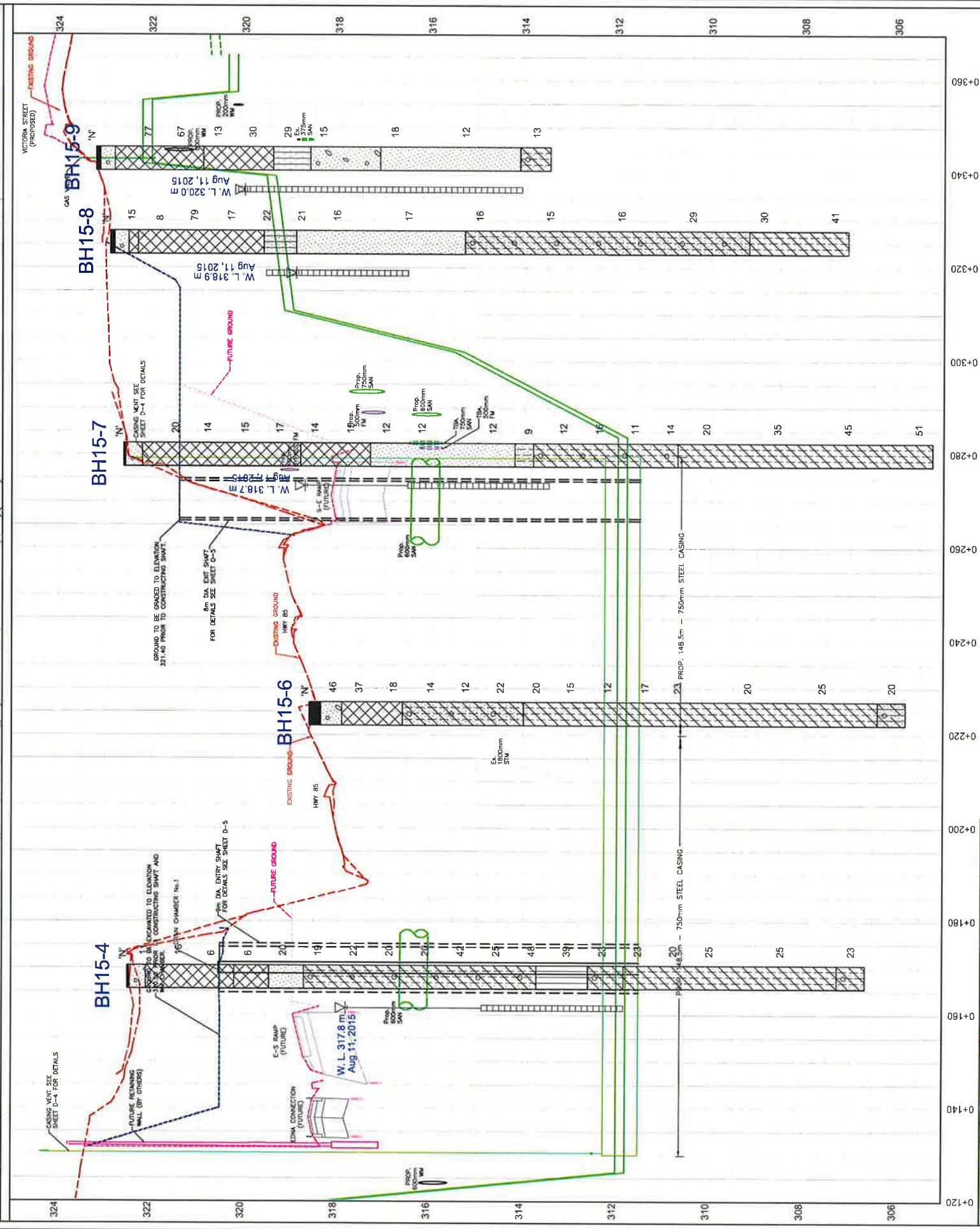
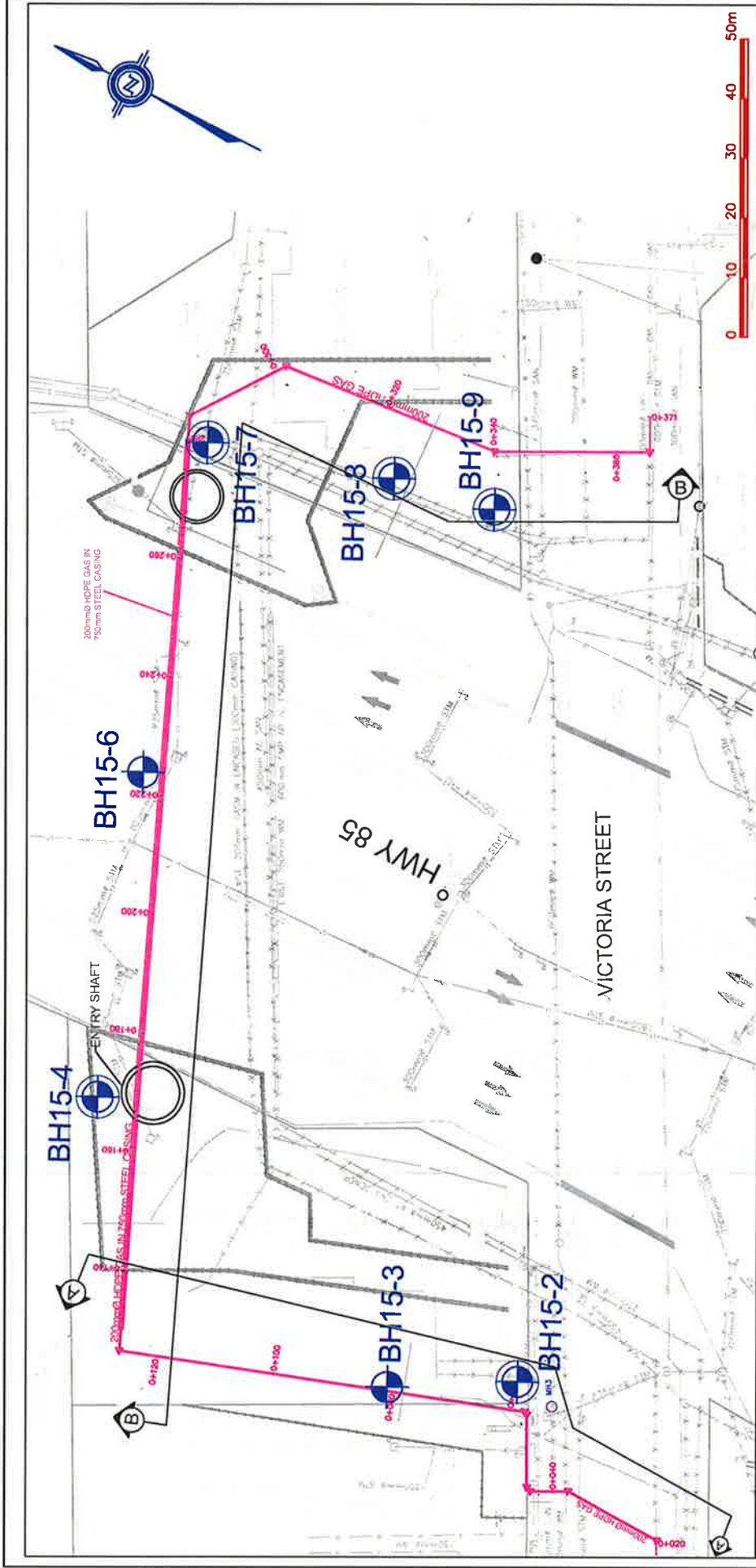










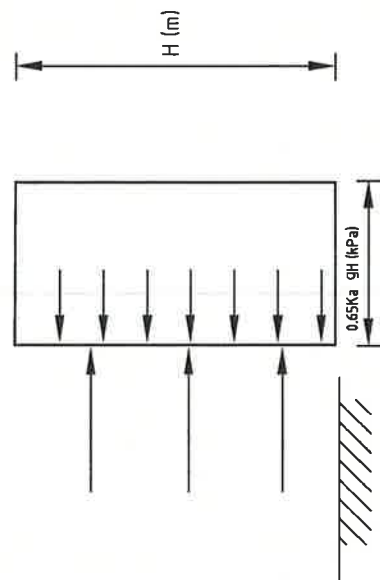


SECTION B-B

LEGEND

- Asphalt
- Silty Clay
- Sandy Silt Till
- Topsoil
- Clayey Silt Till
- Silty Sand
- Silty Sand Till
- Clayey Silt
- Silt
- Fill
- Silty Clay Till
- Sandy Silt
- Sand and Gravel
- Sand and Silt Till
- Silt Till
- Sand

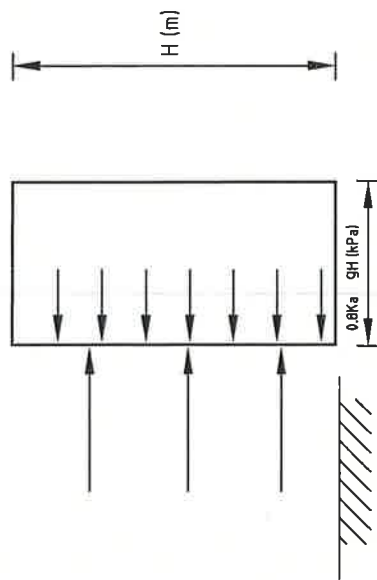
Client: MMM GROUP		Project No.: 10001862	Drawing No.: 3.2.11
Drawn: ZMO	Approved: LC	Title: Geological Sections - Gasmain	
Date: August 27, 2015	Scale: As Shown	Project: Geotechnical Investigation - Victoria Street Utilities Relocations for the Bridge Reconstruction, Town of Kitchener, Ontario	
Original Size: Tabloid	Rev: N/A	SPL Consultants Limited Geotechnical • Environmental • Materials • Hydrogeology	

 $q = \text{unit weight of soil} = 21.0 \text{ kN/m}^3$ 

$\gamma' = \text{submerged unit weight of soil (i.e. below ground water level)} = 11.2 \text{ kN/m}^3$

 $K_a = 0.3$ 

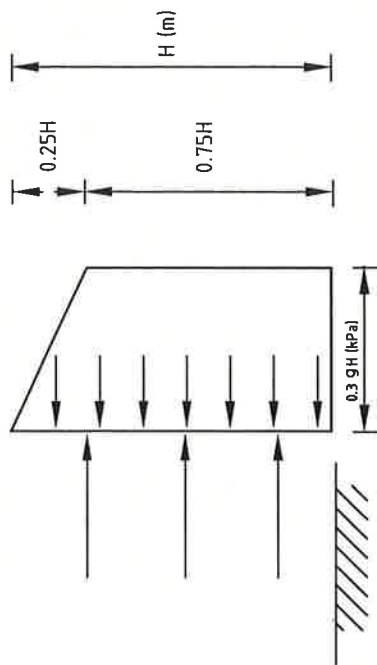
## IN COMPACT TO VERY DENSE NON-COHESIVE SOILS (SANDS AND SILTS)

 $\gamma = \text{unit weight of soil} = 19.0 \text{ kN/m}^3$ 

$g' =$  submerged unit weight of soil (i.e. below ground water level) = 9.2 kN/m<sup>3</sup>

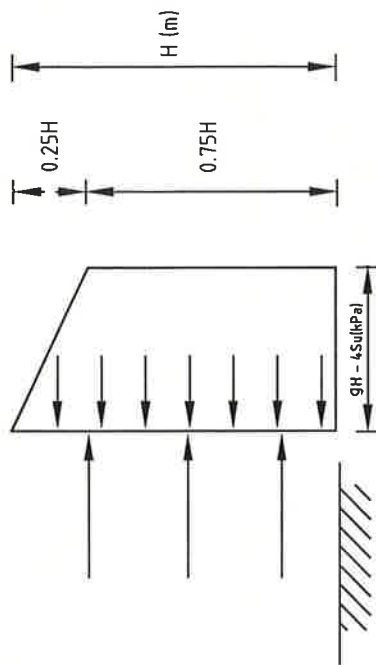
 $K_a = 0.36$ 

## IN LOOSE OR DISTURBED NON-COHESIVE SOILS (SANDS AND SILTS)

 $q = \text{unit weight of soil} = 21.5 \text{ kN/m}^3$ 

$\gamma' =$  submerged unit weight of soil (i.e. below ground water level) =  $11.7 \text{ kN/m}^3$

## IN COHESIVE CLAYS OR CLAYEY SOILS


$$g = \text{unit weight of soil} = 19.0 \text{ kN/m}^3$$

$\gamma' =$  submerged unit weight of soil (i.e. below ground water level) = 9.2 kN/m<sup>3</sup>

 $S_u = 10 \text{ kPa}$ 

**IN VERY SOFT TO FIRM COHESIVE CLAYS OR CLAYEY SOILS**

Notes:

1. Check system for partial excavation condition.
2. If the free water level is above the base of the excavation, the hydrostatic pressure must be added to the above pressure distribution.
3. If surcharge loadings are present near the excavation, these must be included in the lateral pressure calculation.

Client:	MMM GROUP		Project No.:	10001862	Drawing No.:	12
Drawn:	ZMO	Approved:	NE			
Date:	August 31, 2015	Scale:	N.T.S			
Original Size:	Letter	Rev:	0			
		Title:		Earth Pressure Distribution on Braced Excavations		
		Project:		Geotechnical Investigation - Victoria Street Utilities Relocations for the Bridge Reconstruction, Town of Kitchener, Ontario		
				 <b>SPL Consultants Limited</b> Geotechnical • Environmental • Materials Hydrogeology		

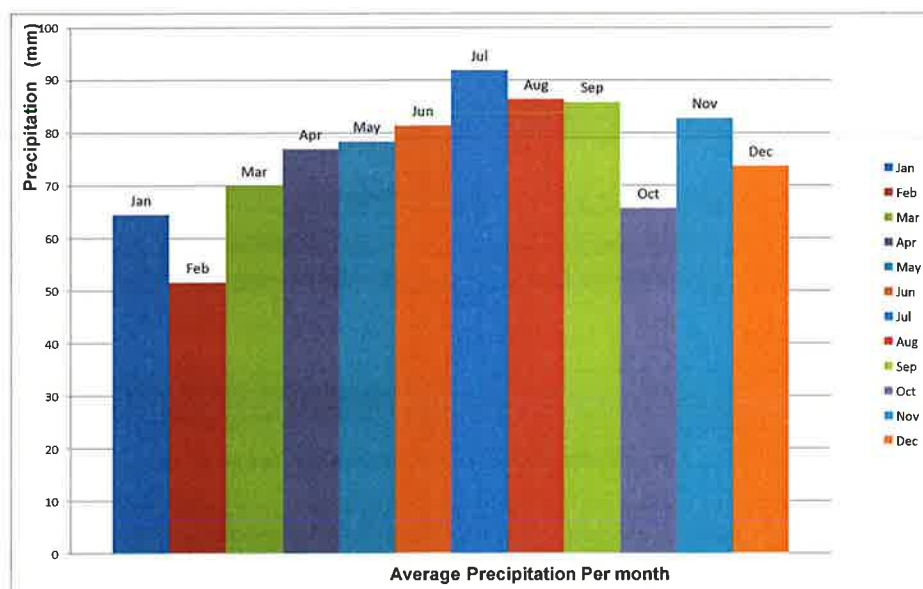
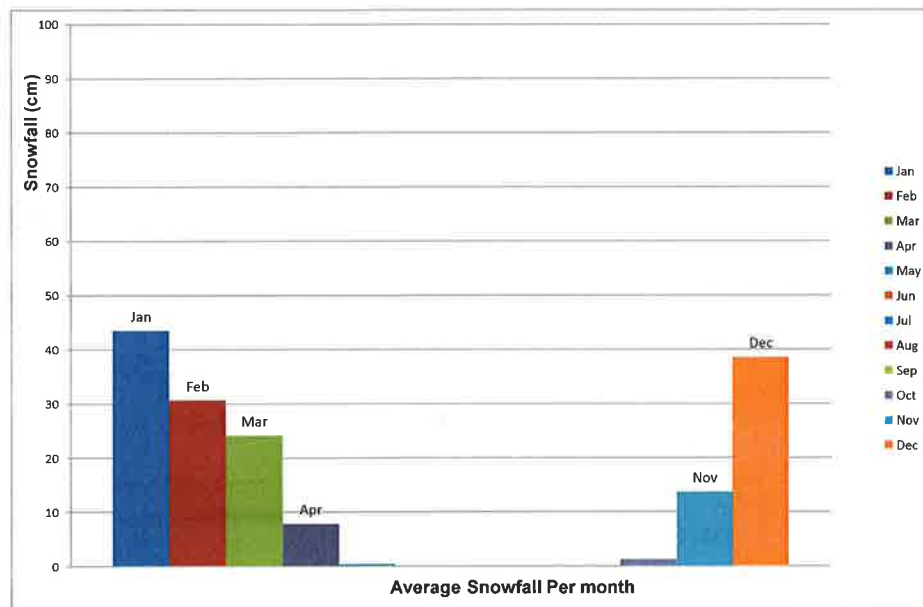
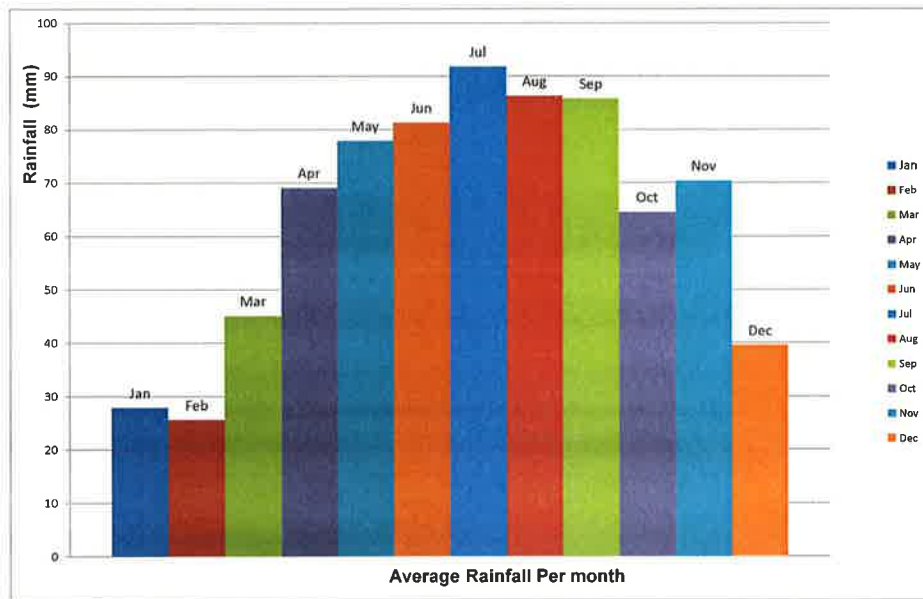
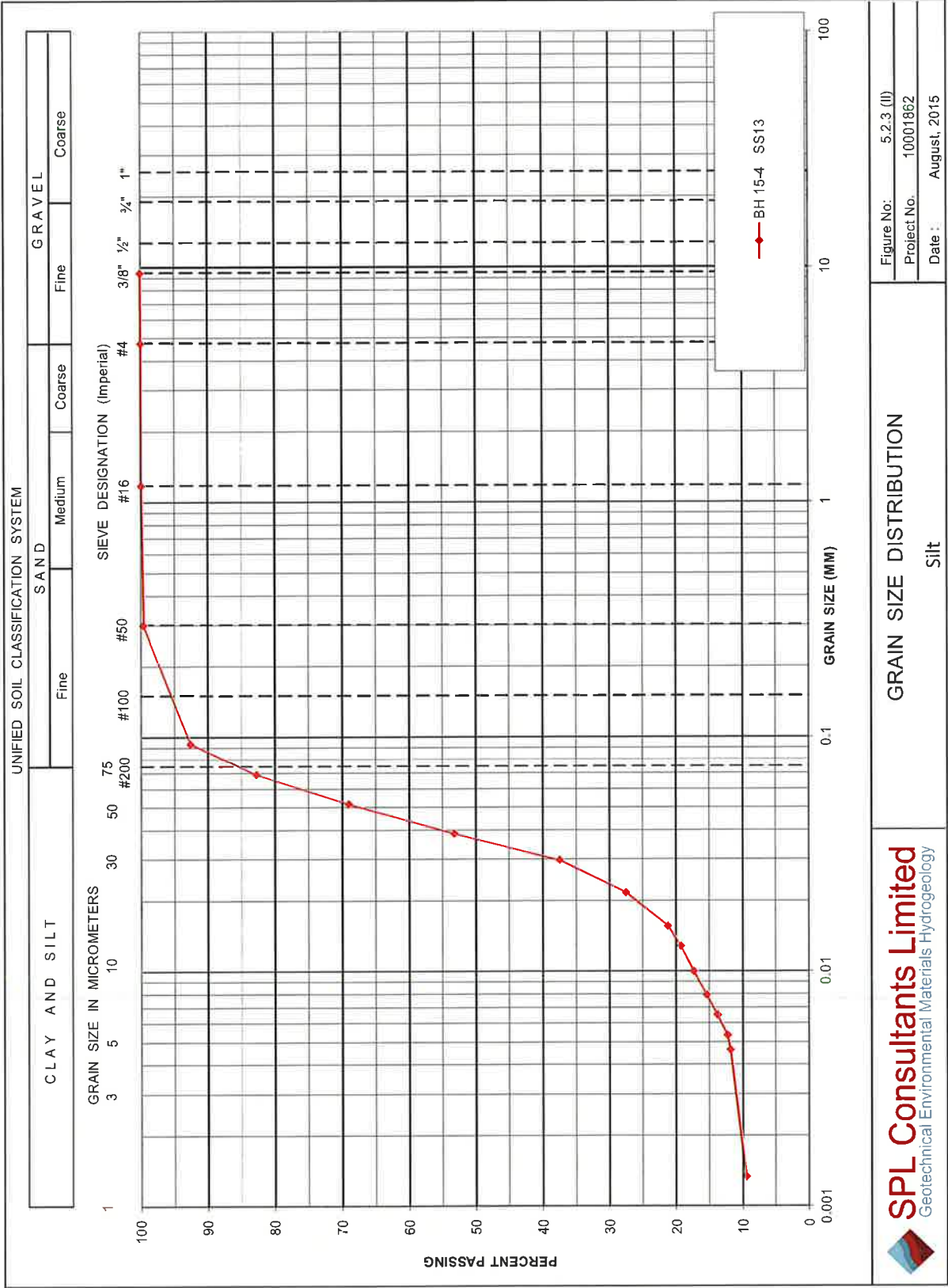
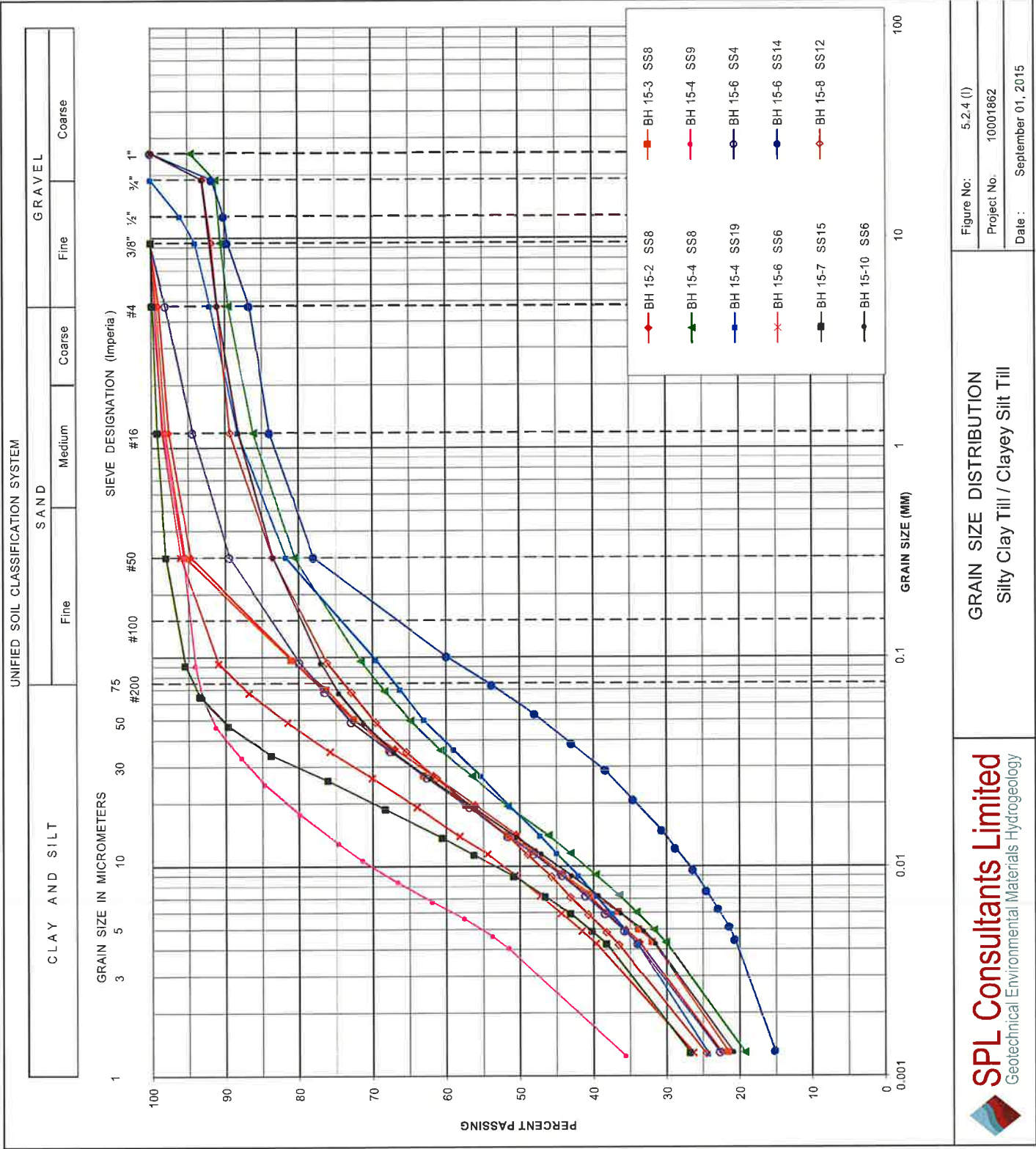


Figure 3.5 Variation of Rainfall, Snowfall and Precipitation with Month (1971~2000)









# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	

