Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION
VOLUME 2: DESIGN REPORT

ZONE 1 INTERCONNECTING WATERMAIN
BURLOAK WPP TO KITCHEN RESERVOIR
REGION OF HALTON
ONTARIO

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Issued for MTO and CN Rail Review
File No. 11-12-2073
August 22, 2013

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1.0 INTRODUCTION

Terraprobe was retained by R.V. Anderson Associations Ltd. to conduct a geotechnical subsurface investigation and background review for the proposed Halton Region Zone 1 Interconnecting Watermain. The findings from the subsurface investigation conducted by Terraprobe, as well as the findings from previously-conducted subsurface investigations by various consultants (including Terraprobe) within the study area, are presented in the Zone 1 Interconnecting Watermain Factual Report (Volume 1, Terraprobe File No. 11-12-2073).

The Zone 1 Interconnecting Watermain Design Report (Volume 2) provides interpretation, analysis and advice with respect to the geotechnical engineering aspects of the proposed watermain. The anticipated construction conditions pertaining to excavation, tunnelling, groundwater control, and backfilling are discussed with regard to how these might influence the project design.

1.1 The Project

The proposed watermain is to connect the Kitchen Reservoir Pumping Station ("RPS") and the Burloak Water Purification Plant ("WPP"). The proposed watermain will be located within a corridor following Burloak Drive from the Burloak WPP at Rebecca Street to Upper Middle Road and along an unopened road allowance across Bronte Creek to the Kitchen Reservoir. Portions of the watermain that run along the east and west sides of Burloak Drive will be located in Oakville and Burlington, respectively. A site location plan showing the proposed alignment is provided as Figure 1.

There are two primary functional requirements for the proposed Zone 1 Watermain. It will augment water conveyance from the Burloak WPP to the Kitchen Reservoir. It will also supply water to a future Zone 2 Booster Pumping Station ("BPS") located to the south of the Burloak / QEW interchange, in a parcel of land that has been acquired by the Region of Halton. The south leg of the Zone 1 Watermain, extending from Burloak WPP to the proposed Zone 2 BPS, will become the supply line for the Zone 2 BPS feed into the Zone 2 distribution system.

The watermain is to be constructed within a tunnel advanced through bedrock. The annular space of the tunnel will be filled, but there will be no tunnel liner. The connections with the Burloak WPP and Kitchen Reservoir facilities will be constructed as open cut sections.

A portion of the proposed watermain will be constructed within a tunnel advanced beneath Bronte Creek, which lies within a 35 to 40 m deep valley located in Bronte Creek Provincial Park (BCPP). The approximate elevation of Bronte Creek at this location is Elev. 105 ± m. Bronte Creek has been designated as an environmentally sensitive area.

1.2 Profile and Alignment

The Preliminary Design Report ("PDR", R.V. Anderson Associations, Ref. No. 112525, dated April 18, 2013) proposes that the entire watermain alignment be tunnelled through Queenston Formation bedrock. The watermain is to comprise a minimum 2440 mm diameter tunnel conveying a 1500 to 1800 mm diameter watermain from Burloak WPP to Kitchen Reservoir, with connections to the Burloak WPP and Kitchen Reservoir facilities constructed as open cut sections. The proposed alignment is shown on Figures 1 and 2, and the proposed shaft locations are summarized in the following table.

Table 1-1: Summary of Proposed Shaft Locations

Station	Shaft Location	Function[1]	Diameter (m)	Tunnel Invert Elev.
1+000	**Burloak WPP**	O&M access and construction (TBM egress) & Watermain Shaft	2 @ 3.6	76.8 m
3+085	**Main Shaft**	O&M access and construction (mucking, TBM access)	15±	74.7 m
4+981	**Ontario Parks Shaft**	O&M access and construction (optional mucking, TBM access)	3.6	83.5 ±
7+300	**Kitchen Reservoir**	Watermain Shaft & O&M access and construction (TBM egress)	2 @ 3.6	94.1 m

Note 1: Source: R.V. Anderson Associates, *Preliminary Design Report: Zone 1 Watermain*, Ref. No. 112525, April 18, 2013

Revisions to the watermain profile were made continuously by RVA as the findings from the geotechnical field investigation became available. Previous iterations of the design have included various additional open-cut sections. These include portions of the south leg of the alignment as well as the end portion of the north leg of the alignment at Kitchen Reservoir. Shallow boreholes were advanced along those potential open-cut sections, should that information be required for the tender.

For the proposed "V" tunnel configuration, the tunnel would grade down towards the shaft at the Zone 2 BPS from both the north and south, at 0.46% and 0.1% respectively.

1.3 Sources of Geotechnical Information

The current Terraprobe investigation involved advancing twenty-seven (27) exploratory boreholes along the proposed watermain alignment. The locations of the boreholes are provided on the Borehole Plan and Profile as Figure 2. The boreholes were laid out in consultation with RVA.

The terms of reference provided to Terraprobe include previous investigations completed by other engineering consultants in the vicinity of the site. The locations of the previously-advanced boreholes are included on the Borehole Plan and Profile as Figure 2. The geotechnical investigation reports from those previous investigations are as follows:

- Coffey Geotechnics Inc., "Report on Preliminary Geotechnical Investigation, Zone 1 Interconnecting Watermain, Bronte Creek Provincial Park, Burlington, Ontario." Project No. GEOTMARK00158AA, dated September 28, 2011.
- Coffey Geotechnics Inc., "Geotechnical Investigation for the Land Acquisition of the Proposed Zone 2 Booster Pumping Station Site, 945 Syscon Road, Burlington, Ontario." Project No. ENVSETOB10863AB, dated November 9, 2011.
- Geo-Canada Ltd., "Report on Geotechnical Investigation, Burloak Water Purification Plant, Intake Tunnel, On-Land Section, The Regional Municipality of Halton. Vol. 1: Factual Data" Project No. G-04.1003, dated January 2005.
- Geo-Canada Ltd., "Report on Geotechnical Investigation, Burloak Water Purification Plant, Intake Tunnel, On-Land Section, The Regional Municipality of Halton. Vol. 2: Geotechnical Interpretation and Recommendations" Project No. G-04.1003A, dated February 2005.
- O'Connor Associates, "Environmental Site Assessment, Bronte Junction Facility, Burloak Drive, Oakville." Job No. 10-6709, dated August 2003.
- Terraprobe Ltd., "Geotechnical Investigation, Upper Middle Road and Burloak Drive, Burlington, Ontario." Project No. 7-05-0163, dated August 6, 2008.
- Terraprobe Ltd., "Additional Geotechnical Investigation, Upper Middle Road and Burloak Drive, Burlington, Ontario." Project No. 7-05-0163-1, dated June 1, 2009.
- Thurber Engineering Ltd., "Geotechnical Investigation, Proposed 750 mm Watermain, Burloak Drive at QEW, Oakville, Ontario." File No. 19-4717-0, dated February 21, 2006.
- Trow Consulting Engineers Ltd., "Geotechnical Investigation, Proposed Residential Development, Burloak Drive and Rebecca Street, Oakville, Ontario." Project No. BRGE0060387a, dated June 22, 2001.
- Trow Consulting Engineers Ltd., "Geotechnical Evaluation, Proposed Road Construction, Rebecca Street and Great Lakes Blvd., Oakville, Ontario." Project No. BRGE0058013f, dated February 27, 2002.

A geophysical survey of the Bronte Creek valley was presented by Coffey Geotechnics as an appendix to the September 2011 report on the subsurface conditions surrounding the valley. The geophysical survey was conducted by Geophysics GPR International Inc. (May 2010) for the purposes of conducting non-destructive testing of the depth to bedrock in the valley. The geophysical study is appended to the Coffey Geotechnics (September 2011) report.

Boreholes advanced as part of the current investigative effort (by Terraprobe) are named using the proposed watermain alignment chainage, and are shown in plan on Figures 2A-F in red. Previously completed boreholes (by others) are accompanied by a prefix to denote which consultant advanced each borehole.

The factual information, including the 2013 borehole logs, rock core photographs, and geotechnical laboratory testing as well as the factual information secured from previous investigations, is provided in Volume 1 of this report.

2.0 SUBSURFACE CONDITIONS

2.1 Stratigraphy

The detailed factual information obtained during this investigation is presented in Volume 1 of this report. The subsurface soil, rock and ground water conditions encountered in the boreholes are presented on the Log of Borehole sheets as Appendix A (present investigation) and Appendix D (previous investigations). A summary of the geotechnical laboratory tests is provided as Appendix B, and the rock core photographs are provided as Appendix C.

The subsurface soil, rock and ground water conditions encountered in the boreholes are presented on the attached Log of Borehole sheets. The stratigraphic boundaries indicated on the Log of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil or rock type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed in a series of widely spaced boreholes, and will vary between and beyond the borehole locations. The discussion has been simplified in terms of the major soil and rock strata for the purposes of geotechnical design.

It should be noted that the subsurface conditions are confirmed at the borehole locations only, and may vary at other locations, particularly with respect to depth and condition of earth fill. The topsoil thickness indicated on the borehole logs is approximate only and should not be used in estimating quantities of depths of topsoil for stripping purposes. A series of test pits should be excavated to better assess anticipated topsoil stripping depths and quantities.

Based on the findings in the boreholes, the subsurface conditions at the proposed Zone 1 Interconnecting Watermain site are generally conceptualized as follows.

2.1.1 Overburden Soils

The ground surface at the site is covered by various types of surficial pavement and earth fill materials. Underlying the surficial materials, most of the boreholes across the site (92 of 104) penetrated a native deposit of glacial till. The glacial till has a cohesive matrix of silt to clayey silt, and contains embedded sand and gravel, and probably cobbles and boulders. The clayey silt glacial till is generally reddish brown to brownish red. The top metre (+/-) of the till has been weathered and mottled by seasonal frost penetration. This zone contains embedded rootlets and trace amounts of organics. Below the weathered zone, the clayey silt till has a very stiff to hard consistency. The vertical extent of the glacial till varies with grade elevation and bedrock elevation. The interpreted overburden thickness over the site area is provided as Figures 2A to F.

2.1.2 Bedrock

Bedrock formations underlying the study area are of Upper to Middle Ordovician age. The uppermost of these is the Queenston Formation of Upper Ordovician age, which gradually overlies the Georgian Bay Formation of Middle Ordovician age. The Queenston Formation is exposed along the Bronte Creek valley walls.

The Queenston Formation is a dark red, low-fissility shale/siltstone with green mottling. The green mottled zones are occasionally harder than the softer red shale (which appears as recessive horizons along the Bronte Creek valley outcrop), possibly indicating a higher carbonate content which is called "limestone" by local convention. However, the Queenston shale within the study area is generally calcareous, and is interbedded with stronger calcareous sandstone and silty bioclastic carbonate (which are observed as the protruding horizons along the Bronte Creek valley outcrop)[1]. Minor amounts of gypsum, in nodules and laminae, are found throughout. These, along with occasional weathered clay seams and partings, indicate the presence of ground water within the bedrock. Bedrock was encountered in ninety-nine (99) of the 104 boreholes. Of these, forty-seven (47) boreholes recovered and logged rock core.

In the Oakville/Burlington Area, the surface of the rock having been scoured and involved by the base of glacial ice, neither Shale Zone III nor IV is present in identifiable form. Where rock core was not retrieved, inferred bedrock was defined based on auger cuttings, samples from split spoons, and drilling observations alone. Inferred bedrock is grouped together with weathered bedrock on the basis that sound bedrock was not directly observed and that inferred bedrock occurs immediately underlying overburden soils, within the weathering zone.

Weathered and/or inferred bedrock was encountered in ninety-nine (99) of the boreholes. The top of weathered/inferred bedrock was encountered at between 0.6 and 12.1 m below grade. Core samples recovered by Terraprobe revealed thicknesses of partially weathered Zone II rock ranging from 0.1 m (BH 7+270) to 7.4 m (BH 4+990). On average, Terraprobe boreholes encountered 3.9 m of weathered bedrock. Weathered bedrock was often not explicitly described in boreholes produced from previous investigations; however, for the purposes of this report, Terraprobe inferred weathered bedrock elevations from other information presented on the borehole logs.

Sound bedrock was observed at depths ranging from 1.3 to 15.4 m below grade. In these boreholes, rock cores were recovered to depths ranging from 4.6 to 70.1 m below grade. The borehole findings observed a thicker layer of overburden along the portion of alignment from Station 7+000 (approximately) to the Kitchen Reservoir shaft. The depth to bedrock within this area is compared with the rest of the alignment in the following table.

[1] Brogly, P.J., Martini, I.P., and Middleton, G.V. (1998). "The Queenston Formation: shale dominated, mixed terrigenous-carbonate deposits of Upper Ordovician, semiarid, muddy shores in Ontario, Canada." Can J. Earth Sci. **35**: 702-719.

Table 2-1: Variations in Bedrock Depth, before and after Stn. 7+000

	Burloak WPP to Stn. 7+000		Stn. 7+000 to Kitchen Reservoir	
	Weathered bedrock, depth (m)	Sound bedrock, depth (m)	Weathered bedrock, depth (m)	Sound bedrock, depth (m)
Average	**2.0**	**5.7**	**11.6**	**13.3**
Standard Deviation	1.0	3.0	n/a	n/a
Minimum	0.6	1.3	11.1	11.2
Maximum	6.4	15.0	12.1	15.4
No. of Boreholes	97	45	2	2

It should be noted that Boreholes 7+165x and 7+250x were also advanced to the east of Stn. 7+000 in the vicinity of the Kitchen Reservoir shaft. These boreholes were originally scheduled for a previously proposed alignment that included an open cut section along Trawden Way (since superseded). These shallow boreholes were advanced to depths of 9.8 and 8.2 m below grade, respectively, and did not encountered bedrock.

Laboratory test data was compiled from the Terraprobe investigation and the previous investigations, and is summarized in the following table. A profile of UCS data versus elevation within the tunnel zone is provided as Figure 3 (note that the figure is not to scale in the horizontal direction).

Table 2-2: Summary of Laboratory Test Results, Bedrock

	Queenston Formation				
	UCS (MPa)	Bulk density,γ (kN/m^3)	Young's modulus, E (GPa)[2]	Point load[3] index (MPa)	
				Axial, PL_A	Diametral, PL_D
Average	**22.1**	**25.8**	**4.9**	**35.4**	**12.5**
Standard Deviation	12.9	0.3	2.2	24.7	13.7
Minimum	1.0	24.5	0.9	5.0	2.0
Maximum	101.5	26.5	8.9	156.0	114.0
No. of Tests	240	210	15	135	93
No. of Boreholes	33	24	5	8	5
	Georgian Bay Formation				
Average	**20.5**	**25.9**	**6.9**	**30.9**	**n/a**
Standard Deviation	5.8	0.2	n/a	10.5	n/a
Minimum	13.8	25.6	5.3	10.0	n/a
Maximum	27.6	26.3	8.4	55.0	n/a
No. of Tests	6	7	2	53	n/a
No. of Boreholes	2	2	1	3	n/a

[2] Determined from UCS laboratory testing
[3] Point load index values are reported as inferred UCS values (see Coffey Geotechnics, Sept 2011, Coffey Geotechnics, Nov 2011, and Geo-Canada 2005)

As part of the 2012-2013 Terraprobe investigation, the Young's modulus of the in situ rock mass was measured using a Probex borehole dilatometer (rock pressuremeter). Six tests were conducted within one of the boreholes (BH 6+390). Tests were carried out along a profile of depths close to the proposed tunnelling zone (since revised). The in situ Young's modulus of the Queenston Formation ranged from 1.4 to 6.0 GPa (on average 4.5 GPa). The results are summarized as follows.

Table 2-3: Young's modulus, from Probex dilatometer (Terraprobe 2013, BH 6+390)

Depth / Elevation, m (BH 6+390)	In situ Young's modulus, E_{PM} (GPa)	RQD (%)
47.2 / Elev. 92.0 m	6.0	100
49.4 / Elev. 89.8 m	5.8	78
51.8 / Elev. 87.4 m	1.4	72
53.3 / Elev. 85.9 m	2.2	71
54.9 / Elev. 84.3 m	5.7	95
56.9 / Elev. 82.3 m	5.9	95

The Queenston Formation is composed of weak shale beds ("shale") interbedded with harder calcareous beds ("limestone"). Test results from the Terraprobe investigation were sorted and compared by rock type, to determine the variation of each parameter according to rock type. Profiles of UCS and Bulk Density are provided against elevation (Figures 6 and 7). The results are summarized in the following table. A histogram representation of the data variability is provided as Figure 8.

Table 2-4: Mechanical properties versus rock type (Terraprobe 2013), Queenston Formation

Queenston Formation Rock Types	UCS (MPa)			Bulk Density (kN/m^3)		
	Average	Standard Deviation	Number of tests	Average	Standard Deviation	Number of tests
Limestone	**22.8**	10.8	52	**25.9**	0.2	53
Shale	**19.4**	10.0	97	**25.9**	0.2	97
Shale & Limestone[4]	**23.2**	11.9	37	**25.8**	0.3	37

2.2 Ground Water

Ground water observations were made in each of the boreholes as they were drilled and after completion, in all of the studies reviewed. It should be noted that ground water levels are subject to fluctuation due to seasonal changes, surface runoff, and storm events.

[4] Both rock types present in sample as tested.

In 2012-2013, Terraprobe installed thirteen (13) monitoring wells in seven (7) boreholes. Monitoring wells were installed in boreholes filled with drill fluid; thus, unstabilized water levels were not measured. The stabilized ground water levels were measured after the drill fluid was purged from the well, and are summarized below.

Table 2-5: Stabilized Water Level Measurements

Borehole No.	Depth of borehole (m)	Screen Elevation, midpoint (m)	Strata Screened Within	Last Recorded Water Level in Well	
				Water Level (Depth / Elev, m)	Date (dd-mon-yy)
BH 1+200	21.6	89.3	Overburden - Bedrock Interface	3.9 / 88.2	12-Feb-13
BH 1+200	21.6	78.4	Bedrock	5.1 / 87.1	12-Feb-13
BH 2+425	35.1	100.7	Overburden - Bedrock Interface	3.2 / 100.6	12-Feb-13
BH 2+425	35.1	77.5	Bedrock	5.3 / 98.4	12-Feb-13
BH 2+640	38.2	76.4	Bedrock	8.8 / 96.8	12-Feb-13
BH 3+065	41.2	109.5	Overburden - Bedrock Interface	2.2 / 110.4	12-Feb-13
BH 3+065	41.2	79.9	Bedrock	6.8 / 105.8	12-Feb-13
BH 4+495	59.2	128.4	Overburden - Bedrock Interface	2.0 / 129.4	12-Feb-13
BH 4+495	59.2	80.8	Bedrock	24.2 / 107.3	12-Feb-13
BH 7+145	61.2	129.7	Overburden	4.3 / 128.4	7-Jan-13
BH 7+145	61.2	80.3	Bedrock	15.0 / 117.7	7-Jan-13
BH 7+270	48.4	124.0	Overburden - Bedrock Interface	8.4 / 124.4	13-Mar-13
BH 7+270	48.4	96.5	Bedrock	37.6 / 95.2	13-Mar-13

The hydraulic conductivity of the sands and gravels encountered east of Stn. 7+000 was estimated from grain size distribution curves (Appendix B) of samples recovered from these strata. The hydraulic conductivity for these deposits was estimated to be around 10^{-4} cm/s.

Rising head tests were conducted by Terraprobe at six monitoring well locations (BH 1+200, BH 2+425, BH 3+065, BH 4+495, BH 7+145, and BH 7+270). The tested monitoring wells were installed within the Queenston Formation bedrock. The analyses were completed using the Bouwer and Rice method. In situ hydraulic conductivity (rising head test) results generally ranged from 10^{-7} to 10^{-9} cm/s, with the notable exception of BH 1+200 which was screened at a relatively shallow depth (12 ± m below grade) and measured an in situ hydraulic conductivity of 10^{-4} cm/s. The results of the hydraulic conductivity analyses are summarized in the following table.

Table 2-6: In Situ Permeability, Queenston Formation

Monitoring Well	Well Screen Depth (m BG)	Well Screen Elevation (m)	Hydraulic Conductivity (rising head test, cm/s)
BH 1+200	12.2 to 15.2 ±	80.0 to 76.9 ±	1×10^{-4}
BH 2+425	24.7 to 27.7 ±	79.1 to 76.0 ±	2×10^{-8}

Monitoring Well	Well Screen Depth (m BG)	Well Screen Elevation (m)	Hydraulic Conductivity (rising head test, cm/s)
BH 3+065	31.1 to 34.3 ±	81.5 to 78.3 ±	7 x 10^{-8}
BH 4+495	49.1 to 52.1 ±	82.4 to 79.3 ±	3 x 10^{-7}
BH 7+145	50.9 to 54.0 ±	81.9 to 78.8 ±	3 x 10^{-7}
BH 7+270	35.1 to 38.1 ±	97.8 to 94.7 ±	2 x 10^{-9}

Hydraulic conductivity testing by packer test was conducted by Coffey Geotechnics (2011) and Geo-Canada (2005) at fifty-five (55) intervals in both formations, within seven (7) boreholes. Testing pressures varied in accordance with the position of the piezometric water level[5]. The hydraulic conductivity results from previous investigations are presented on the respective borehole logs, and are summarized as follows.

Table 2-7: Hydraulic Conductivity from Packer Testing

Borehole	Testing Intervals			Range of hydraulic conductivity (from Packer Testing, cm/s)[6]	Formation
	Depth	Elevation	Intervals		
COF-BC-1	6.1 – 54.9 m	Elev. 138.4 – 89.6 m	7	9 x 10^{-7} to 2 x 10^{-6}	Queenston
COF-BC-2	36.6 – 51.5 m	Elev. 105.8 – 90.9 m	3	5 x 10^{-7}	Queenston
COF-BC-3	33.5 – 47.9 m	Elev. 105.1 – 90.7 m	3	2 x 10^{-7} to 3 x 10^{-7}	Queenston
COF-BC-4	16.2 – 46.0 m	Elev. 122.9 – 93.1 m	5	5 x 10^{-7} to 1 x 10^{-4}	Queenston
GC-A	4.0 – 28.5 m	Elev. 85.9 – 61.4 m	6	3 x 10^{-6} to 1 x 10^{-3}	Queenston
GC-B	6.3 – 27.9 m	Elev. 84.7 – 63.1 m	5	6 x 10^{-6} to 2 x 10^{-3}	Queenston
GC-C	5.2 – 28.0 m	Elev. 84.1 – 61.3 m	6	3 x 10^{-6} to 2 x 10^{-3}	Queenston
GC-A	28.5 – 61.2 m	Elev. 61.4 – 28.7 m	8	2 x 10^{-7} to 3 x 10^{-6}	Georgian Bay
GC-B	30.0 – 55.2 m	Elev. 61.1 – 35.8 m	6	9 x 10^{-7} to 3 x 10^{-6}	Georgian Bay
GC-C	28.0 – 53.6 m	Elev. 61.3 – 35.7 m	6	7 x 10^{-7} to 9 x 10^{-6}	Georgian Bay

2.3 Bronte Creek

The Bronte Creek watercourse has been identified as an area of importance, through the Risk Management Workshop process. Four (4) boreholes were advanced at locations within the valley that intersect the proposed alignment. The boreholes within Bronte Creek valley (BHs 5+855, 5+885, 5+900,

[5] See Coffey Geotechnics (Nov 2011) and Geo-Canada (2005) for further details.
[6] Data do not include tests where no water take was recorded.

and 5+935) were advanced through overburden by hand auger, and cored using restricted-access rock coring equipment. The acquired overburden and bedrock information in this area is similar to the bedrock information secured across the site.

3.0 SHAFT DESIGN

Five shafts are proposed along the watermain alignment, as summarized in Table 1-1. The construction shafts will be sunk through up to 11 ± m of overburden material. All of the shafts will penetrate weathered rock of the Queenston Formation and continue well into sound bedrock, terminating at their respective design elevations.

Table 3-1: Summary of Subsurface Depths at Shaft Locations

		Overburden		Weathered bedrock encountered at		Sound bedrock encountered at	
Station	**Shaft Location**	**Earth fill, thickness (m)**	**Native soils, thickness (m)**	**Depth (m)**	**Elevation (m)**	**Depth (m)**	**Elevation (m)**
1+000	**Burloak WPP**	1.5 to 2.3	0.7 to 1.8	2.3 to 3.8	87.5 ±	7.8 to 8.0	82 ±
3+078	**Proposed Zone 2 BPS**	0.1 to 1.5	0.7 to 1.5	0.8 to 3.0	110 to 114 ±	4.2 to 6.0	108 to 109 ±
4+981	**Ontario Parks**	0 to1.0	0 to 1.5	1.0 to 1.5	136 ±	8.4 to 9.2	129 ±
7+300	**Kitchen Reservoir**	1.5	9.6	11.1	122 ±	11.2	122 ±

* Overburden not observed due to daylighting, inferred to be earth fill.

The overburden soils and the weathered rock will need to be shored when constructing the shafts. Alternatively, the overburden soils may be cut back to a stable inclination, space permitting. OHSA safe slopes for open cut excavations are provided in Section 6.1. The underlying sound bedrock of the Queenston Formation is effectively stable in a vertical cut.

It is envisioned that the shafts will be extended through the overburden soils using typical caisson augering equipment, in which case a liner can be used to retain the overburden soils and prevent groundwater infiltration. The remaining excavation can be made in sound bedrock in a vertical cut.

Alternatively, the shafts can be advanced within a shoring system consisting of interlocking drilled caissons socketed 2 metres into the bedrock. The shoring system must be designed by a professional engineer. This will shore the excavation and constitute the primary ground water barrier at the shaft perimeter.

3.1 Shoring Considerations

A summary of the proposed shaft locations is as follows:

- The proposed shaft (comprising 2 shafts at 3 m diameter) at Burloak WPP is bounded to the north by Rebecca Street, to the south by the Burloak WPP, and to the east and west by public lands.

- The proposed 15 m-diameter Zone 2 BPS shaft is bounded to the east by Burloak Drive, to the south by an existing gas station, to the north by an existing parking lot at 945 Syscon Road, and to the west by private lands also belonging to 945 Syscon Road.

- The proposed 3 m-diameter shaft at the Ontario Parks entrance will be advanced on the east side of Burloak Drive, within the northbound lanes and/or the road shoulder. The shaft is bounded to the east by a swale and an underground Enbridge oil pipeline and to the north, south, and west by Burloak Drive.

- The proposed shaft (comprising 2 shafts at 3 m diameter) at Kitchen Reservoir is bounded to the west by Colonel William Parkway (offset about 5 to 10 m), and by Kitchen Reservoir lands elsewhere.

Where they cannot be sloped, the sides of the shaft excavation may be supported through the overburden using conventional soldier pile and lagging walls. Depending on Enbridge's requirements for their pipeline, a rigid shoring system may be required along the east side of the shaft excavation at the Ontario Parks shaft. This would be achieved using a caisson shoring wall.

3.2 Shoring Support

The anticipated shored excavations will extend through shallow surficial soils and weathered bedrock (less than 3.8 m) in most locations, with the exception of the shaft at Kitchen Reservoir.

3.2.1 Earth Pressure Distribution

Where multiple supports are used to support an excavation, research has shown that a distributed pressure diagram more realistically approximates the earth pressure on a shoring system of this type, when restrained by pre-tensioned anchors. The multi-level supported shoring can be designed based on an earth pressure distribution consisting of a trapezoidal pressure distribution with a maximum pressure defined by:

$P = 0.80\,K\,(\gamma h+q)$ (for cohesive soils – majority of site)
$P = 0.65\,K\,(\gamma h+q)$ (for cohesionless soils – Kitchen Reservoir)

where: **P** = the horizontal pressure at depth, h (kPa)

K = the earth pressure coefficient
h = depth below surface (m)
γ = the bulk unit weight of the soil (kN/m^3)
q = the complete surcharge loading (kPa)

3.2.2 Soldier Pile Toe Design

Where the excavation penetrates the bedrock, the rock excavation is nominally self-supporting in a vertical face, provided the rock bedding is horizontally oriented. The rock induces no pressure on shoring systems that require structural support.

The maximum factored geotechnical resistance at ULS for the design of the soldier pile toe embedded in the bedrock of Queenston Formation is 4 MPa. The maximum factored lateral capacity of the bedrock is 1000 kPa.

It needs to be noted that the bearing capacity of the rock is predicated on intact rock. The exposed Queenston Formation deteriorates with time. Exposed excavation faces have been found to flake and recede as much as 300 mm with 6 months exposure. This recession generally takes the form of coin-sized shale particles dropping from the face on a constant basis. The deteriorated rock loses internal integrity and bearing capability.

Where shoring systems are made perched in the rock above the excavation base, great care and consideration must be given to providing protection and support for the rock in the area of influence directly beneath the base of the caisson or soldier pile toe as appropriate. It has become accepted practice in the local shoring design community to leave a minimum one metre wide shelf to carry soldier pile toes perched above the level of the excavation base and thereby minimize rock protection and support requirements. Regardless of the approach taken in the design it is required to drape and bolt a steel screen on the excavation face to collect and direct spalling rock fragments to the base of the excavation in a way that protects workers in the rock shaft.

It needs to be noted that there are zones of material in the subsurface soils which are sufficiently wet and permeable such that augured borings for soldier piles made into these soils will likely be unstable. This is particularly the case at Kitchen Reservoir. In these cases, it will be necessary to advance temporarily cased holes to prevent excess caving during the soldier pile installations.

3.2.3 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.

Alternatively, rock anchors made in the Queenston Formation are nominally designed using a maximum factored design bond stress of 400 kPa at ULS. Higher bond stresses are possible but performance testing of anchorages on a site by site basis is required. These anchors would be made with a continuous flight auger. The anchors can be installed and stressed as excavation proceeds to minimizing the potential for relaxation in the supported soil. It should be noted that it was necessary to use lower values on some sites after load testing the first of the anchorages yielded lower capacities. There is a risk in adopting a higher design value, since the shoring anchors will have to be reassessed if the first load test does not prove out. Use of the higher value saves material if the higher value can be demonstrated successfully. There are practical limits to the length of anchor that can be drilled with conventional equipment which must be recognized. If the shoring design is based on the most optimistic design adhesion value, the practical implications of a test that proves a lesser adhesion must be recognized. The design must have to have sufficient flexibility to accommodate a variation in the design adhesion value.

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 is not anticipated at Kitchen Reservoir, given that the adjacent lands are owned by the Region. If soil anchors do extend beyond the Region's property limits, this requires the consent of the adjacent land owners, expressed in encroachment agreements. The contractor is required to obtain all necessary permits to suit their purposes.

4.0 TUNNEL DESIGN

4.1 Tunnel Design Considerations

Experience in the Greater Toronto Area over a number of years has indicated tunnels of the proposed diameter, when made by hand/mechanical excavation or tunnel boring machine, are stable with limited roof slabbing or coning, when primary support to the tunnel crown is provide promptly in conjunction with the advance of the tunnel. Primary support is nominally in the form of steel sets with a series of steel ribs and spanning media such as timber or mesh. The purpose of this primary support is to maintain the rock in intact form as much as practically possible thereby preserving the strength and integrity of the rock mass.

The indications from the investigation programme are the tunnel alignment will be at sufficient depth to be made well into undisturbed rock, with at least 8 m of bedrock cover. The contractor's programme for primary support will have to consider the measures to be implemented when the tunnel crew finds that the drilling for sets is meeting less resistance, indicative of discontinuities or weak rock in the ceiling profile.

4.2 Ground Water and Gas

Terraprobe has prepared a Hydrogeological Report for this project under separate cover.

The tunnel will be made beneath the prevailing ground water level. The investigation found no specific fractured zones of rock expected to yield significant volumes of water. Experience with similar works suggests that the Queenston Formation joints are sufficiently tight so as to generally preclude the free flow of ground water, but occasionally fractured limestone or dolostone layers contain limited stored volumes.

Rising head tests, conducted in wells screened at least 24 m below grade in sound Queenston Formation bedrock, measured in situ hydraulic conductivities ranging from 10^{-7} to 10^{-9} cm/s. A rising head test conducted in a well screened adjacent to a creek in relatively shallow bedrock (BH 1+200, screened at around 11 m below grade) measured an in situ hydraulic conductivity of 10^{-4} cm/sec.

Although not observed in any of the boreholes, the possibility of gas emissions from the Queenston Formation needs to be recognized and the tunnels must be monitored and vented appropriately. It is known that the Queenston Formation produces nominally small quantities of gas when penetrated. While there was no specific indication of gas emissions from the borings made in this investigation, the potential for gas emissions from this formation is recognized as a design issue to be addressed.

It should be noted that the underlying Georgian Bay Formation has been known to issue gases when penetrated. There are instances where both methane and hydrogen sulphide gas emissions have been detected in excavations made in the Georgian Bay Formation.

4.3 Time-Dependant Deformation

The Queenston Formation has been reported in the literature to have locked-in residual horizontal stresses. Excavation in the rock results in relief of these in situ stresses. The relief takes place over time with a subsequent extension of the cut rock face. This phenomenon manifests itself as apparent creep of the rock face. On this basis there is a significant reduction in stress realized when there is a delay on the order of 90 to 120 days prior to permanent lining. This time period can be shorter where previous adjacent excavations or natural features (e.g. Bronte Creek valley) have penetrated the rock and provided some measure of primary stress relief above and beside the tunnel zone.

There may also be some swelling of the rock unrelated to stress. However, the effects are indistinguishable for practical purposes. There are recorded measurements of time dependant stress relief rates in published literature taken from measurements in actual tunnels made in the bedrock in the Greater Toronto Area. These rates are different from those observed and reported in excavations.

The component of rock swelling that is unrelated to stress relief may be measured in the laboratory as described by Lo et al.[7]. It is possible to measure the swell potential of rock samples under no confining pressure (i.e. "free swell") as well as under a confining pressure. According to Hawlader et al.[8], the swelling strain at time *t* may be expressed as:

$$\varepsilon_i(t) = m_{i(s)} log\left(\frac{t}{t_0}\right)$$

where *m* is the slope of the straight line of strain versus the logarithm of time measured between 10 and 100 days (as per Lo et al., 1978), the subscript *i* represents the direction of swelling, and the subscript *s* denotes the stress applied. Swelling begins at reference time t_0. It should be noted that the non-linear portion of the swelling curve between 0 and 10 days is attributed in the literature to the specimen reaching equilibrium after being moved from in situ to laboratory conditions (Hawlader et al., 2003).

Samples of rock from the Queenston Formation were selected in the field based on their proximity to the proposed tunnel depth (since revised). They were sealed with foil and wax to preserve their in situ pore water properties. The samples were then transferred to an external laboratory, which conducted the swell potential testing as per Madsen[9] and Lo et al.[10]. Eight (8) sealed rock core samples were submitted for

[7] K.Y. Lo et al., *Time-dependent deformation of shaly rocks in Southern Ontario,* Can. Geotech. J. **15** 537-547 (1978).
[8] Hawlader, Lee, and Lo, *Three-dimensional stress effects on time-dependent swelling behavior of shaly rocks*. , Can. Geotech. J. **40** 501-511 (2003).
[9] F.T. Madsen, *Suggested methods for laboratory testing of swelling rocks*, ISRM **36** 291-307 (1999).
[10] K.Y. Lo, R.S.C. Wai et al., *Time-dependent deformation of shaly rocks in Southern Ontario,* Can. Geotech. J. **15** 537-547 (1978).

free swell testing. Of these, six were tested in fresh water and two were tested in a saline solution (200 g NaCl/L). The free swell potential testing method consisted of submerging a cut core sample (trimmed to a cube) in either distilled water (6 samples) or saline solution (200 g/L NaCl, 2 samples), and measuring the axial strain in three directions. The saline concentration agrees with Queenston shale pore water salinity measurements made by Lo and Lee[11].

Free swell testing on these samples measured a rate of vertical deformation of 0.33 to 0.75% strain per log cycle time (10 to 100 days) when immersed in distilled water. Conversely, free swell testing on samples immersed in saline solution (200 g NaCl/L) measured a rate of vertical deformation of between 0.06% and 0.08% strain per log cycle time (10 to 100 days). This could be interpreted as an indication that pore water salinity does play a role in Queenston Formation swell potential. The results of the swell testing are tabulated as follows.

Table 4-1: Summary of Free Swell Potential Results, Queenston Formation

Immersed in	Direction of Strain	No. of Tests	Strain (%) per log cycle time (10 to 100 days)		
			Minimum	Maximum	Average
Distilled Water	vertical	5	0.33	0.75	0.54
	horizontal	10[12]	-0.06	0.22	0.12
Saline solution (200 g NaCl/L)	vertical	2	0.06	0.08	0.07
	horizontal	4	0.03	0.12	0.08

Note: one sample (distilled water, free swell) still in progress

Regardless of whether the sample was immersed in fresh or saline water, swelling generally started to taper off after 90 days, and the amount of swelling after 120 days was negligible.

For comparative purposes, the free swell rate of deformation (vertical) in Queenston Formation shale has been reported as 0.14% per log cycle of time (9 tests) by Lo et al. (1978), and as 0.44% per log cycle of time (11 tests ranging from 0.37 to 0.54%) by Lo and Lee[13]. Based on comparison data, the Queenston Formation shale at this site may be said to have a slightly higher free swell potential than recorded in the literature.

The magnitude of swelling is anisotropic due to the inherent structure of the shale. Free swell measurements made in orthogonal directions on the samples recovered from this investigation show that swell potential is 3 to 5 times higher in the vertical direction than in the orthogonal direction. This agrees with measurements reported by Lo et al (1978).

[11] K.Y. Lo and Y.N. Lee, *Time-dependent deformation of Queenston shale*, Can. Geotech. J. **27,** 461-471 (1990).
[12] Two horizontal directions per sample
[13] K.Y. Lo and Y.N. Lee, *Time-dependent deformation of Queenston shale*, Can. Geotech. J. **27,** 461-471 (1990).

Swell pressure testing conducted on samples of the Queenston Formation measured the pressure required to restrain the sample from swelling. Tests conducted in fresh water measured the restraining pressure at 95 to 156 kPa, which reduced the vertical swell to effectively nil (within ± 0.1% strain, dependent on the response of the testing equipment than on the range of expected strains). The implication is that a relatively small amount of applied stress to the rock mass may be sufficient to counteract the swell potential.

4.4 Permanent Installations

It is understood that the proposed watermain tunnel is to be constructed with no tunnel liner, with the exception of the portion of tunnel underlying the QEW, which is to be lined. The tunnel's annular space is to be filled with cellular grout.

The design of permanent installations must take into account the time-dependant deformation characteristics of the bedrock. If permanent installations are installed shortly after excavation for the tunnel or in conjunction with the tunnel advance, the swell of the rock as stress is relieved will place significant pressure on the installation. It is understood that the tunnel is to be left open for at least 60 days after it is excavated, which should be enough to limit any anticipated time-dependant deformation.

The Young's modulus of the in situ Queenston Formation was measured in BH 6+390. The in situ modulus ranged from 1.4 to 6.0 GPa, between Elev. 82.3 to 90.0 m.

In the completed tunnel the maximum residual stress would be expressed in the spring-line of the tunnel diameter where the unbalanced horizontal stress is a maximum. The horizontal and tangential pressure on the permanent tunnel lining is a function of the vertical in situ pressure which is given by:

$$\boldsymbol{P = \gamma(h - h_w) + \gamma' h_w + q + \gamma_w h_w}$$

where,

- $\mathbf{P}$ = the horizontal pressure at depth, $\mathbf{h}$ (m)
- $\mathbf{h_w}$ = the depth below the ground water level (m)
- $\boldsymbol{\gamma}$ = the bulk unit weight of soil, (kN/m^3)
- $\boldsymbol{\gamma'}$ = the submerged unit weight of the exterior soil, (γ - 9.8 kN/m^3)
- $\mathbf{q}$ = the complete surcharge loading (kPa)

4.5 Infrastructure Crossings

The proposed watermain alignment will cross under several existing noteworthy utilities, infrastructure, and natural features as indicated in the following subsections. The measurements conservatively assume a tunnel diameter of 3.05 m, although the allowable minimum tunnel diameter is 2.44 m (as discussed in the PDR).

4.5.1 Trans-Northern Pipeline

The proposed tunnel alignment crosses beneath the Hydro One Corridor located north of Prince William Drive, at around Stn. 2+450. The Trans-Northern Pipeline is also located within this easement at around 2 m below grade. The proposed tunnel springline at this crossing is at around Elev. 76.5 ± m, which is approximately 27.5 m below grade. This is equivalent to about 9 tunnel diameters. The tunnel will be located about 8 tunnel diameters below the top of bedrock.

Given that the at-grade utility and shallow pipeline are far outside the zone of influence of tunneling activities, additional monitoring is not recommended.

4.5.2 CNR Tracks

The proposed tunnel alignment crosses beneath the CN Rail (“CNR”) Tracks located north of Prince William Drive, at around Stn. 2+600. In this location, the tunnel springline is at around Elev. 76.5 ± m, which is approximately 31 m below grade (equivalent to about 10 tunnel diameters). The tunnel will be located about 9 tunnel diameters below the top of bedrock.

Although the CNR tracks in this location are far outside the zone of influence of tunneling activities, additional settlement monitoring of the railway tracks may be conducted as per CN requirements. Recommendations for this monitoring work are provided as a Non-Standard Special Provision, as Appendix A. The Settlement Monitoring Plan provided in Figure 9 indicates the approximate locations of monitoring instruments and provide typical instrument details.

4.5.3 MTO Lands

The proposed tunnel alignment crosses beneath MTO lands (the QEW) at around Stn. 3+400. In this location, the tunnel springline is at around Elev. 77.0 ± m, which is approximately 42 m below grade (equivalent to about 13 tunnel diameters). The tunnel will be located about 12 tunnel diameters below the top of bedrock in this location.

Although the QEW is far outside the zone of influence of tunneling activities, additional settlement monitoring of the roadway may be conducted as per MTO requirements. Recommendations for this monitoring work are provided as a Non-Standard Special Provision, as Appendix A. The Settlement Monitoring Plan provided in Figure 10 indicates the approximate locations of monitoring instruments and provide typical instrument details.

4.5.4 Enbridge Pipeline

The proposed alignment crosses underneath an existing 508 mm oil pipeline within the Enbridge right-of-way at Upper Middle Road and Burloak Drive, at around Stn. 5+420. The pipeline is assumed to be founded at around 2 metres below grade. The minimum amount of rock cover overlying the proposed tunnel springline is approximately 55 m, or 18 tunnel diameters. The tunnel will be located about 17 tunnel diameters below the existing pipeline in this location.

Given that the pipeline is far outside the zone of influence of tunneling activities, additional monitoring is not recommended.

4.5.5 Bronte Creek Provincial Park

The proposed tunnel alignment passes underneath Bronte Creek Provincial Park starting at around Stn. 5+500, and runs beneath it for approximately 1.4 km before exiting at around Stn. 6+900. Bronte Creek (Elev. 105 ± m) intersects the middle of the park crossing at around Stn. 5+900. At its nearest point beneath Bronte Creek (Stn. 5+900), the proposed tunnel springline is at around 16 m below grade (equivalent to around 5 tunnel diameters). The minimum amount of total bedrock cover and sound rock cover overlying the proposed tunnel obvert is approximately 13.5 m and 10 m in this location, respectively. Elsewhere, the minimum amount of sound rock cover overlying portions of the alignment not under Bronte Creek is at least 40 m.

Given that the majority of Bronte Creek Provincial Park is far outside the zone of influence of tunneling activities, additional monitoring is not recommended overall. Bronte Creek itself will be subject to additional monitoring as determined by the appropriate regional and/or provincial authorities.

5.0 GEOTECHNICAL DESIGN

5.1 Foundation Design Parameters

The proposed watermain construction will comprise two shallow valve chambers at Burloak WPP, and no valve chambers at Kitchen Reservoir. It is understood that the valve chambers will be constructed to around 5 m below grade. The nominal foundation depth is therefore around 5.5 m.

At the Burloak WPP, inferred bedrock was encountered at around 3.8 m below grade. Conventional spread footing foundations made to on the weathered bedrock may be designed using a maximum geotechnical resistance at ULS of 3500 kPa. The maximum net allowable geotechnical reaction at SLS is 2500 kPa, for an estimated total settlement of 25 mm.

The minimum width of continuous strip footings supported on the undisturbed soils or inferred bedrock must be 500 mm and the minimum size of isolated spread footings must be 800 mm x 800 mm on native soils regardless of loading considerations, in conjunction with the above recommended geotechnical resistance. Settlement will occur as load is applied and is linear elastic and non-recoverable. Differential settlement is a function of spacing, loading and foundation size.

The nominal design earth cover for frost protection of foundations or grade beams exposed to ambient environmental temperatures is 1.2 metres for these locations.

5.2 Open Cut Sections

The watermain construction is to include open cut sections for connection installations at Burloak WPP and Kitchen Reservoir. The depth to invert will be around 4 to 6 metres below grade at Burloak WPP, and around 3 to 7 m below grade at Kitchen Reservoir.

5.2.1 Excavations

Trench excavations through the glacial till and upper 2 m of the bedrock are typically carried out using conventional excavators with ripper teeth. The glacial till, however, can contain cobbles and boulders and a contingency should be allocated to the costs and risk associated with these costs. Some ground water seepage may be encountered in the excavations, but can likely be managed by pumping from conventional filtered sumps located as required in the excavations.

Excavations made for the shallow connections at the Burloak WPP will penetrate 1 to 2 m into the inferred bedrock, which is assumed to be Zone II (partially weathered) bedrock. It is possible that sound bedrock may be encountered at this depth. The sound bedrock of the Queenston Formation is a rippable rock that typically does not require blasting. Effective techniques in this formation include the use of hoe ramming equipment, rippers, and line drilling.

Excavations made for the shallow connections at Kitchen Reservoir will generally be in the Halton Till at above Elev. 126.7 m, with the connection to the Kitchen Reservoir made lower. The cohesionless strata (sands and gravels) at this location were observed at elevations ranging from Elev. 126.7 to 125.3 m, and are moist to wet. The ground water elevations range from Elev. 124 to 125 m. The ground water level is close to the elevation of the top of the sands and gravels.

Open cut excavations made above the sands and gravels are not expected to yield significant seepage; however, excavations that penetrate the lower cohesionless gravel and sand will yield free-flowing water, and will need to be positively dewatered ahead of the excavation. It is recommended that consideration be given to conducting trial excavations (test pits) to assess the stability of the excavation and ground water influx once the design details of the development are finalized (including the invert elevations). This information would help finalize the requirements for ground water control and dewatering. A professional dewatering contractor should be consulted to review the subsurface conditions and to design a site specific dewatering system if dewatering is necessary. It is the dewatering contractor's responsibility to make an assessment of the factual data and to provide recommendations on dewatering system requirements.

Excavations must be carried out in accordance with the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The OHSA soil types and maximum recommended minimum slope inclinations are summarized in Section 6.1. As such, temporary support or protection must be provided for excavations that are steeper than provided for in the OHSA, within the overburden and upper weathered zone of the bedrock. Any shoring and bracing must be designed to resist the at-rest earth pressure in the surrounding soil mass. Excavations within the bedrock may be cut at a near vertical inclination provided that regular monitoring of the excavation is conscientiously performed by a professional geotechnical engineer. Some deterioration (ravelling) of the rock faces is expected in the bedrock excavations that remain open for long periods of times.

5.2.2 Bedding

The existing clayey silt glacial till and bedrock at each of the sites 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. All granular bedding must be compacted to a minimum of 95 percent of Standard Proctor maximum dry density (SPMDD. Where disturbance of the trench base have occurred, such as due to ground water seepage or construction traffic, the disturbed soils must be excavated and replaced with suitably compacted granular fill.

5.2.3 Backfill

Excavated existing clean, inorganic soils can generally be re-used as backfill provided the moisture content of these materials is within optimum or 2 percent greater of optimum to ensure adequate compaction, and the trenches are wide enough to accommodate a large sheepsfoot compaction roller. If narrow trenches are excavated then use of aggregate fill (such as OPSS 1010 Granular B) is required if there is to be post-construction grade integrity. The utility trench backfill must be compacted to at least 98% of SPMDD.

Where backfill underlies pavement areas, backfill must be compacted to at least 98% of SPMDD. It needs to be noted that post-compaction settlement of fine grained fills on the order of ½ to 1 percent of total height are common, even when adequately placed to specified compaction. It is best to schedule deep fill placement as far in advance of finish surfacing as possible for best grade integrity.

It should be noted the moisture content of the site soils within the upper weathered zone may be locally wet of optimum moisture content to compact effectively. In this case, the materials will require either drying and/or mixing with drier material. The native soils are not free draining, and will be difficult to handle and compact should they become wetter as a result of inclement weather or seepage. It can be expected that earthworks will be difficult during wet periods (i.e., spring and fall) of the year. Soils which are or become overly wet as the result of rainfall or seepage may prove difficult to compact, and should be mixed with drier soil, left aside, or wasted. Should construction be conducted during the winter season, it is imperative to ensure that frozen material is not utilized as trench backfill or foundation wall backfill. Foundation backfill must be brought up evenly on both sides of foundation walls not designed to withstand lateral pressures.

The excavated material at Burloak WPP will in part consist of shale. Even with stringent controls and measures to condition the excavated shale to an optimum state for placement and compaction, as well as careful monitoring of lift thicknesses and compaction effort, significant post-construction settlement should be expected. This is due to the natural degradation of crushed shale into clay, when subjected to weathering and ground water percolation. Therefore, excavated bedrock must not be used as trench backfill.

The excavated cohesionless sands and gravels at Kitchen Reservoir are potentially suitable for re-use as engineered backfill when the moisture content has been reduced by dewatering.

5.3 Earth Pressure Design Parameters

The parameters used in the determination of earth pressures acting on retaining walls are defined below.

Parameter	Definition	Units
φ	internal angle of friction	degrees
γ	bulk unit weight of soil	kN / m^3
K_a	active earth pressure coefficient (Rankin)	dimensionless
K_o	at-rest earth pressure coefficient (Rankin)	dimensionless
K_p	passive earth pressure coefficient (Rankin)	dimensionless

The appropriate values for use in the design of structures subject to unbalanced earth pressures at this site are tabulated as follows:

Table 5-1: Earth Pressure Design Parameters

Area	Stratum/Parameter	φ	γ	K_a	K_o	K_p
Burloak WPP Shaft (Stn. 1+000)	Compact Granular Fill Granular 'B' (OPSS 1010)	32	21	0.31	0.47	3.26
	Existing Earth Fill	28	19	0.35	0.52	2.88
	Clayey Silt Till	34	21	0.28	0.44	3.54
	Queenston Formation Bedrock	26	26	n/a		
Zone 2 BPS Shaft (Stn. 3+078)	Compact Granular Fill Granular 'B' (OPSS 1010)	32	21	0.31	0.47	3.26
	Existing Earth Fill	28	19	0.35	0.52	2.88
	Silty Sand	30	19	0.33	0.50	3.00
	Clay Till	34	21	0.28	0.44	3.54
	Queenston Formation Bedrock	26	26	n/a		
Ontario Parks Shaft (Stn.4+981)	Compact Granular Fill Granular 'B' (OPSS 1010)	32	21	0.31	0.47	3.26
	Existing Earth Fill	28	19	0.35	0.52	2.88
	Clayey Silt Till	34	21	0.28	0.44	3.54
	Queenston Formation Bedrock	26	26	n/a		
Kitchen Reservoir Shaft (Stn. 7+300)	Compact Granular Fill Granular 'B' (OPSS 1010)	32	21	0.31	0.47	3.26
	Existing Earth Fill	28	19	0.35	0.52	2.88
	Clayey Silt Till	35	21	0.27	0.43	3.69
	Gravel and Sand	36	20	0.26	0.41	3.85
	Queenston Formation Bedrock	26	26	n/a		

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

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

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

where,

P = the horizontal pressure at depth, **h** (m)
K = the earth pressure coefficient
$\mathbf{h_w}$ = the depth below the ground water level (m)
γ = the bulk unit weight of soil, (kN/m^3)
γ' = the submerged unit weight of the exterior soil, (γ - 9.8 kN/m^3)
q = the complete surcharge loading (kPa)

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall that would otherwise act in conjunction with the earth pressure, this equation can be simplified to:

$$\boldsymbol{P = K[\gamma h + q]}$$

To ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure, where the structure is made directly against a shored excavation, drainage is provided by forming a drained cavity with prefabricated drain core material covering the excavation face and designed to discharge collected water into an underfloor drainage system.

The factored geotechnical resistance to sliding of earth retaining structures is developed by friction between the base of the footing and the soil. This friction (**R**) depends on the normal load of the soil contact (**N**) and the frictional resistance of the soil (**tan** φ) expressed as $\boldsymbol{R_f = N\, tan\varphi}$, which is the unfactored resistance. The factored geotechnical resistance at ULS is $\boldsymbol{R_f = 0.8\, N\, tan\varphi}$.

6.0 DESIGN CONSIDERATIONS FOR CONSTRUCTABILITY

6.1 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) broad classifications of soils to stipulate appropriate measures for excavation safety. The overburden soils at this site are summarized according to their OHSA classification in the following table.

Table 6-1: Summary of OHSA Soil Types at Shaft Locations

Soil	Burloak WPP (Stn. 1+000)	Zone 2 BPS (Stn. 3+078)	Ontario Parks (Stn. 4+981)	BCPP (Stn. 5+719)	Kitchen Reservoir (Stn. 7+300)
Earth Fill	Type 3	Type 3	Type 3	Type 3	Type 3
Cohesive Till	Type 2	Type 2	Type 2	Type 2	Type 2
Silts and Sands	n/a	Type 3	n/a	n/a	Type 2 – above water table Type 3 – below water table

Excavations within the bedrock may be cut at a near vertical inclination provided that regular monitoring of the excavation is conscientiously performed by a professional geotechnical engineer.

Where workmen must enter excavations advanced deeper than 1.2 m, the trench walls should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The regulation stipulates maximum slopes of excavation by soil type as follows.

Table 6-2: Summary of OHSA Soil Types and Maximum Slope Inclinations

Soil Type	Base of Slope	Maximum Slope Inclination
1	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
2	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
3	from bottom of trench	1 horizontal to 1 vertical
4	from bottom of trench	3 horizontal to 1 vertical

Minimum support system requirements for steeper excavations are stipulated in the Occupational Health and Safety Act and Regulations for Construction Projects, and include provisions for timbering, shoring and moveable trench boxes.

It must be noted that larger size particles (cobbles and boulders) that are not specifically identified in the boreholes will be present in glacial till soils. Similarly, larger size debris may be found in the fill material. The size and distribution of such obstructions cannot be predicted with borings, because the borehole sampler size is insufficient to secure representative samples of particles of this size.

The glacial till may contain some cohesionless zones that were not specifically identified in the boreholes. It is expected that some amount of ground water will seep into excavations in the short-term.

6.2 Ground Water Control

Terraprobe has prepared a Hydrogeological Report for this project under separate cover, detailing the ground water control considerations.

In general, the volume of water to be anticipated to flow into open excavations is such that temporary pumping from the excavations is expected to suffice for the control of the ground water. The clayey silt till deposit and bedrock of the Queenston Formation beneath the site are of low hydraulic conductivity and preclude the free flow of water. However, from Stn. 7+000 to the end of the proposed alignment, the boreholes encountered a coarse sand and gravel deposit which is sufficiently permeable as to yield free flowing water when penetrated.

The proposed open cut section at Kitchen Reservoir may require significant dewatering to depress the ground water table, to facilitate construction within the watermain within the sands and gravels. Dewatering will take some time to accomplish prior to the start of excavation. The dewatering will require a Permit to Take Water from the Ministry of Environment.

Without prior positive dewatering, the subgrade will become weak and lose its integrity to support the watermain. Consideration should be given to install a skim coat of lean concrete (mud-slab) in conjunction with positive dewatering to preserve the subgrade integrity, and to provide a working platform. Utility structures such as catchbasins, manholes and utility chambers 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.

It will be necessary to lower the ground water level at least 1.2 m below the excavation base prior to and during the subsurface construction, and therefore positive dewatering in the form of either staged well points or eductor wells will be required. Consideration should be given to conducting trial excavations (test pits) to assess the stability of the excavation and ground water influx once the design details of the development are finalized (including the invert elevations of the underground utilities). This information would help finalize the requirements for ground water control and dewatering.

The design of a dewatering system will depend on various site specific parameters including soil permeability, subsurface stratigraphy, height of lift, size of the work area and depth of the ground water table. A typical dewatering system includes pumping from sumps located at the base of excavation or well points. Pumping from the sumps may be effective for shallow excavations, up to about 1.5 m below the ground water level. A well point system may be required for excavations carried below this depth. Well points are small-diameter (about 50 mm) tubes with slots near the bottom that are inserted into the ground from which water is drawn by a vacuum generated by a dewatering pump. Wellpoints are typically installed at close centers in a line along or around the edge of an excavation. As a vacuum is limited to 0 bar, the height to which water can be drawn is limited to about 6 meters (in practice). Wellpoints can be installed in stages, with the first reducing the water level by up to five meters, and a second stage, installed at a lower level, lowering it further.

The eductor system is generally used in areas where the soils have a low permeability. It is especially well suited for deep excavations with stratified soils. The eductors are installed at relatively close spacing similar to the array in well point systems, but require only a single stage to effect draw downs of up to 30 feet.

It is recommended to consult a professional dewatering contractor 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.

7.0 LIMITATIONS AND USE OF REPORT

7.1 Procedures

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained from this investigation, as well as factual data reported by other engineering consultants and made available to Terraprobe. Terraprobe takes no responsibility for the quality or accuracy of data reported by other engineering consultants.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. A comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe 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.

7.2 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 Terraprobe as well as prior investigations made by Terraprobe and other consultants, 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. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

7.3 Use of Report

This report is prepared for the express use of R.V. Anderson Associates Ltd., Halton Region, and their retained design consultants. It is not for use by others. This report is copyright of Terraprobe Inc., and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe. R.V. Anderson Associates Ltd., Halton Region, and their retained design consultants are authorized users.

It is recognized that municipal/regional governing bodies, in their capacity as the planning and building authority under Provincial statues, will make use of and rely upon this report, cognizant of the limitations thereof, both as are expressed and implied.

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.

Terraprobe Inc.

LICENSED PROFESSIONAL ENGINEER
M. DIEZ DE AUX
100159147
20/8/2013
PROVINCE OF ONTARIO

Michael Diez de Aux, M.A.Sc., P.Eng.
Geotechnical Engineer

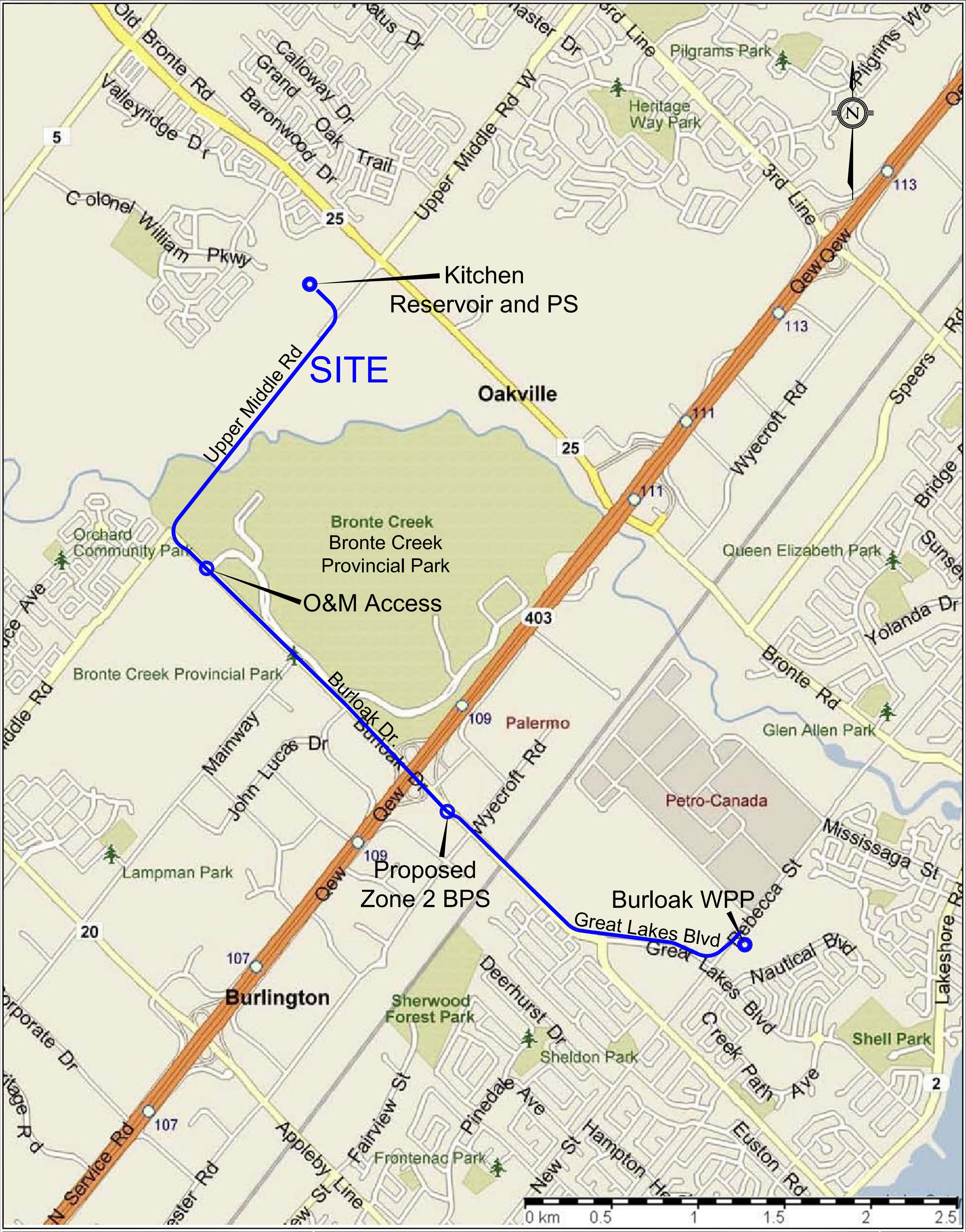
for Jason Crowder, Ph.D., P.Eng.
Associate

LICENSED PROFESSIONAL ENGINEER
T. G. ORPWOOD
20/8/2013
PROVINCE OF ONTARIO

Tim Orpwood, M.A.Sc., P.Geo., P.Eng.
Principal

FIGURES

TERRAPROBE INC.



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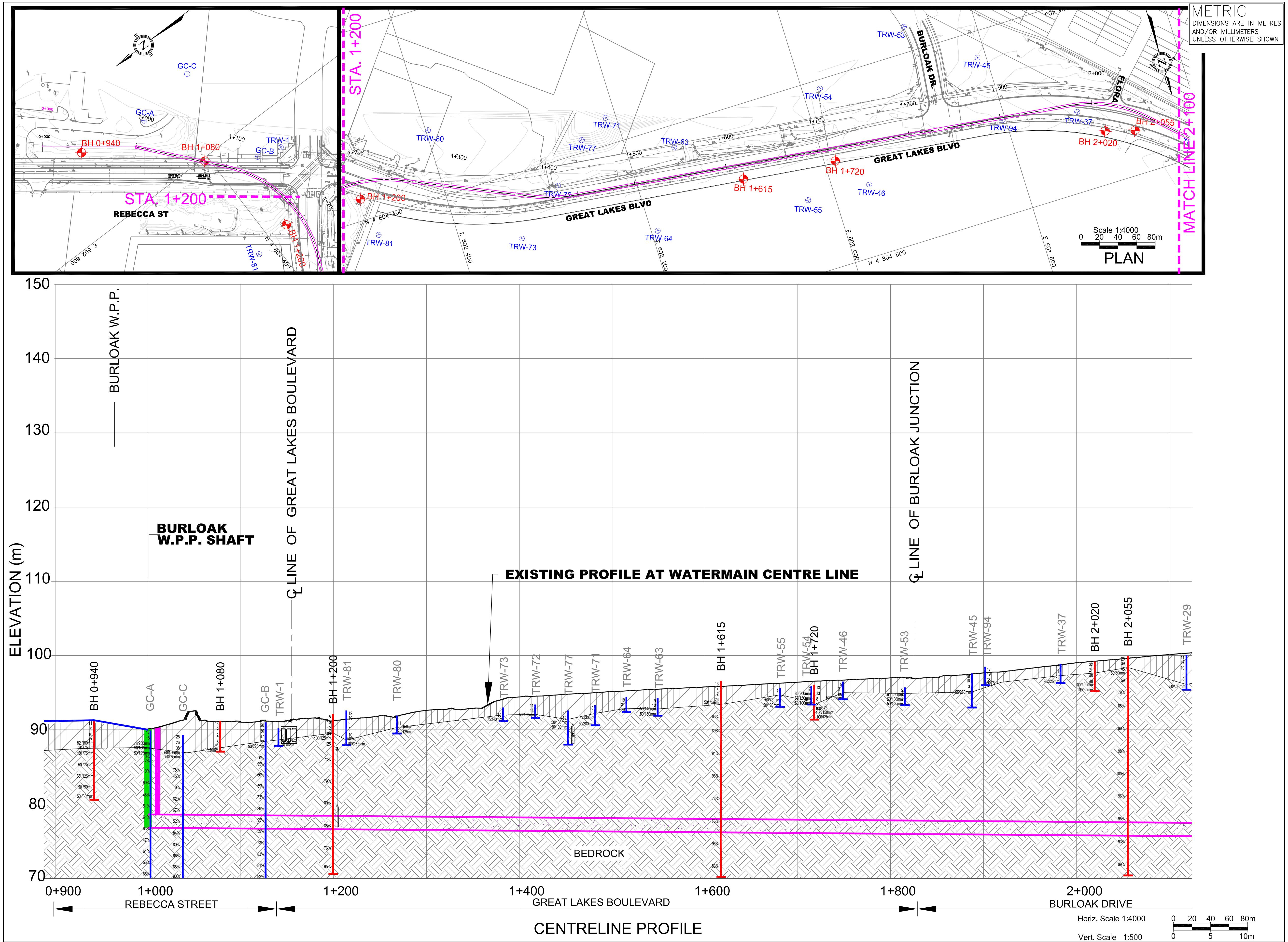
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:	FIGURE :
SITE LOCATION PLAN	1
File No. 11-12-2073	

\\PDC\Server\1-Project Files\11-Geo\2012\11-12-2001 - 2099\11-12-2073 Halton Zone 1 Watermain\A. Dwgs, Logs\AutoCAD\11-12-2073 BH Location Plan Final (R3).dwg, Kamal



Halton Zone 1 Watermain
Burlington–Oakville, Ontario
PROJECT No. 11–12–2073

BOREHOLE LOCATIONS AND PROFILE

Halton REGION — The Regional Municipality of Halton

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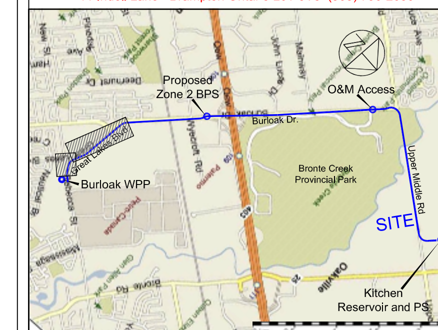


KEY PLAN

LEGEND

- Terraprobe Borehole 2012
- Previous Terraprobe Borehole
- Borehole By Others
- 'N' — Blows/0.3m (Std Pen Test, 475 J/blow)
- WL in Piezometer
- Piezometer
- 90% — Rock Quality Designation
- Overburden Soils
- Bedrock

Terraprobe Borehole 2012

BH No.	ELEV.	COORDINATES	
		NORTHING	EASTING
BH 0+940	91.3	4 804 511.2	602 676.6
BH 1+080	91.2	4 804 400.8	602 598.2
BH 1+200	92.1	4 804 377.3	602 487.5
BH 1+615	96.6	4 804 486.6	602 088.5
BH 1+720	96.1	4 804 496.6	601 988.5
BH 2+020	99.2	4 804 543.5	601 699.6
BH 2+055	99.9	4 804 553.0	601 670.0

NOTE

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

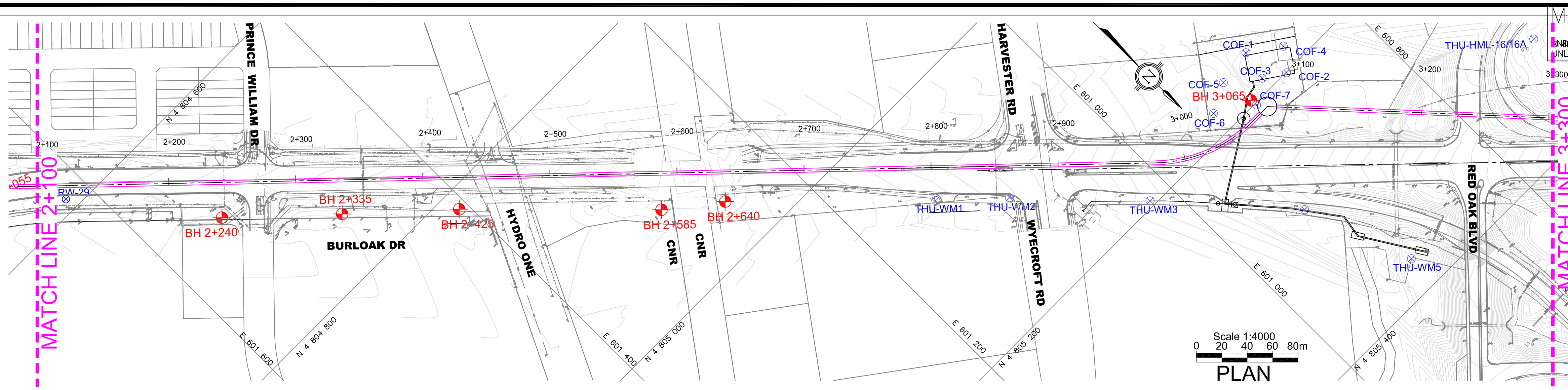
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

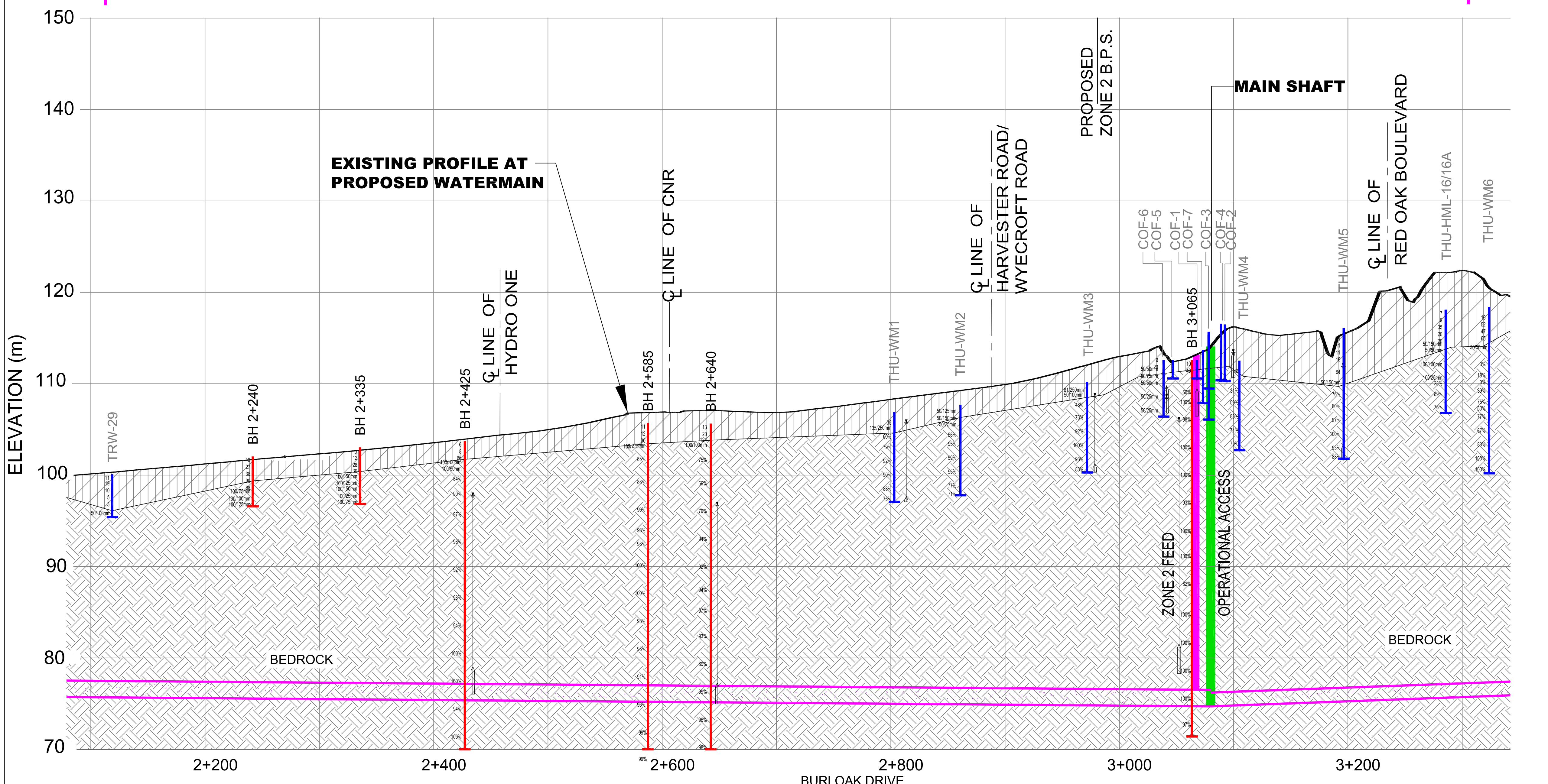
DATE:

FIGURE No.: 2A

\\PDC\Server\1-Project Files\11-Geo\2012\11-12-2001 - 2099\11-12-2073 Halton Zone 1 Watermain\A. Dwgs, Logs\AutoCAD\11-12-2073 BH Location Plan Final (R4).dwg, Kamal



METRIC
DIMENSIONS ARE IN METRES UNLESS OTHERWISE SHOWN



CENTRELINE PROFILE

Horiz. Scale 1:4000 — 0 20 40 60 80m

Vert. Scale 1:500 — 0 5 10m

Halton Zone 1 Watermain
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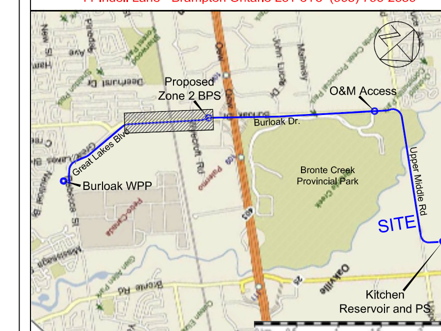

KEY PLAN

LEGEND

	Terraprobe Borehole 2012
	Previous Terraprobe Borehole
	Borehole By Others
'N'	Blows/0.3m (Std Pen Test, 475 J/blow)
	WL in Piezometer
	Piezometer
90%	Rock Quality Designation
	Overburden Soils
	Bedrock

Terraprobe Borehole 2012

BH No.	ELEV.	COORDINATES	
		NORTHING	EASTING
BH 2+240	102.0	4 804 687.7	601 543.1
BH 2+335	103.0	4 804 751.5	601 474.3
BH 2+425	103.7	4 804 814.7	601 407.8
BH 2+585	105.7	4 804 927.7	601 294.2
BH 2+640	105.6	4 804 961.1	601 250.1
BH 3+065	112.6	4 805 189.7	600 915.5

NOTE

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REVISIONS	DATE	BY	DESCRIPTION

DATE:

FIGURE No.: 2B

METRIC
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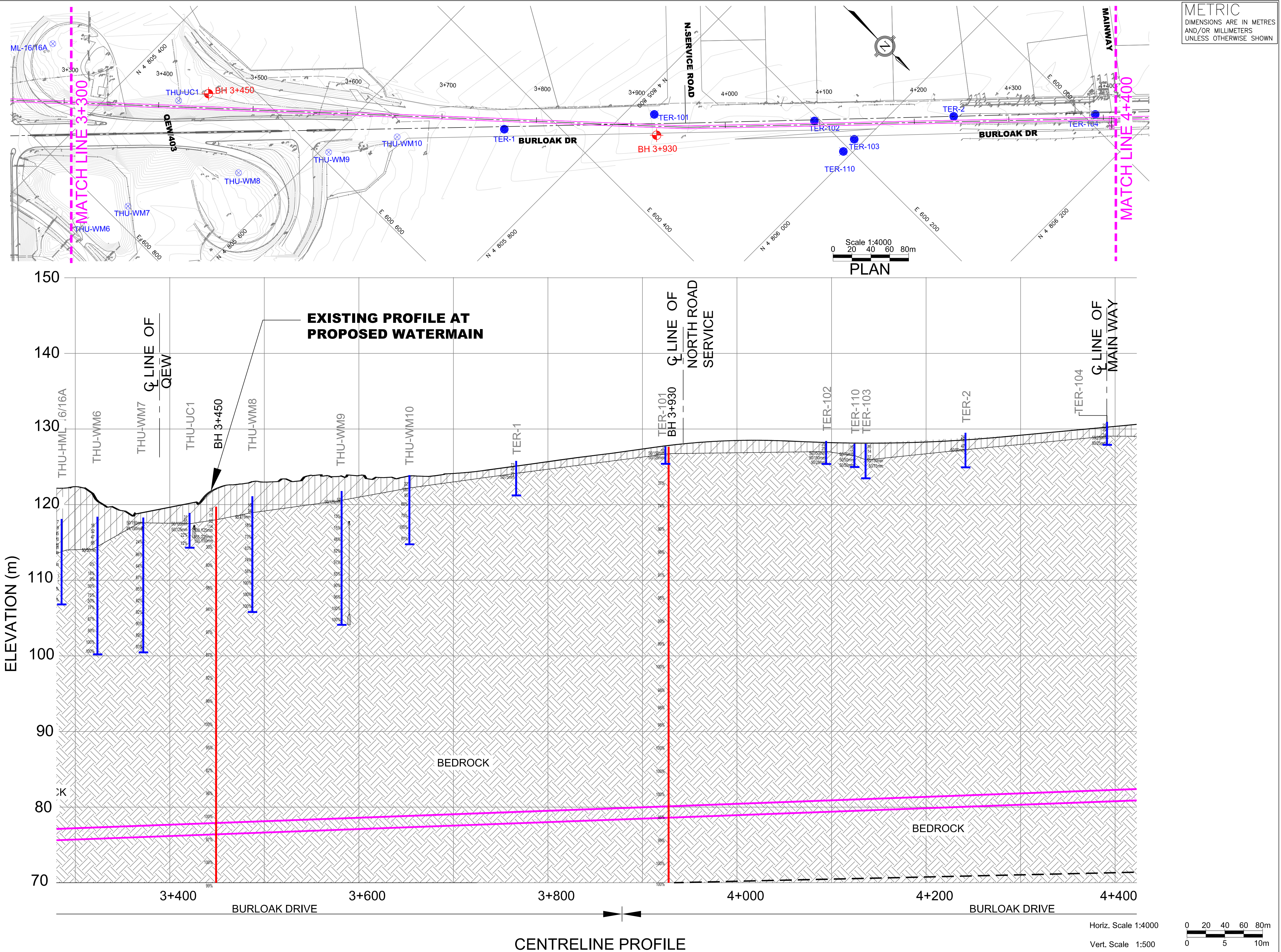

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PROJECT No. 11–12–2073

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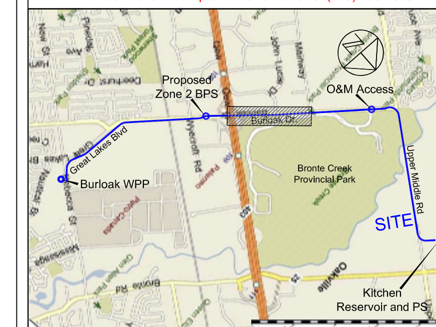

KEY PLAN

LEGEND

	Terraprobe Borehole 2012
	Previous Terraprobe Borehole
	Borehole By Others
'N'	Blows/0.3m (Std Pen Test, 475 J/blow)
	WL in Piezometer
	Piezometer
90%	Rock Quality Designation
	Overburden Soils
	Bedrock

Terraprobe Borehole 2012

BH No.	ELEV.	COORDINATES NORTHING	COORDINATES EASTING
BH 3+450	119.7	4 805 474.8	600 642.8
BH 3+930	127.6	4 805 842.0	600 335.0

NOTE

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

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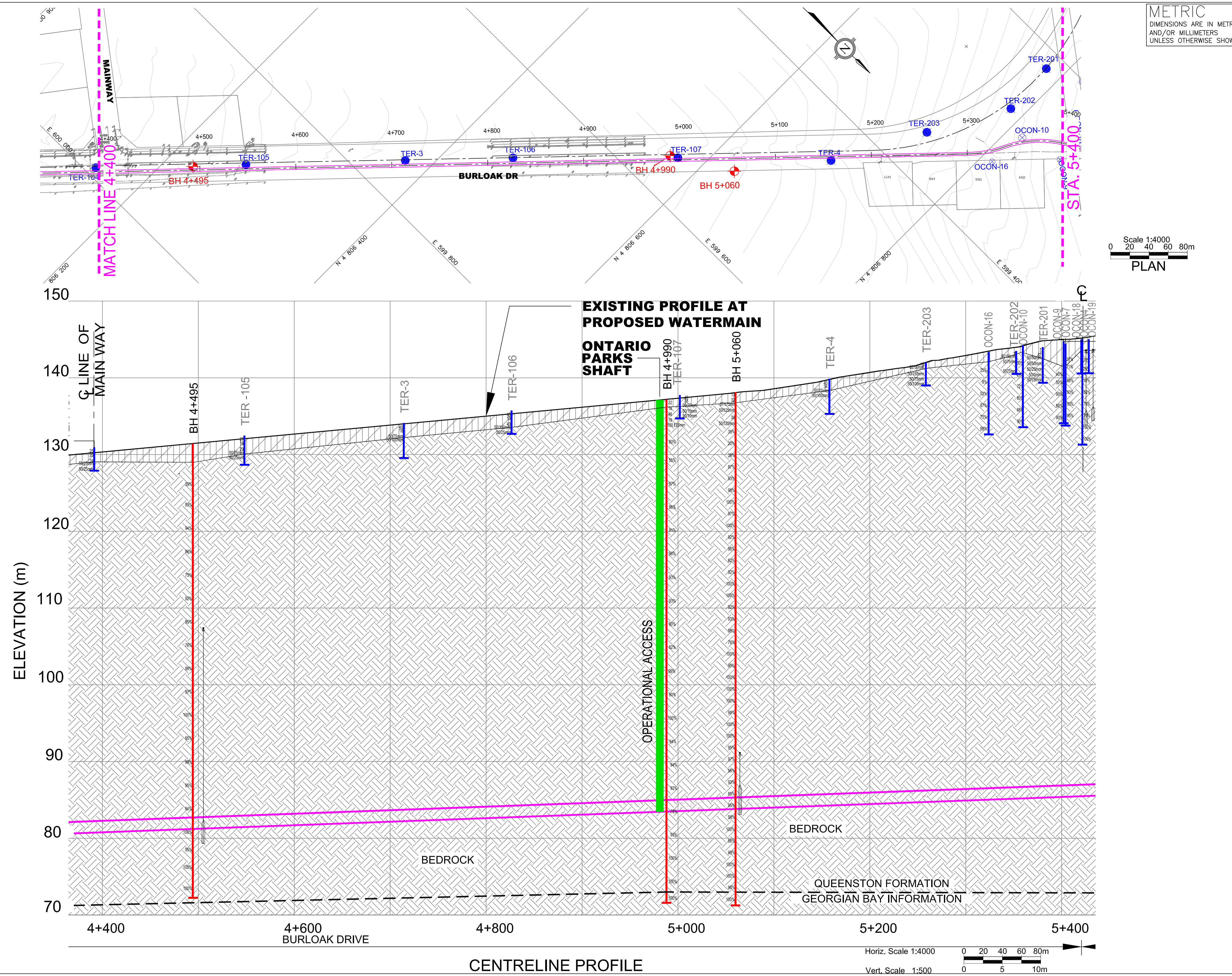
REVISIONS	DATE	BY	DESCRIPTION

DATE:

FIGURE No.: 2C

\\PDC\Server\1-Project Files\11-Geo\2012\11-12-2001 - 2099\11-12-2073 Halton Zone 1 Watermain\A. Dwgs, Logs\AutoCAD\11-12-2073 BH Location Plan Final (R4).dwg, Kamal



METRIC
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Halton Zone 1 Watermain
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PROJECT No. 11–12–2073

BOREHOLE LOCATIONS AND PROFILE

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KEY PLAN

LEGEND

- Terraprobe Borehole 2012
- Previous Terraprobe Borehole
- Borehole By Others
- 'N' Blows/0.3m (Std Pen Test, 475 J/blow)
- WL in Piezometer
- Piezometer
- 90% Rock Quality Designation
- Overburden Soils
- Bedrock

Terraprobe Borehole 2012

BH No.	ELEV.	COORDINATES NORTHING	COORDINATES EASTING
BH 4+495	131.4	4 806 229.8	599 920.9
BH 4+990	137.2	4 806 571.2	599 564.2
BH 5+060	138.1	4 806 627.9	599 523.3

NOTE

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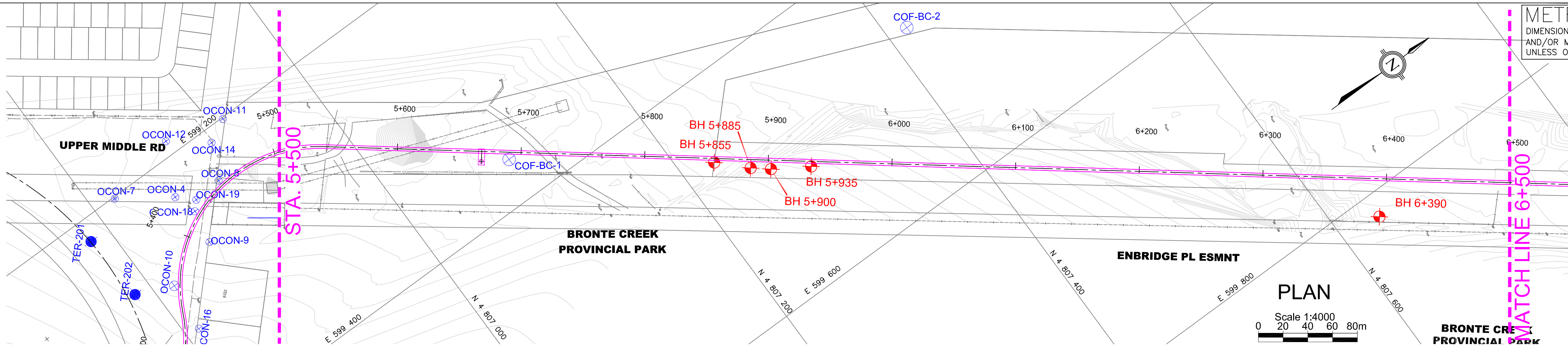
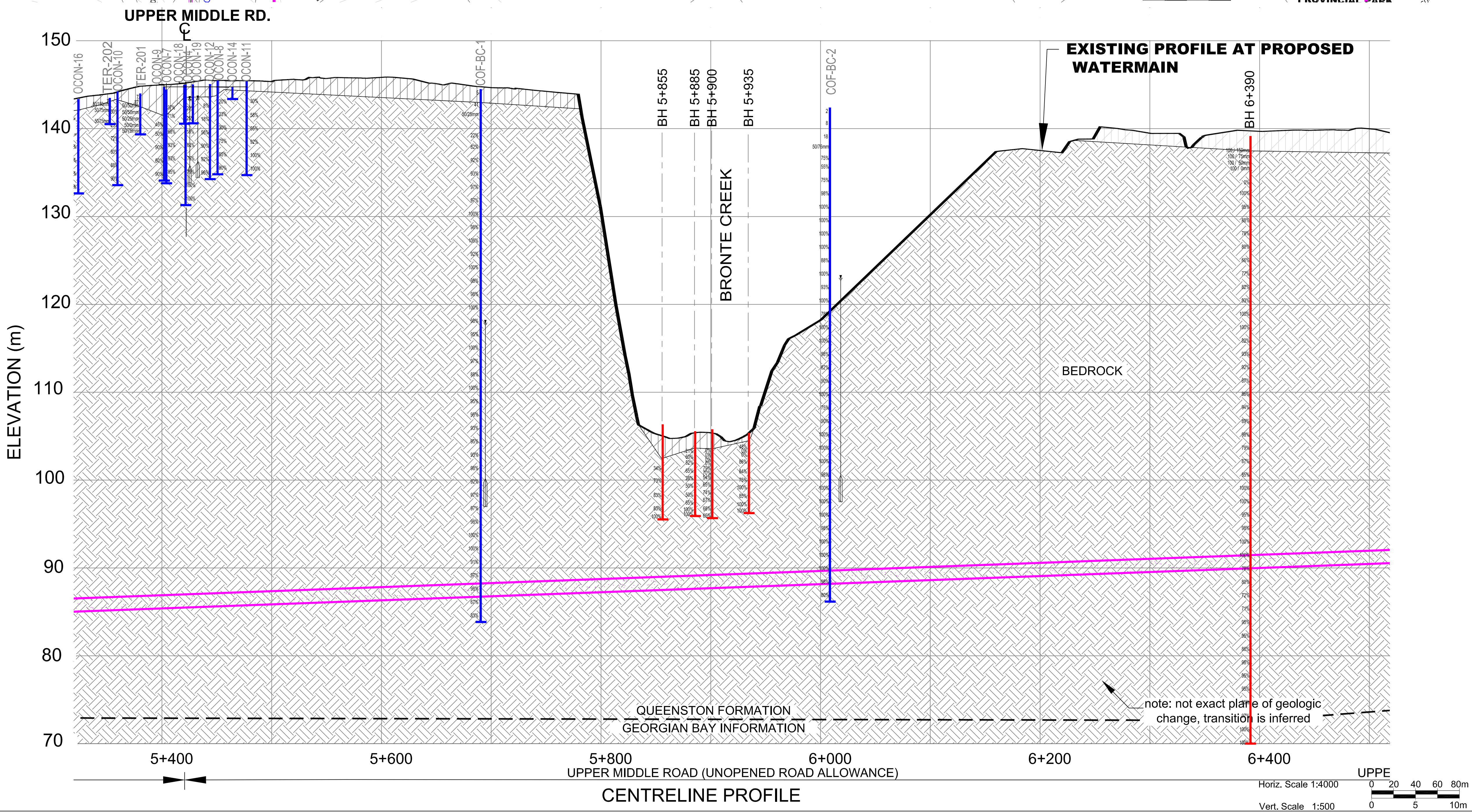
REVISIONS DATE	BY	DESCRIPTION

DATE:

FIGURE No.: 2D

\\PDC\Server\1-Project Files\11-Geo\2012\11-12-2001 - 2099\11-12-2073 Halton Zone 1 Watermain\A. Dwgs, Logs\AutoCAD\11-12-2073 BH Location Plan Final (R4).dwg, Kamal





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PROJECT No. 11–12–2073

BOREHOLE LOCATIONS AND PROFILE

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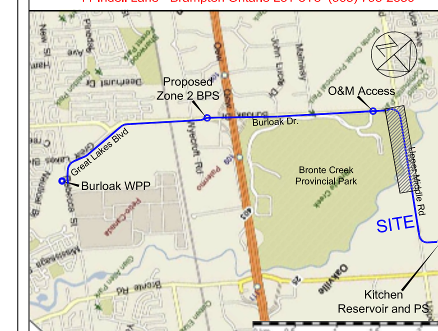

KEY PLAN

LEGEND

- Terraprobe Borehole 2012
- Previous Terraprobe Borehole
- Borehole By Others
- 'N' — Blows/0.3m (Std Pen Test, 475 J/blow)
- WL in Piezometer
- Piezometer
- 90% — Rock Quality Designation
- Overburden Soils
- Bedrock

Terraprobe Borehole 2012

BH No.	ELEV.	COORDINATES	
		NORTHING	EASTING
BH 5+855	106.3	4 807 226.7	599 465.4
BH 5+885	105.6	4 807 247.8	599 486.0
BH 5+900	105.8	4 807 259.8	599 496.0
BH 5+935	105.4	4 807 288.0	599 514.4
BH 6+390	139.2	4 807 621.5	599 829.4

NOTE

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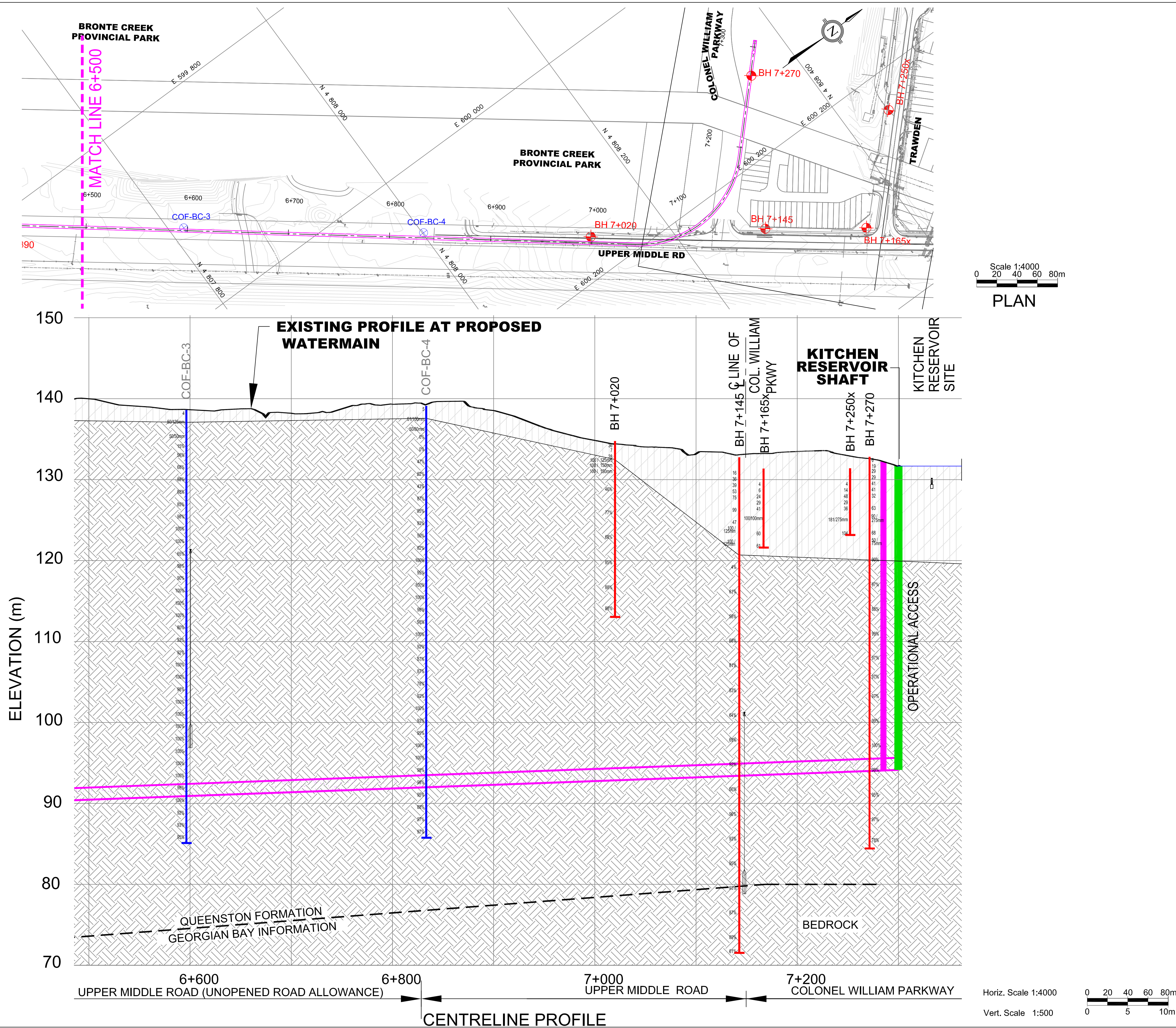
REVISIONS

DATE	BY	DESCRIPTION

DATE:

FIGURE No.: 2E



METRIC
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PROJECT No. 11–12–2073

BOREHOLE LOCATIONS AND PROFILE

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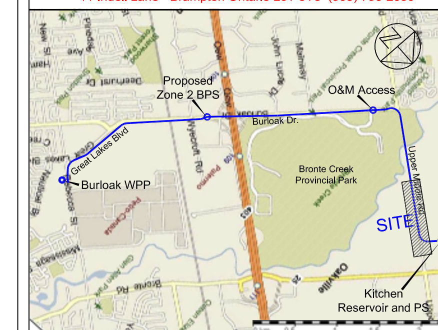

KEY PLAN

LEGEND

Terraprobe Borehole 2012
Previous Terraprobe Borehole
Borehole By Others
'N' Blows/0.3m (Std Pen Test, 475 J/blow)
WL in Piezometer
Piezometer
90% Rock Quality Designation
Overburden Soils
Bedrock

Terraprobe Borehole 2012

BH No.	ELEV.	COORDINATES	
		NORTHING	EASTING
BH 7+020	134.8	4 808 148.9	600 174.9
BH 7+145	132.8	4 808 273.6	600 256.9
BH 7+165x	132.8	4 808 354.2	600 311.8
BH 7+250x	131.4	4 808 443.0	600 240.6
BH 7+270	132.8	4 808 354.0	600 128.0

NOTE

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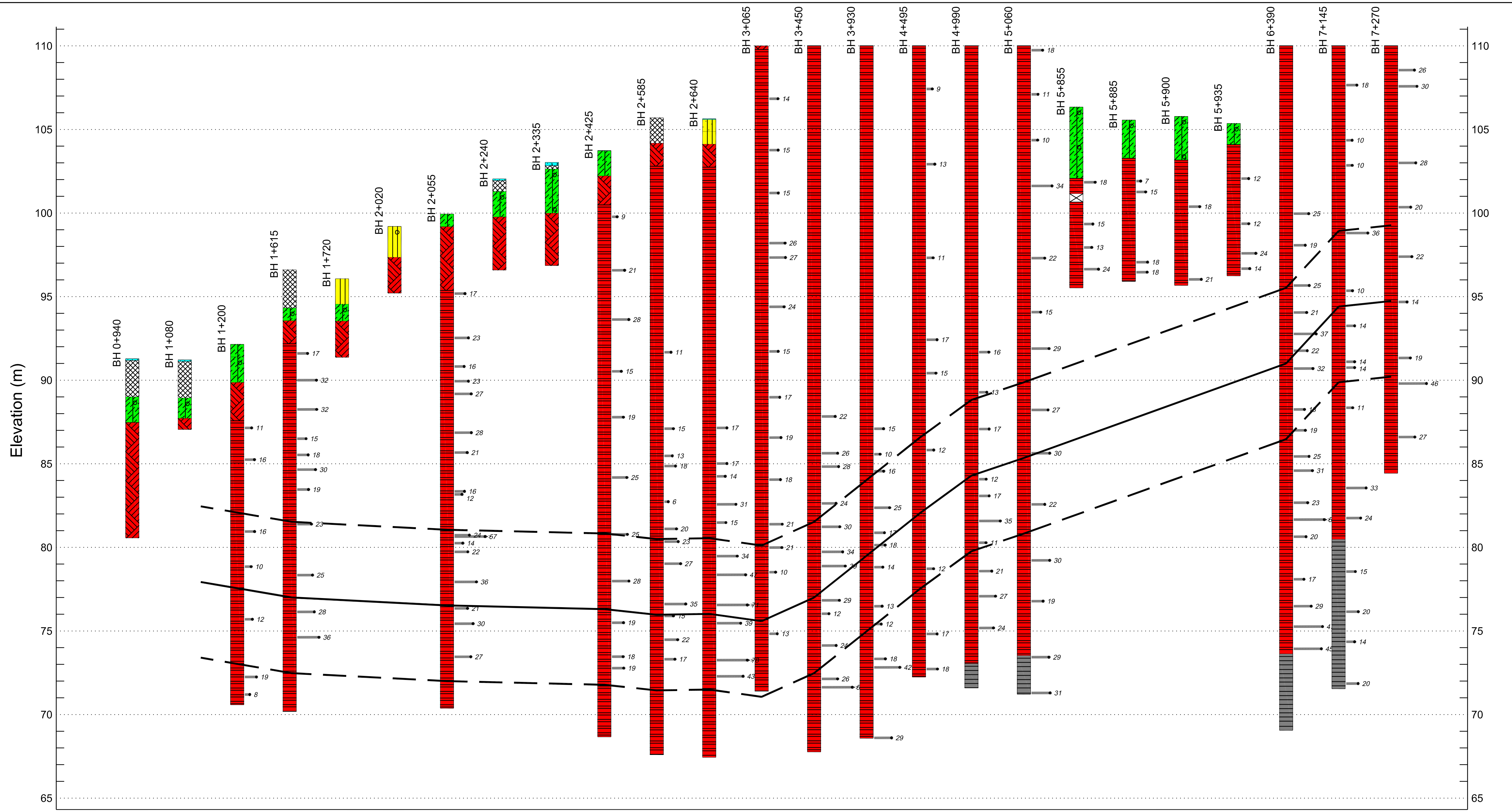
REVISIONS

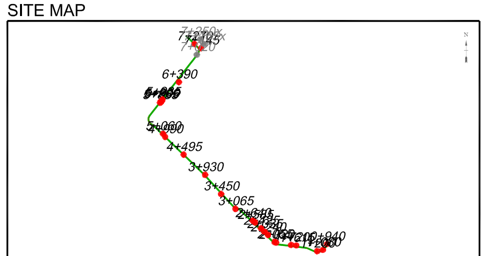
DATE	BY	DESCRIPTION

DATE:

FIGURE No.: 2F



SITE MAP



LITHOLOGY GRAPHIC LEGEND

- CORE LOSS
- Fill
- Silty Till
- Bedrock
- Queenston Formation
- Sandy Silt
- Clayey Silt
- Georgian Bay Formation
- Topsoil
- Clayey Silt Till
- Silt
- Approximate tunnel springline location
- ± 4.5m zone around springline

Note: Boreholes are evenly spaced. Horizontal distance is not to scale.
For interpretive purposes only.

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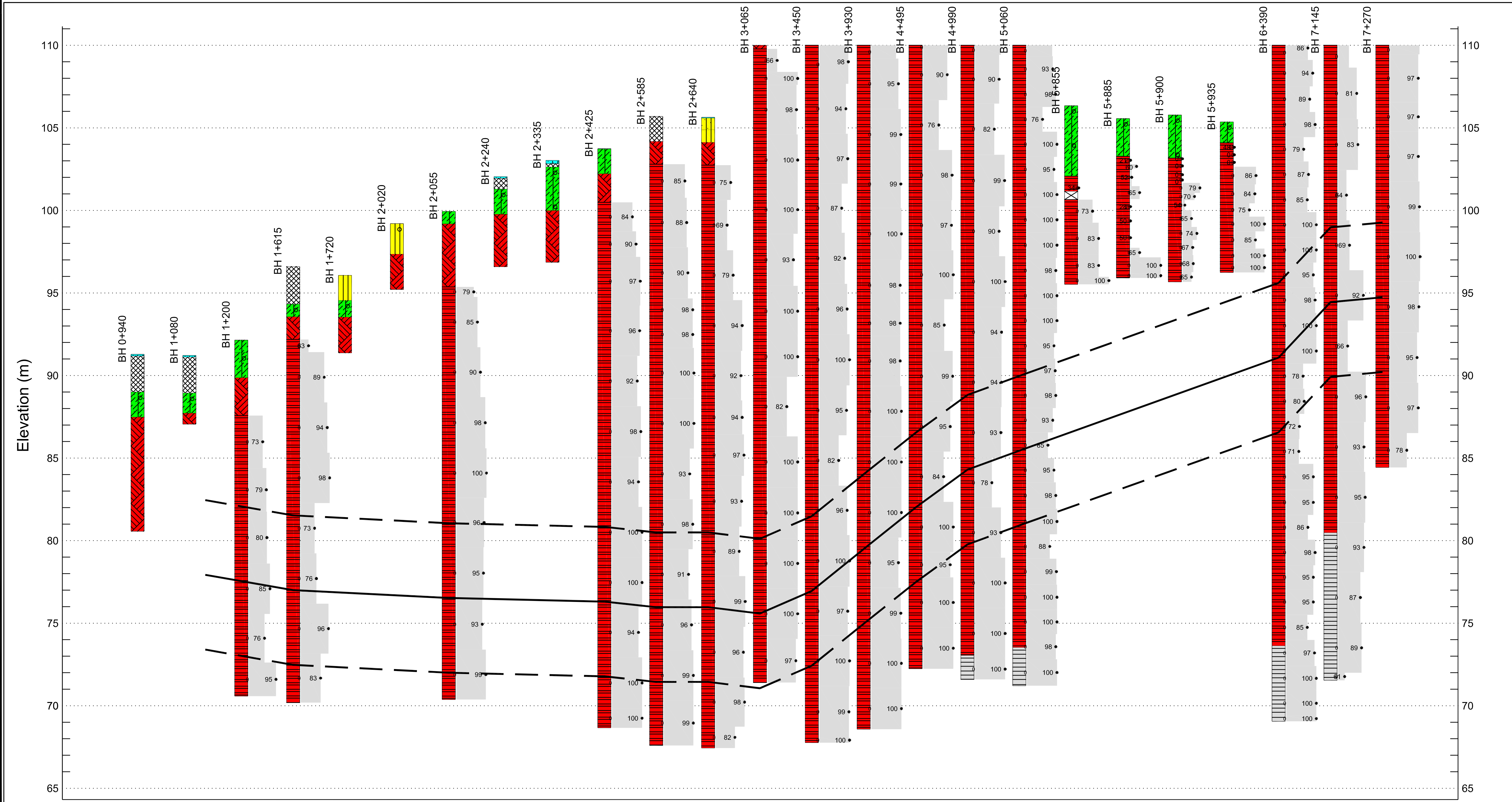
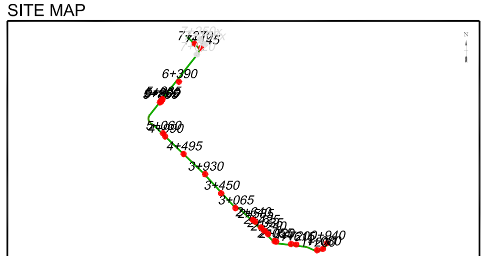
Title:	SUBSURFACE PROFILE UCS VS. ELEVATION
File No.	11-12-2073
FIGURE :	3



Note: Boreholes are evenly spaced. Horizontal distance is not to scale.
For interpretive purposes only.

SITE MAP



LITHOLOGY GRAPHIC LEGEND

- CORE LOSS
- Fill
- Silty Till
- Bedrock
- Queenston Formation
- Sandy Silt
- Clayey Silt
- Georgian Bay Formation
- Topsoil
- Clayey Silt Till
- Silt
- Approximate tunnel springline location
- ± 4.5m zone around springline

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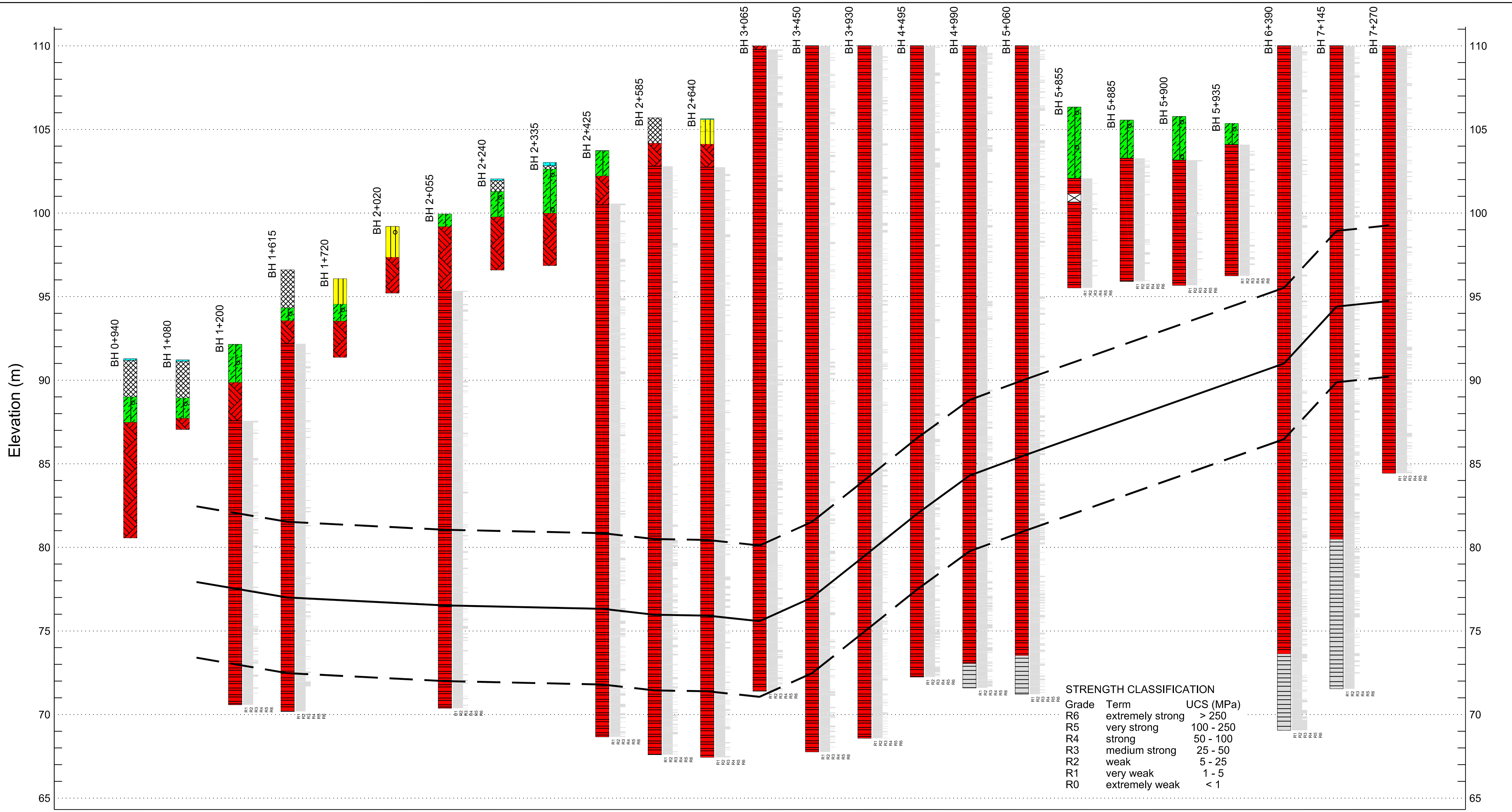
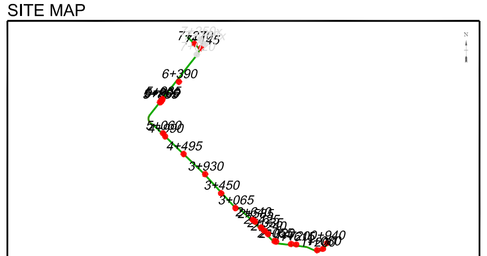
Title:	FIGURE :
SUBSURFACE PROFILE RQD VS. ELEVATION	4
File No. 11-12-2073	



SITE MAP



LITHOLOGY GRAPHIC LEGEND

- CORE LOSS
- Bedrock
- Clayey Silt
- Clayey Silt Till
- Fill
- Queenston Formation
- Georgian Bay Formation
- Silt
- Silty Till
- Sandy Silt
- Topsoil
- Approximate tunnel springline location
- ± 4.5m zone around springline

Note: Boreholes are evenly spaced. Horizontal distance is not to scale.
For interpretive purposes only.

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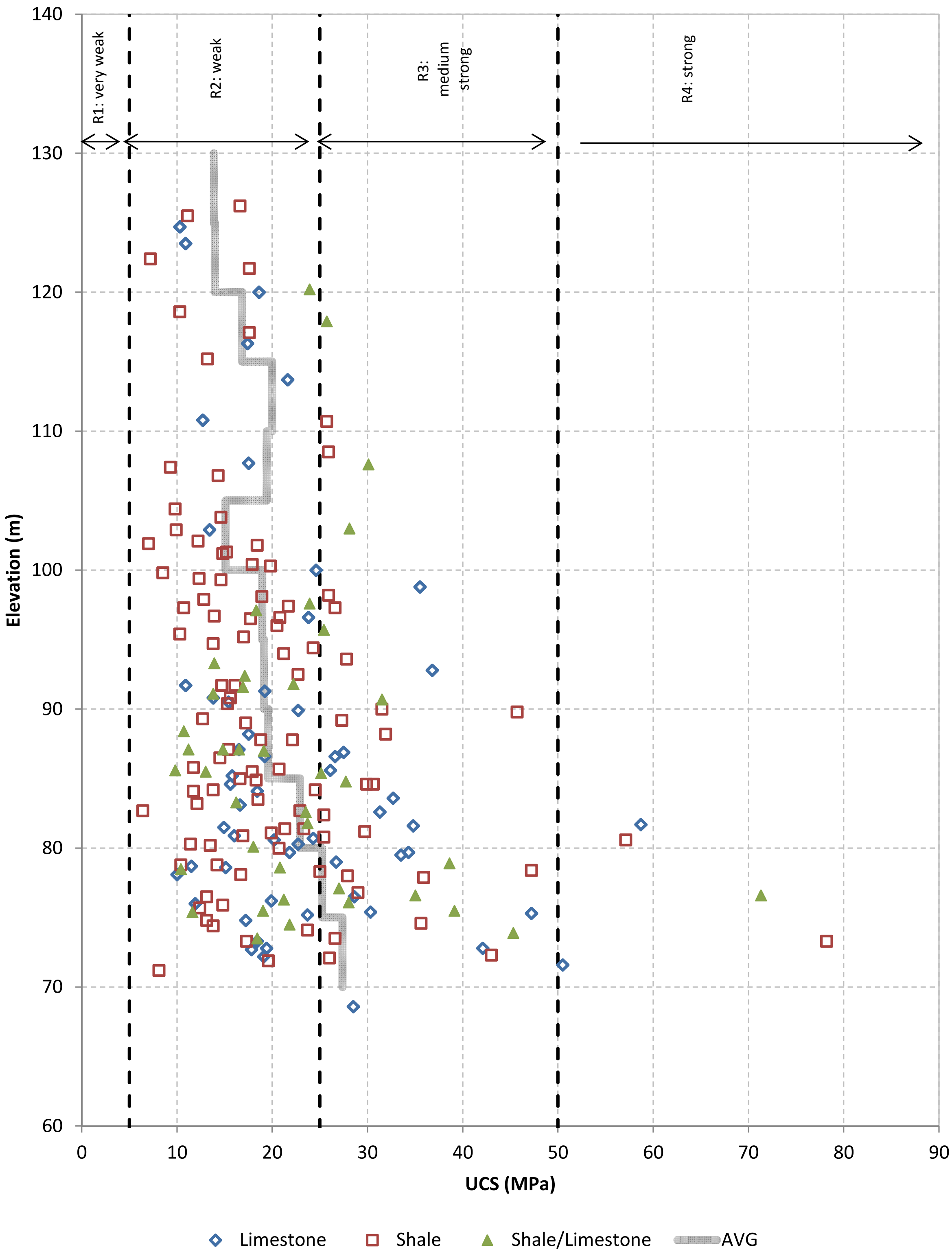
Title:	SUBSURFACE PROFILE HARD LAYERS VS. ELEVATION	FIGURE : 5
File No.	11-12-2073	

UCS vs. ELEVATION	Figure 6



Project No. : 11-12-2073

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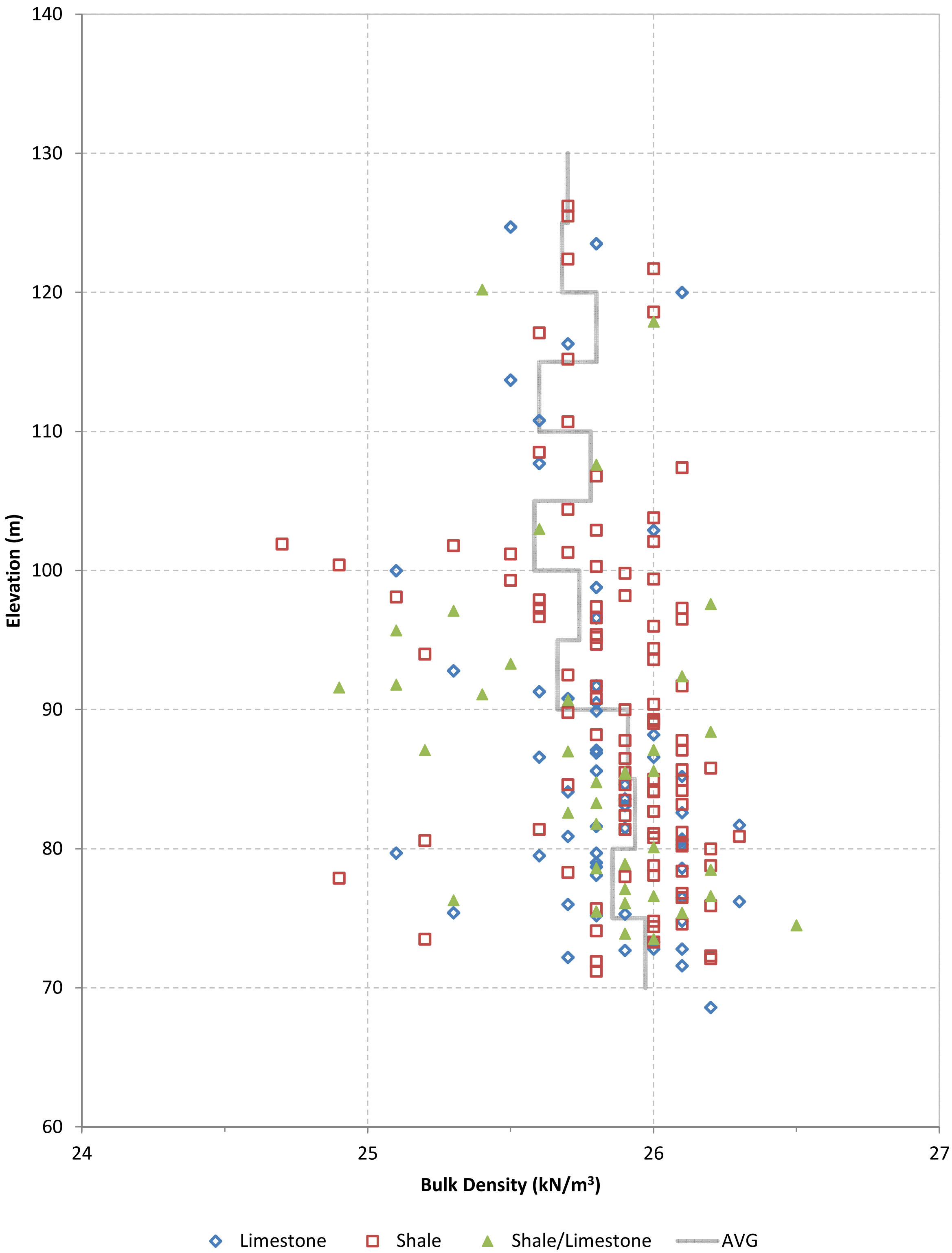
BULK DENSITY vs. ELEVATION	Figure 7



Project No. : 11-12-2073

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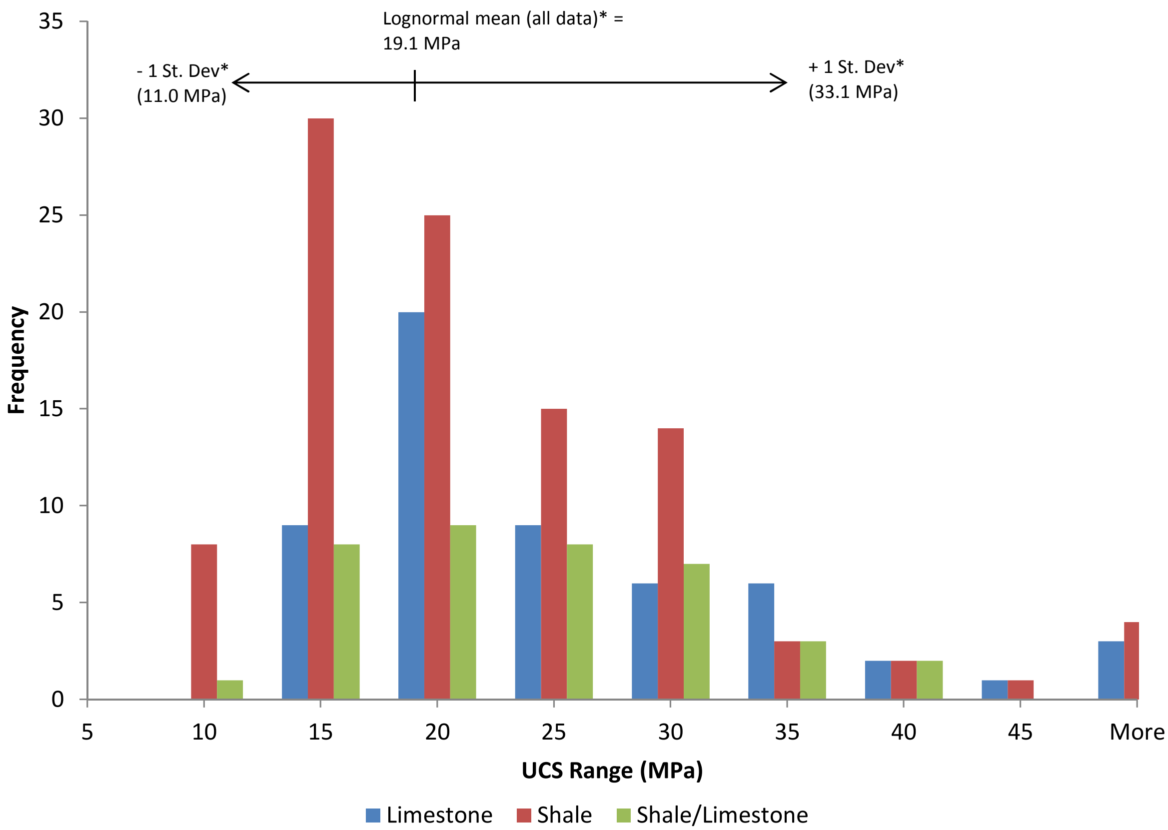
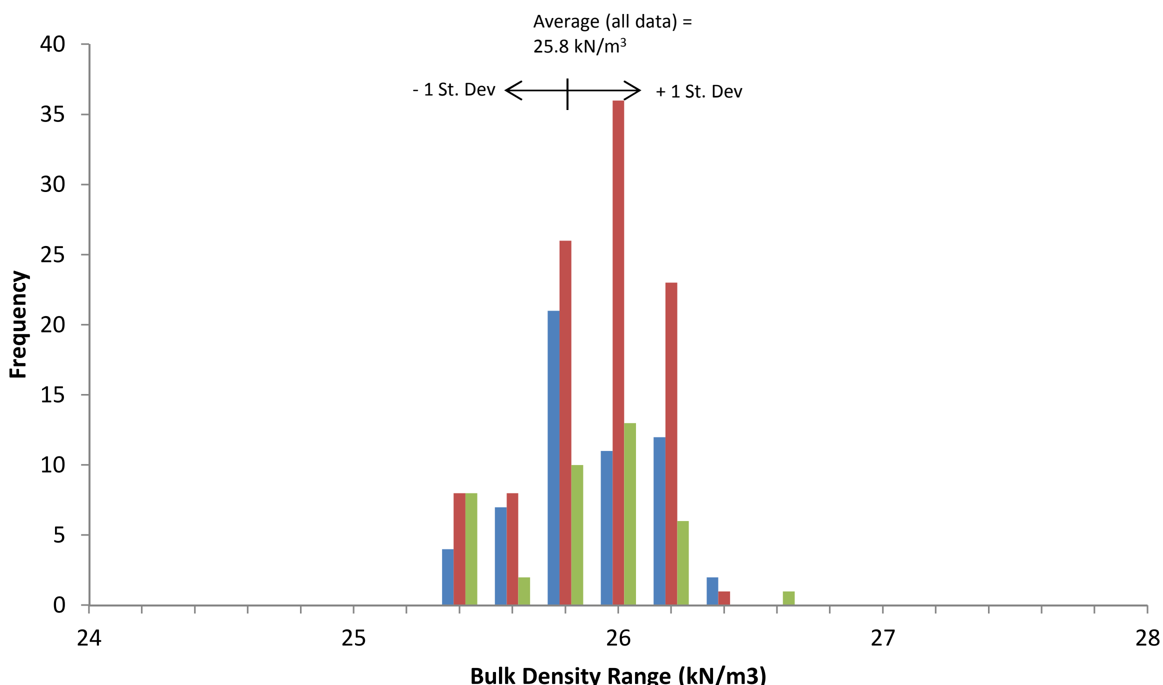
UCS AND BULK DENSITY - STATISTICS | **Figure 8**





** UCS is lognormally distributed*

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PROJECT No. 11–12–2073

SETTLEMENT MONITORING PLAN
CNR TRACKS

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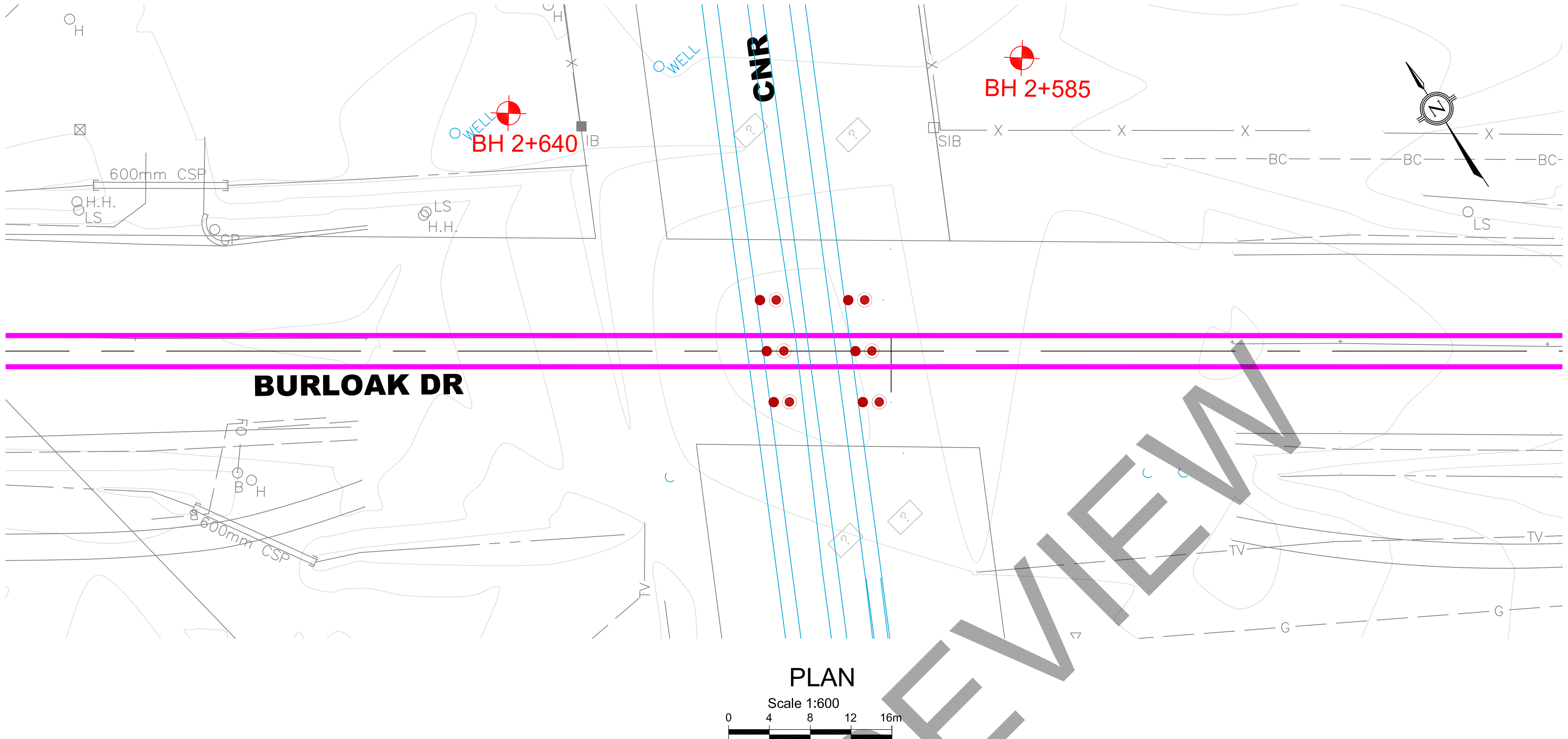
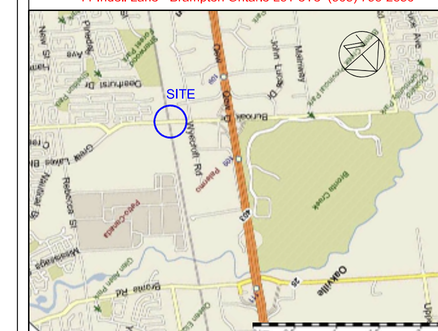
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN



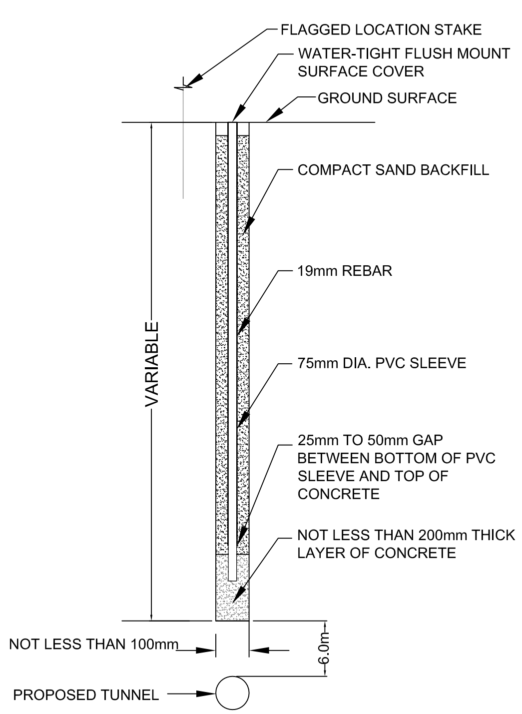
PLAN



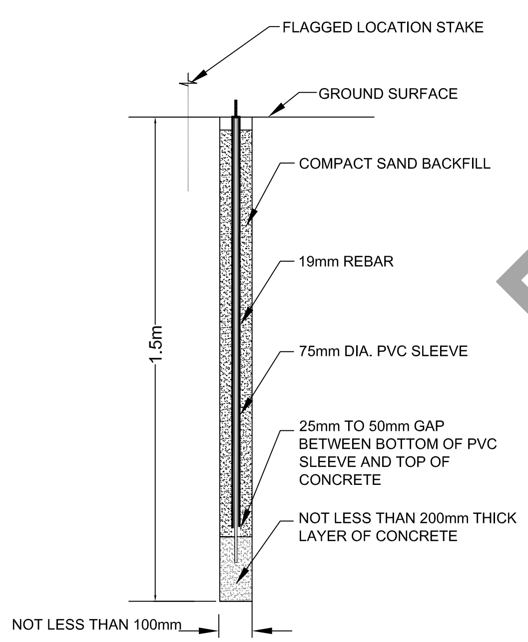
KEY PLAN

LEGEND

- BOREHOLE
- SURFACE MONITORING POINT (Array of point installed at 5m± intervals)
- IN–GROUND MONITORING POINT (Array of point installed at 5m± intervals)



IN-GROUND MONITORING POINT INSTALLATION DETAIL
N.T.S



SURFACE MONITORING POINT INSTALLATION DETAIL
N.T.S

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH TERRAPROBE INC. REPORT VOL 2: DESIGN REPORT. FILE No. 11–12–2073, DATED AUG. 2013.
2. ALL MONITORING LOCATIONS SHOULD BE CONSIDERED APPROXIMATE AND MUST BE CONFIRMED BY THE CONTRACTOR IN CONSULTATION WITH THE CONTRACT ADMINSTRATOR, GEOTECHNICAL ENGINEER, R.V.ANDERSON AND CNR PRIOR TO INSTALLATION/CONSTRUCTION AND MAY HAVE TO BE ADJUSTED IN THE FIELD TO SUIT LOCAL CONDITIONS/CONSTRAINTS.
3. THE CONTRACTOR SHALL RETAIN A SURVEYOR REGISTERED IN ONTARIO FOR ESTABLISHING AND SURVEYING THE MONITORING POINTS FOR THE DURATION OF CONSTRUCTION.
4. ALL MONITORING INSTRUMENTS SHALL BE INSTALLED AT LEAST 14 DAYS PRIOR TO ANY EXCAVATION OR TUNNELLING TAKING PLACE.
5. IN–GROUND MONITORING POINTS INSTALLED SHALL BE FOUNDED BELOW FROST PENETRATION DEPTH (1.2m).
6. THE CONTRACTOR SHALL ESTABLISH A TEMPORARY BENCHMARK OUTSIDE THE AREA OF CONSTRUCTION. THE CONTRACTOR SHALL SUBMIT THE PROPOSED BENCHMARK LOCATION TO THE CONTRACT ADMINISTRATOR FOR APPROVAL. PRIOR TO CONSTRUCTION ALL MONITORING POINTS SHALL BE SURVEYED FOR ELEVATION AND LOCATION TO A TOLERANCE OF NOT MORE THAN 1mm IN THE VERTICAL AND HORIZONTAL DIRECTION.
7. DURING TUNNELLING, ALL POINTS SHALL BE SURVEYED ONCE PER SHIFT.
8. THE SPECIFIED SETTLEMENT REVIEW LEVEL IS 8mm AND THE SPECIFIED SETTLEMENT ALERT LEVEL IS 12mm RELATIVE TO THE BASELINE READING. THE REQUIRED ACTIONS TO BE TAKEN IF THE REVIEW AND ALERT LEVELS ARE REACHED OR EXCEEDED ARE SPECIFIED IN THE NON–STANDARD SPECIAL PROVISION, SETTLEMENT INSTRUMENTATION AND MONITORING.
9. THE FREQUENCY AT WHICH THE CONTRACTOR IS REQUIRED TO SURVEY THE MONITORING POINTS ARE SPECIFIED IN THE NON–STANDARD SPECIAL PROVISION, SETTLEMENT INSTRUMENTATION AND MONITORING.
10. WITHIN 24 HOURS OF COMPLETION OF ANY MEASUREMENT A COPY OF THE RESULTS SHALL BE MADE AVAILABLE TO THE CONTRACT ADMINISTRATOR, GEOTECHNICAL ENGINEER, R.V.ANDERSON AND CNR.
11. THE CONTRACTOR SHALL MAKE ALL ARRANGEMENTS FOR TRAFFIC CONTROL IN ACCORDANCE WITH ONTARIO TRAFFIC MANUAL BOOK 7.
12. REMOVE ALL MONITORING POINTS ON COMPLETION OF SURVEY, SUBJECT TO APPROVAL FROM THE CONTRACT ADMINISTRATOR, GEOTECHNICAL ENGINEER, R.V.ANDERSON AND CNR.

Terraprobe Borehole 2012

BH No.	ELEV.	COORDINATES	
		NORTHING	EASTING
BH 2+585	105.7	4 804 928	601 295
BH 2+640	105.6	4 804 959	601 255

NOTE

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

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

DATE:

FIGURE No.: 9

Halton Zone 1 Watermain
Burlington–Oakville, Ontario
PROJECT No. 11–12–2073

SETTLEMENT MONITORING PLAN

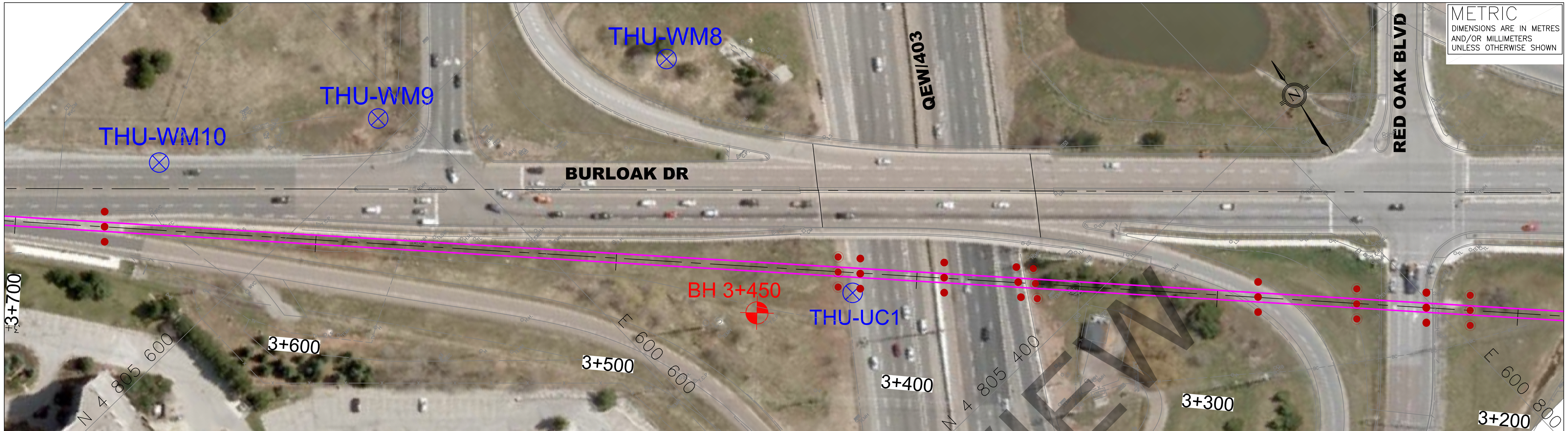
QEW / BURLOAK DR

Halton The Regional Municipality of Halton

R.V. Anderson Associates Limited engineering • environment • infrastructure

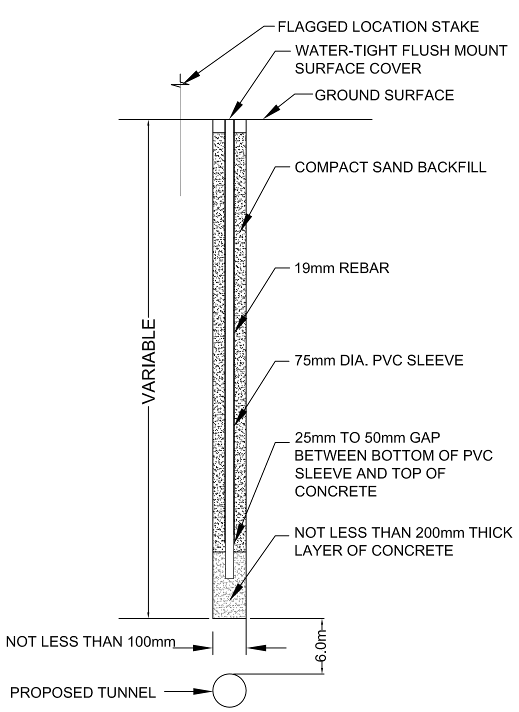
Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials, Inspection & Testing
11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN

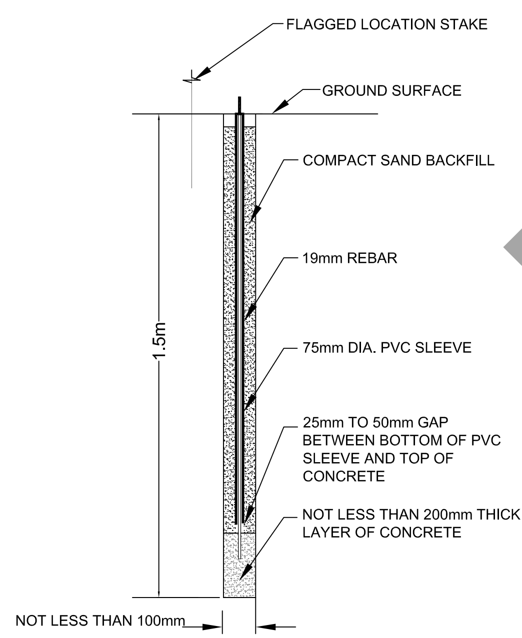


PLAN

Scale 1:150
0 8 16 24 32m



IN-GROUND MONITORING POINT INSTALLATION DETAIL
N.T.S



SURFACE MONITORING POINT INSTALLATION DETAIL
N.T.S

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH TERRAPROBE INC. REPORT VOL 2: DESIGN REPORT. FILE No. 11–12–2073, DATED AUG. 2013.
2. ALL MONITORING LOCATIONS SHOULD BE CONSIDERED APPROXIMATE AND MUST BE CONFIRMED BY THE CONTRACTOR IN CONSULTATION WITH THE CONTRACT ADMINISTRATOR, GEOTECHNICAL ENGINEER, R.V.ANDERSON AND MTO PRIOR TO INSTALLATION/CONSTRUCTION AND MAY HAVE TO BE ADJUSTED IN THE FIELD TO SUIT LOCAL CONDITIONS/CONSTRAINTS.
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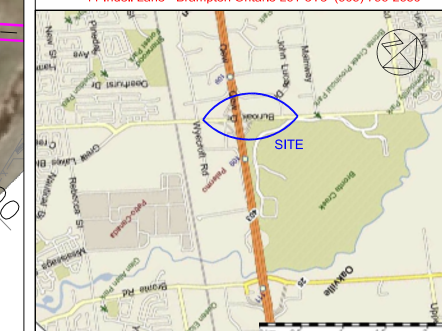
4. ALL MONITORING INSTRUMENTS SHALL BE INSTALLED AT LEAST 14 DAYS PRIOR TO ANY EXCAVATION OR TUNNELLING TAKING PLACE.
5. IN–GROUND MONITORING POINTS INSTALLED SHALL BE FOUNDED BELOW FROST PENETRATION DEPTH (1.2m). SURFACE MONITORING POINTS CONSISTING OF HARDENED STEEL MARKERS TREATED OR COATED TO RESIST CORROSION, WITH AN EXPOSED CONVEX HEAD HAVING A MINIMUM DIAMETER OF 12 MM AND SIMILAR TO A SURVEYOR'S PK NAIL ARE RECOMMENDED FOR USE ON ASPHALT PAVED SURFACES.
6. THE CONTRACTOR SHALL ESTABLISH A TEMPORARY BENCHMARK OUTSIDE THE AREA OF CONSTRUCTION. THE CONTRACTOR SHALL SUBMIT THE PROPOSED BENCHMARK LOCATION TO THE CONTRACT ADMINISTRATOR FOR APPROVAL. PRIOR TO CONSTRUCTION ALL MONITORING POINTS SHALL BE SURVEYED FOR ELEVATION AND LOCATION TO A TOLERANCE OF NOT MORE THAN 1mm IN THE VERTICAL AND HORIZONTAL DIRECTION.
7. DURING TUNNELLING, ALL POINTS SHALL BE SURVEYED ONCE PER SHIFT.
8. THE SPECIFIED SETTLEMENT REVIEW LEVEL IS 10mm AND THE SPECIFIED SETTLEMENT ALERT LEVEL IS 15mm RELATIVE TO THE BASELINE READING. THE REQUIRED ACTIONS TO BE TAKEN IF THE REVIEW AND ALERT LEVELS ARE REACHED OR EXCEEDED ARE SPECIFIED IN THE NON–STANDARD SPECIAL PROVISION, SETTLEMENT INSTRUMENTATION AND MONITORING.
9. THE FREQUENCY AT WHICH THE CONTRACTOR IS REQUIRED TO SURVEY THE MONITORING POINTS ARE SPECIFIED IN THE NON–STANDARD SPECIAL PROVISION, SETTLEMENT INSTRUMENTATION AND MONITORING.
10. WITHIN 24 HOURS OF COMPLETION OF ANY MEASUREMENT A COPY OF THE RESULTS SHALL BE MADE AVAILABLE TO THE CONTRACT ADMINISTRATOR, GEOTECHNICAL ENGINEER, R.V.ANDERSON AND MTO.
11. THE CONTRACTOR SHALL MAKE ALL ARRANGEMENTS FOR TRAFFIC CONTROL IN ACCORDANCE WITH ONTARIO TRAFFIC MANUAL BOOK 7.
12. REMOVE ALL MONITORING POINTS ON COMPLETION OF SURVEY, SUBJECT TO APPROVAL FROM THE CONTRACT ADMINISTRATOR, GEOTECHNICAL ENGINEER, R.V.ANDERSON AND MTO.



KEY PLAN

LEGEND

- TERRAPROBE BOREHOLE
- Borehole By Others
- SURFACE MONITORING POINT (Array of point installed at 5m± intervals)
- IN–GROUND MONITORING POINT (Array of point installed at 5m± intervals)

Terraprobe Borehole 2012

BH No.	ELEV.	COORDINATES	
		NORTHING	EASTING
BH 3+450	105.7	4 805 475	600 642

NOTE

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

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS DATE	BY	DESCRIPTION

DATE:

FIGURE No.: 10

APPENDIX A

TERRAPROBE INC.

1. INSTRUMENTATION MONITORING

The work specified in this section includes furnishing and installing instruments for 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 ± 2 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. Traffic shall be managed by the contractor in accordance with the Ontario Traffic Manual (OTM).

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. The assembly shall be placed in a drill hole and backfilled with uniform sand as shown on the Contract Drawings.

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

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

1.1 CNR Tracks Crossing

The Settlement Monitoring Plan provided in Figure 9 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/constrains.

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);
- Once per shift during tunnelling operations period; and
- Weekly after completion of the work for one month, or until such time at which all parties agree that further movement has stopped.

All readings shall be submitted to the Contract Administrator, Geotechnical Engineer, RV Anderson, and provided to CN Railway 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.

1.2 MTO QEW Crossing

The Settlement Monitoring Plan provided in Figure 10 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/constrains.

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);
- Once per shift during tunnelling operations period; and
- Weekly after completion of the work for one month, or until such time at which all parties agree that further movement has stopped.

All readings shall be submitted to the Contract Administrator, Geotechnical Engineer, RV Anderson, 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.

2. CRITERIA FOR ASSESSMENT OF SUBSIDENCE/HEAVE

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.

2.1 CNR Tracks Crossing

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

- Review Level: If a maximum value of 8 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, CNR, Geotechnical Engineer, and RV Anderson, 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 12 mm relative to the baseline readings is reached, the Contractor shall cease construction operations, inform the Contract Administrator, CNR, Geotechnical Engineer, and RV Anderson and execute pre-planned measures to secure the site, to mitigate further movements and to assure safety of public and maintain railway 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.
  -

2.2 MTO QEW Crossing

Based on the monitoring of ground movement as specified in Subsection 1, 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 Geotechnical Engineer, and RV Anderson 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 Geotechnical Engineer, and RV Anderson 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, RV Anderson, and the land owner (CNR or MTO as applicable).