GEOTECHNICAL MEMORANDUM FOR DESIGN - CAWTHRA ROAD SANITARY SEWER REPLACEMENT (PHASE 3), MISSISSAUGA, ONTARIO

PEEL PROJECT 15-2300

REGIONAL MUNICIPALITY OF PEEL

REVISION 1

PROJECT NO.: 171-08406-04
DATE: MAY 14, 2020

WSP
2 INTERNATIONAL BLVD, SUITE 201
TORONTO, ON
CANADA M9W 1A2

T: +1 416 798-0065
F: +1 416 798-0518
WSP.COM

SIGNATURES

LICENSED PROFESSIONAL ENGINEER
K. SINGH
100052179
2020-05-14
PROVINCE OF ONTARIO

Kulbir Singh, M.Eng., P.Eng.
Senior Engineer, Geotechnical

LICENSED PROFESSIONAL ENGINEER
L.F.Cao
100097270
2020-05-14
PROVINCE OF ONTARIO

Laifa Cao, Ph.D., P.Eng.
Principal Engineer, Geotechnical

WSP Canada Inc. prepared this report solely for the use of the intended recipient, Regional Municipality of Peel, in accordance with the professional services agreement. The intended recipient is solely responsible for the disclosure of any information contained in this report. The content and opinions contained in the present report are based on the observations and/or information available to WSP Canada Inc. at the time of preparation. If a third party makes use of, relies on, or makes decisions in accordance with this report, said third party is solely responsible for such use, reliance or decisions. WSP Canada Inc. does not accept responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken by said third party based on this report. This limitations statement is considered an integral part of this report.

The original of this digital file will be conserved by WSP Canada Inc. for a period of not less than 10 years. As the digital file transmitted to the intended recipient is no longer under the control of WSP Canada Inc., its integrity cannot be assured. As such, WSP Canada Inc. does not guarantee any modifications made to this digital file subsequent to its transmission to the intended recipient.

TABLE OF CONTENTS

TABLES

DRAWINGS AND APPENDICES

1 INTRODUCTION

WSP Canada Inc. (WSP) was retained by the Regional Municipality of Peel (Region of Peel) to undertake a geotechnical investigation for the Cawthra Road Sanitary Sewer Replacement - Phase 3, in the City of Mississauga within the Region of Peel, Ontario. The project involves design and construction of sanitary sewer along Burnhamthorpe Road East, Wilcox Road, Tomken Road, and Runningbrook Drive. Associated works include new manholes and connections to the existing City infrastructure. The proposed sanitary sewer alignments and project details are summarized in **Table A1, Appendix A**. The general plan showing the proposed alignments is shown on the attached **Drawing No. D1**.

The objectives of the investigation were to determine the subsurface conditions at the designated locations by means of twenty-one (21) exploratory boreholes, installation of monitoring wells, and geophysical survey (ground penetrating radar), and to provide associated geotechnical recommendations for the design and construction of the proposed sewers. The "Geotechnical Data Report for the "Cawthra Road Sanitary Sewer Project - Phase 3" dated April 3, 2020 [1] includes review of regional geology and available geotechnical data, the method of investigation, the field and laboratory work, the soil and rock geotechnical laboratory test data, groundwater level records, and descriptions of the subsurface conditions encountered during the investigation.

This memorandum interprets the ground and groundwater conditions as relevant to the geotechnical design of the open cut and trenchless sections of proposed alignment and provides geotechnical parameters and recommendations. In addition, the construction conditions are evaluated to assist the Region of Peel and its designers in establishing constructability for detail design. This memorandum must be read in conjunction with the aforementioned geotechnical data report.

This report deals with geotechnical aspects of the site only. A Phase I & II Environmental Site Assessment (ESA) and Hydrogeological investigation were carried out by WSP concurrently with the geotechnical investigation. The results of these investigations are reported under separate covers.

This report has been prepared for Region of Peel and its designers. Third party use of this report without WSP Canada Inc. consent is prohibited.

In the following, the north direction is taken towards the City of Brampton (left of the true north) and perpendicular to the Burnhamthorpe Road East.

2 GEOTECHNICAL INTERPRETATION AND RECOMMENDATIONS

In this section, the subsurface conditions are interpreted as relevant to the design and construction of the proposed sewers at the aforementioned site. Comments relating to construction are intended for the guidance of the Region of Peel and its designers to establish constructability.

The construction methods described in this report must not be considered as being specifications or direct recommendations to contractors, or as being the only suitable methods. Prospective contractors should evaluate all of the factual information, obtain additional subsurface information as they might deem necessary and should select their construction methods, sequencing and equipment based on their own experience in similar ground conditions. The readers of this report are also reminded that the conditions are known only at the borehole locations and in view of the generally wide spacing of the boreholes, conditions may vary significantly between boreholes.

2.1 OVERVIEW OF SUBSURFACE CONDITIONS AND RECOMMENDED GEOTECHNICAL PARAMETERS

2.1.1 OVERVIEW OF SUBSURFACE CONDITIONS

In general, as indicated by the geology maps, subsurface conditions at this site was found to be fairly uniform. The typical stratigraphic sequence consists of pavement structure or topsoil over fill (mainly cohesive) underlain by native deposits (in descending order): glacial till (mainly cohesive till) and till/shale complex. The till/shale complex overlies shale bedrock, interbedded with thin layers of siltstone and limestone, of the Georgian Bay Formation at shallow depths ranging from 3.1 m to 12.2 m. **The elevation on top of the bedrock varied between 122.0 m and 141.5 m** across the investigated holes where weathered shale was contacted.

The Bedrock was cored in seventeen (17) boreholes with coring advanced exceeding 2.5 m in length within the investigated depths. Along the east-west direction of the proposed alignment, the highest bedrock/inferred surface was encountered at Elev. 141.5m at BH2-20 (west end of Wilcox Road) and the bedrock surface dips east to Elev. 131.3 m at BH2-49 (near intersection of Tomken Road and Runningbrook Drive. Along the north-south direction of the alignment, the highest bedrock/inferred surface was encountered at Elev. 137.8m at BH2-42 (Burnhamthorpe Road East) and the bedrock surface dips both towards north to Elev. 122.0m at BH3-02 (near Little Etobicoke Creek). The inferred bedrock surface level should not be considered accurate to better than ±1.5m. It is noted that undulations in the bedrock surface level between borehole locations should be expected.

For the design purpose, the upper 1.5m of the bedrock should be considered as the weathered bedrock and the bedrock at 1.5m below the bedrock surface can be considered as the sound bedrock.

At the monitoring well locations, the observed groundwater level across the site was between 0.5 m (Elev. 143.4 m) and 4.4 m (Elev. 132.2 m) based on the measurements between January 11, 2018 and March 8, 2018, on February 25, 2020 and March 3, 2020. Perched water should be expected in the shallow granular fill and in any granular fill in existing nearby utility trenches.

Borehole locations and subsurface profiles along the proposed alignment are presented in **Drawing Nos. D1** through **D13**.

2.1.2 RECOMMENDED DESIGN PARAMETERS FOR SOIL, BEDROCK AND GROUNDWATER

Recommended soil parameters (unfactored) for the design of tunnel, chambers/manholes and ground support systems are summarized in **Table 2.1**. The suggested soil parameters are based on field and laboratory test results and supplemented by the judgement based on local and regional experience with these soil types.

Table 2.1 Recommended Unfactored Soil Parameters (*)

SOIL TYPE	NEW GRANULAR FILL		EXISTING FILL	NON COHESIVE NATIVE SOILS		COHESIVE NATIVE SOILS			
				SAND AND GRAVEL AND SILTY SAND/SAND TO SANDY SILT (TILL)		SILTY CLAYEY TO CLAYEY SILT (TILL) AND TILL/SHALE COMPLEX			
SPT'N'	'A'	'B'	2- >30	44-50	>50	10-14	15-29	30-50	>50
Unit weight (kN/m^3)	22	21	19.5	22	22.5	20.5	21	21.5	22.5
Effective angle of internal friction (º), ϕ'	35	32	28	34	37	28	30	32	34
Effective cohesion, c' (kPa)	-	-	-	-	-	2	5	10	10
Undrained shear strength (kPa) (**)	-	-	50	-	-	75	100	200	300
Coefficient of Lateral Earth Pressure Active, K_a	0.27	0.31	0.36	0.28	0.25	0.36	0.33	0.31	0.28
At rest, K_o	0.43	0.47	0.53	0.60	0.80	0.53	0.50	0.60	1.00
Passive, K_p	3.69	3.25	2.77	3.54	4.03	2.77	3.00	3.25	3.54
Elastic modulus (MPa)	-	-	4	40	50	8	15	30	50
Poisson's ratio	-	-	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Vertical Modulus of subgrade reaction, kv (MN/m^3) (***)	-	-	4/B	40/B	50/B	8/B	15/B	30/B	50/B
Horizontal modulus of subgrade reaction, kh (MN/m^3) (***)	-	-	4/B	40/B	50/B	8/B	15/B	30/B	50/B

(*) For locations, depths and detail descriptions of specific soil layer, see GDR.
(**) The undrained shear strength is recommended for the design of manholes and ground support system, not for the selection of excavation machine, for which the undrained shear strength up to 1000 kPa should be considered.
(***) B is the width of footing or width of shaft in metres.

Recommended rock parameters (unfactored) for the design of chambers/manholes and ground support systems are summarized in **Table 2.2**. The suggested rock parameters are based on the laboratory testing on the rock core samples and supplemented by judgement based on local and regional experience with this rock formation.

Table 2.2 Recommended Unfactored Rock Parameters

PARAMETERS	WEATHERED BEDROCK (*)	SOUND BEDROCK
Unit Weight, γ (kN/m^3)	24.0 – 24.5	25.5 – 26.0
Unconfined Compressive Strength of Intact Rock (MPa) (**)	10 – 20 (average 15)	20 – 100 (average 30)
Young's Modulus of Rock Mass (GPa)	0.5 – 5 (average 2)	1 – 15 (average 8)
Poisson's ratio	0.20 – 0.25	0.2 – 0.24
Major In-situ Horizontal Stress, ho (MPa) (oriented at an azimuth of 0.07°)	2 – 3 (average 2.5)	3 – 7 (average 5)
Minor In-situ Horizontal Stress, h_o (MPa)	1 – 2 (average 1.5)	1 – 5 (average 3)
Free Swell Strain in Horizontal Direction	0.01 – 0.05%/Log cycle of time	0.01 – 0.1%/Log cycle of time
Free Swell Strain in Vertical Direction	0.05 – 0.4%/Log cycle of time	0.1 – 0.75%/Log cycle of time
Swell Strain in Horizontal Direction under 100 kPa confined pressure	0.0 – 0.01%/ Log cycle of time	0.0 – 0.02%/ Log cycle of time
Swell Strain in Vertical Direction under 100 kPa confined pressure	0.02 – 0.05%/Log cycle of time	0.05 – 0.1%/Log cycle of time
Geological Strength Index	50	60
Intact Rock Constant, Mi	4	8
Hoek's Envelope Constant: Intact Compressive Strength Dilation parameter M_b (peak/residual) S (peak/residual) (***) A (peak/residual)	 10 – 20 (average 15) 0 1.006/0.671 0.0039/ 0.0039 0.50/0.50	 20 – 100 (average 30) 0 1.438/0.959 0.0117/0.0117 0.50/0.50
Slake Durability Index, $I_{d(2)}$	N/A	41 – 85%
CERCHAR Abrasiveness Index, CAI	N/A	2.2 – 4.0

(*) The weathered bedrock must be taken as not less than 1.5m below bedrock surface.
(**) The unconfined compressive strength is recommended for the design. For the selection of equipment for bedrock excavation, consideration should be given to the highest value of 141 MPa for the limestone or siltstone.
(***) Using Disturbance Factor, D=0 for tunnel excavation using TBM.

For the design purpose, the groundwater level must be taken as 1m higher than the measured groundwater level in the nearest monitoring wells installed within the overburden as shown in Drawing Nos. D2 through D13.

2.1.3 COBBLES AND BOULDERS

Boulders/cobbles or floating (rafted) bedrocks were inferred based on auger grindings and refusal SPT blow counts in the glacial tills and in the till/shale complex. A very slow rate of drilling advancement was experienced during augering of these deposits given their heavily overconsolidated nature and presence of cobbles/boulders or rafted bedrock. The current investigation method of borehole drilling could not determine the size and frequency of the boulder/cobbles or rafted bedrock.

Cobbles are defined as rock fragments that cannot pass through a screen with 75mm square openings and are less than 300mm in maximum dimension. Boulders are defined as rock fragments with their maximum dimension being equal to or greater than 300mm. Removal of cobbles during excavations is considered part of routine construction and these materials will not be considered as obstructions for this project.

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

The majority of boulders within the till and till/shale complex are expected to be generally less than 1m in diameter; however, boulders with maximum dimensions of between 2 and 3m have been encountered in excavations in the till and till/shale deposits of Southern Ontario. Boulder and cobble shall comprise intact, rounded rock with unconfined compressive strength up to 250 MPa.

2.1.4 FROST DEPTH AND PROTECTION

The frost depth for the project site area is 1.2 m based on the OPSD 3090.101. All pipes must have at least 1.2m of earth cover for frost protection. If it is not feasible to bury these facilities below the frost penetration depth of 1.2 m, insulation will be required. Insulation should be protected by a minimum of 300 mm of cover.

2.2 SEWER INSTALLATION USING OPEN CUT

The anticipated behaviour of the soil and rock as related to the support of the pipe and the stability of open cut excavations are summarized on **Table 2.3** and are also briefly discussed in the following sections.

Table 2.3 Soil Behaviour in Open Cut

SOIL TYPE	PIPE SUPPORT	STABILITY DURING CONSTRUCTION IN OPEN EXCAVATION	POSSIBLE MEANS OF GROUNDWATER CONTROL
Fill material	Not suitable	Stable at 1.5H:1V	Gravity drainage and pumping from filtered sumps established inside the base of trench.
Silty clay to clayey silt or sandy silt to silty sand (till, till/shale complex)	Satisfactory	Stable at 1H:1V (*)	
Shale embedded with limestone/siltstone	Satisfactory	Near-vertical (**) (1H:10V)	

(*) above groundwater table.
(**) Benches in bedrock cuts may require welded wire mesh of fibre-reinforced shotcrete and rock bolting

2.2.1 TRENCH EXCAVATION AND DEWATERING

Based on the presently available information, excavations will be to depths between 3.5 m and 5.5 m within the fill, cohesive clayey soil, glacial till and till/shale complex or shale bedrock can be expected in the open cut sections.

While general stability requirements for varying soil conditions are outlined below, the Occupational Health and Safety Act (OHSA) of Ontario stipulates that any excavation deeper than 1.2m must be shored or cut back at a slope of 1V:1H or flatter, depending on the soil type.

EXCAVATION IN OVERBURDEN SOILS

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA) of Ontario. Due to the dense to very dense/stiff to hard condition of the glacial till, till/shale complex and the presence of cobbles, and boulders in these deposits, excavation progress in the overburden soils may be slow. Heavy equipment will be required. As these deposits just overlie the bedrock, boulders and rock slabs/blocks with maximum dimension of greater than 1m should be expected.

The fill would be classified as Type 3 and locally Type 4 Soil defined by the Occupational Health and Safety Act of Ontario above the groundwater table and Type 4 Soil below the groundwater table. Very stiff to hard cohesive glacial tills and till/shale complex fall into the category of Type 1 to Type 2 Soils. At any given trench location, the Soil Type with the highest Type number governs.

Unsupported excavations would be temporarily stable for a few days at an angle of 1.5H:1V in the existing fill material and at 1H:1V in dense to very dense/stiff to hard till, and till/shale complex.

Vertical cuts in any of the soil types should be supported with shoring and bracing to safeguard the stability of the sides of the trenches. The earth pressures on the shoring system should be evaluated by using the pressure distribution diagrams shown on Drawing No. D14. All shoring designs should be in accordance with the 4th Edition of the Canadian Foundation Engineering Manual and must be reviewed by a professionally qualified geotechnical engineer.

Shoring walls may be required to support adjacent structures. The requirement for shoring wall supporting adjacent structures is given on Drawing No. D15. As the excavation will mainly be within the stiff to hard cohesive soils, there will be no issue of a base heave due to excavation.

More specifically, from the records of the boreholes, the following potential problem areas have been identified:

- Perched water should be expected in the boundary between the overburden and bedrock;
- The rate of excavation in “hard” or “very dense” soils with unconfined compressive strength up to 2000 kPa at depth will be slow and laboured and heavy machine will be required. Cobbles/boulders/rock slabs with an unconfined compressive strength up to 250 MPa within the glacial till and till/shale complex could also present difficulties.
- Granular base/granular fill material containing perched water and water bearing layers at shallow depths are expected to be encountered. Depending on the thickness of these layers, locally flatter slopes are expected to be required. Alternatively, the walls of the trench should be shored and braced, or, where appropriate, trench boxes may be employed for Types 2 and 3 Soils with trench depths less than 6m. Where trench depths exceed 6m or in Type 4 Soils of any trench depth, Engineered Support Systems are required under the OHSA as defined in the Regulation. In such cases, the use of prefabricated support systems (trench boxes) is not recommended.
- Unstable conditions are also expected where the trench is near previous excavations and existing buried utilities, particularly if these were backfilled with granular materials. This condition is expected to be fairly common throughout the alignment.

EXCAVATION IN BEDROCK

The bedrock can generally be excavated without blasting. The top weathered and weaker portion of the shale bedrock can generally be removed with a powerful excavator equipped with rock buckets and rock teeth, assisted by hoe ramming. The removal of the underlying fresh and stronger rock and especially the interbedded limestone and siltstone layers and fair to excellent rock (RQD $\geq$ 50%), however, will be arduous and time consuming, and will require use of impact breakers and line-drilling. The relative ease/difficulty in excavation of bedrock will also depend on the size (width) and depth of the excavation.

Excavations cut into the bedrock can be on a near-vertical face. The face of the excavation, however, must be scaled of any loose rock to protect the workers working in the excavation. Rock cuts deeper than 1.5m, should also be provided with welded wire mesh and rockbolts or shotcrete and rockbolts for worker safety. Rock cuts that will remain open for more than 3 months should be shotcreted to minimize slaking and desiccation deterioration. Shotcrete must be drained to prevent build-up of hydrostatic pressures. Strips of drainage membrane and weeper holes will be needed behind the shotcrete.

TRENCH BOXES

Where permissible under the OHSA and where its use is considered to be a safe alternative for shoring and bracing, contractors may decide to utilize trench boxes for temporary trench wall support for trenches less than 6m deep in Type 2 and 3 Soils. Where trench depths exceed 6m, Engineered Support Systems are required under the OHSA as defined in the Regulation. In such cases, the use of prefabricated support systems (trench boxes) is not permitted.

While the use of trench boxes is an effective and economical trench-support method, its use can cause increased loss of ground relative to properly braced shoring, especially when working close to granular base courses below existing pavements or along existing utility trenches backfilled with granular materials. Trench boxes also reduce the contractor's ability to compact backfill materials placed between the trench wall and the outer trench box shell, thereby increasing the likelihood of post-construction settlements along the trench walls.

It is important that the trench not be over-excavated to ensure a tight fit between the box and the trench walls. Trench boxes need to be installed expediently. When moving the box, the void space between its outer walls and the trench must be backfilled and compacted. This may require raising the box sequentially prior to sliding it laterally.

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

CONSTRUCTION DEWATERING

The rate of groundwater seepage through the overburden soils at the open cut sections is expected to be slow to moderate and can be handled by gravity drainage and pumping from filtered sumps established at the base of the excavation. Increased dewatering efforts using well points may locally be required when the excavation reaches the granular material and sandy layers embedded within the till and shale/till complex below the groundwater table.

For hydrogeological study and the discussions of Ministry of Environment, Conservation and Parks (MECP) Permit To Take Water (PTTW) or Environmental Activity and Sector Registry (EASR) for the construction dewatering, please refer to the separate "Hydrogeological Data Report and Impact Assessment, Cawthra Road Sanitary Sewer and Watermain Project 17-24525, Phase 3, Mississauga, ON, April 2020" [2].

Soil fines suspended in water which accumulates in trenches will require removal by clarification / sedimentation before discharge using enviro-tanks or other means.

In order to minimize any long term drainage effects caused by the sewer installation, it is recommended that clay/silty clay trench cut-off rings (i.e. "trench plugs") be constructed around the pipe and through the bedding at intervals of 50m. Use of concrete in place of clay/silty clay is not recommended for trench plug construction due to point loading concerns.

2.2.2 PIPE SUPPORT AND BEDDING

The borehole records indicate that the majority of the soils/bedrock encountered at the invert of the proposed PVC sewer pipes (open cut method) are capable of providing adequate pipe support using Region of Peel Granular "A" bedding. The subgrade condition must be inspected and verified by geotechnical personnel. If weak or otherwise unsuitable fill/soils materials are present at the proposed pipe invert or trench invert elevation, the unsuitable fill material should be sub-excavated and replaced using OPSS Granular "A". The replacement fill should be placed in loose lifts not exceeding 150 mm in thickness and then compacted to 100% of Standard Proctor Maximum Dry Density (SPMDD) at a placement water content of ±2% of optimum. Each loose layer shall be compacted to 100% SPMDD prior to the placement of the next upper layer.

The bedding material and its minimum thickness for the pipes should be in accordance with the current revision of Region of Peel Standard Drawings. If concrete is used as bedding material or the pipe is installed in a narrow trench excavated in the rock (the clearance between pipe and trench wall less than 300mm), a 100mm thick layer of compressible material (e.g. EPS GeoSpan Compressible Fill) must be placed between the concrete and the bedrock to compensate rock squeeze as discussed in Section 2.4.3.

With respect to backfill above the pipe zone, please refer to Section 2.5 of this report.

2.2.3 THRUST BLOCK BEARING RESISTANCE

An allowable (or SLS) horizontal and vertical bearing resistance of 50 kPa and factored ULS bearing resistance of 75 kPa can be used in the design of thrust blocks constructed at depths between 2.0 to 3.0m on undisturbed native soils. An allowable (or SLS) horizontal and vertical bearing resistance of 75 kPa and factored ULS bearing resistance of 110 kPa can be used in the design of thrust blocks constructed at a depth of 3.0m or deeper on undisturbed native soils. For the thrust blocks installed within the sound bedrock, an allowable (or SLS) bearing resistance of 1000 kPa and factored ULS bearing resistance of 1500 kPa can be used in the design.

2.2.4 MOVEMENTS, MONITORING AND CONDITION SURVEY

Open excavation will result in ground displacement adjacent to the excavation with maximum values occurring adjacent to or near the excavation perimeter, reducing with increasing distance from the excavation edge. Ground surface movements due to shaft/trench construction within the overburden soils are dependent on many factors and most important factors are, soil type and condition, wall type, construction procedure, and workmanship. Deformation of supported excavations will cause both vertical (settlement) and lateral movements. In general, the maximum lateral and vertical movements are of the same order of magnitude and can be considered equal. Maximum settlements at or near the excavation edge with properly engineered and constructed support systems can be considered typically less than a value equal to 0.3% of the depth of excavation with maximum lateral displacement of an equal magnitude. For example, for a 5m deep excavation within the fill overburdens, the maximum ground settlement is expected to be in the order of 15mm. The width of the zone of influence for the settlements extends horizontally from the excavation can be assumed to be two times of the depth of excavation.

A ground settlement monitoring network may be warranted in some areas of the open cut excavations depending on proximity of existing utilities and roads as well as weaker ground conditions, such as soft to firm cohesive fill. Ground settlement monitoring may also be required during construction dewatering. WSP can propose a monitoring plan and specification once the details of open cut and construction dewatering are finalized.

A preconstruction condition survey of the existing utilities, pavement, sidewalk curbs, culverts and buildings in the immediate vicinity of the proposed alignment should be carried out prior to the start of construction.

Baseline noise and vibration survey and monitoring during construction are also recommended.

2.3 SEWER INSTALLATION USING TUNNELLING/ TRENCHLESS METHODS

The subsurface conditions encountered in boreholes at the locations of the proposed tunnel/trenchless sections are summarized in **Table 2.4**.

Table 2.4 Generalized Subsurface Conditions at Tunnel/Trenchless Sections

LOCATION	PROP INVERT ELEV (m)	APPROX. LENGTH (m)	DRAWING NO.	BH NO.	SOIL/ROCK TYPE AT PROPOSED TUNNEL DEPTH	SOIL / ROCK COVER (m)	WATER TABLE IN MONITORING WELLS	ANTICIPATED GROUND BEHAVIOR
1500mm Sanitary along Burnhamthorpe Road (Sta. 3+093 to Sta. 3+180)	121.07 – 120.93	87.3	D2	BH 2-15 BH 2-16	Georgian Bay Formation of shale with limestone /siltstone	15.8 to 16.0 (1)	2.6m below existing grade (Elev. 140.9m) (4)	Bedrock
1200mm Sanitary along Burnhamthorpe Road (Sta. 3+180 to Sta. 4+590)	121.17 – 124.27	1411	D2 and D6 to D10	BH 2-41 BH 2-42 BH 2-43 BH 2-44 BH 2-45 BH 2-46 BH2-51 BH3-01 BH3-02 BH3-47	– Georgian Bay Formation of shale with limestone /siltstone Till/Shale complex Sandy Silt Till Fill	– 7.1 to 16.0 between Wilcox Road and Tomken Road (2a) – Generally, 3.5 to 10.0 between Tomken Road and Little Etobicoke Creek Crossing (2b)	2.0m to 3.3m below existing grade (Elev. 141.9 to 127.5m) (4)	– Bedrock at Boreholes BH2-41 to BH2-46 and BH2-51 – Bouldery; firm or slow ravelling to flow at BH3-01, BH3-02 and BH3-47(6)
900mm Sanitary along Tomken Road (Sta. 14+364 to Sta. 14+495)	128.62 – 127.96	131.4	D13	BH2-46 BH2-47	Georgian Bay Formation of shale with limestone /siltstone	2.5 to 3.5(3)	2.0m to 4.0m below existing grade (Elev. 134.3m to 132.3m) (5)	Bedrock

Notes:

(1) Rock cover for 1500 mm sanitary sewer installed in 1800 mm diameter tunnel.

(2a) Rock cover for 1200 mm sanitary sewer installed in a 1500 mm diameter tunnel.

(2b) Soil cover for 1200 mm sanitary sewer installed in a 1500 mm diameter tunnel.

(3) Rock cover for 900 mm sanitary sewer installed in a 1100 mm diameter tunnel.

(4) Groundwater level measured in overburden wells.

(5) Groundwater level measured in both bedrock well and in overburden well

(6) The indicated "anticipated ground behaviour" is based on Terzaghi's tunnelling classification system (Refer to **Appendix C** for definitions of ground performance in tunnelling). It relates to stand up time and primary tunnel lining installation. It does not relate to the ease or difficulty expected in excavating the soil. Tunnelling in soil and rock

2.3.1 TUNNELLING IN BEDROCK

1500MM SANITARY SEWER ALONG BURNHAMTHORPE ROAD (STA. 3+093 TO STA. 3+180)

The length of the 1500mm diameter sanitary along Burnhamthorpe Road is about 87.3 m (**Drawing No. D2**). A 1.8m diameter tunnel proposed in the design is expected to be mined through bedrock for the installation of 1500mm dia. sanitary. The rock cover shown in **Table 2.4** is considered to be sufficient.

At the proposed tunnelling depths along Burnhamthorpe Road, the tunnel will be mined through fresh shale and limy shale with interbedded limestone and siltstone rock characterized by RQD values ranging between 76% and 97% (Drawing No. D2) suggesting a "Good" to "excellent" rock quality using ISRM's classification. Fractural zones should be expected along most of the tunnel length. Also, a broken zone was encountered within the tunnel zone near the end at BH2-15. Significant groundwater seepage may occur in these fractural/broken rock zones and grouting or other dewatering methods may be required.

1200MM SANITARY SEWER ALONG BURNHAMTHORPE ROAD EAST (STA. 3+180 TO STA. 4+560)

The length of the 1200mm diameter sanitary sewer along Burnhamthorpe Road East is approximately 1411m (**Drawing Nos. D2** and **D6** to **D10**).

The tunneling along Burnhamthorpe Road East between Wilcox Road and Tomken Road will mined through bedrock, and between Tomken Road and Little Etobicoke Creek, the tunneling will be mined through mixed ground condition from bedrock to sandy silt till and fill.

Between Wilcox Road and Tomken Road: The rock cover shown in **Table 2.4** for tunneling is considered to be sufficient (assuming sanitary sewer installed in a 1.5m diameter tunnel). At the proposed tunnelling depths along Burnhamthorpe Road East, the tunnel will be mined through mostly fresh shale and limy shale with interbedded limestone and siltstone rock characterized by RQD values ranging between 59% and 100% (**Drawing Nos. D2** and **D6** to **D10**), suggesting a "fair" to "excellent" rock quality using ISRM's classification. Fractural zones should be expected along most of the tunnel length. Also, a broken zone was encountered within the tunnel zone near the start at BH2-51. Significant groundwater seepage may occur in these fractural/broken rock zones and grouting or other dewatering methods may be required.

Between Tomken Road and Little Etobicoke Creek: The soil cover above the tunnel varies approximately between 3.5 m and 10m based on the borehole data or about 2.3 times the mined tunnel diameter. This is considered to be sufficient soil cover. At the Creek Crossing at Sta. 4+570, the bedrock is well below the tunnel invert and soil cover above the tunnel is approximately 3.5m or about 2.3 times the mined tunnel diameter. This is considered to be sufficient soil cover and we consider the potential impact of tunnelling to the creek to be very low.

At the proposed tunnelling depths at this section, the tunnel will be mined through from high weathered/fresh bedrock to very dense sandy silt to silty sand till and firm to stiff silty clay fill with boulders/cobble and rock slabs. For the purpose of tunnelling, the compact to very dense very dense sandy silt to silty sand till with boulders/cobbles can be categorized as "flowing" and "bouldery" ground, and the firm to stiff silty clay with boulders/cobbles can be categorized as "firm", "slow ravelling" and potential "bouldery" ground in accordance with the behaviouristic ground classification system established by Terzaghi in 1950 (Refer to **Appendix C** for definitions of ground performance in tunnelling). The silty clay fill should be considered as "sticky" clay for tunnelling.

900MM SANITARY SEWER ALONG TOMKEN ROAD (STA. 14+364 TO STA. 14+495)

The length of the 900mm diameter sanitary sewer along Tomken Street is approximately 131.4m (**Drawing No. D13**). The rock cover shown in **Table 2.4** is considered to be sufficient (assuming sanitary sewer installed in a 1.1m diameter mined tunnel).

At the proposed tunnelling depths, the tunnel will be driven through mostly fresh shale and limy shale with interbedded limestone and siltstone rock characterized by RQD values ranging between 46% and 87% (Drawing No. 13), suggesting a "poor" to "good" rock quality using ISRM's classification. Fractural zones should be expected along most of the tunnel length. Significant groundwater seepage may occur in these fractural rock zones and grouting, or other dewatering methods may be required.

It would be prudent to expect some variability in the rock quality between the boreholes.

TUNNELLING METHODS AND TEMPORARY SUPPORT REQUIREMENTS

In carrying out the tunnelling through bedrock, consideration should be given to:

(1) the mixed face conditions consisting of clayey fill, sandy till and clayey till/shale complex with boulders/cobbles and bedrock slabs, and bedrock of shale and limestone/siltstone below the groundwater table;

(2) possible surface settlement due to relatively shallow tunneling through soils at some locations;

(3) the high in-situ horizontal stresses existing in the rock, causing compressive stress concentrations at the invert and crown of the opening and tensile stresses at the sidewalls;

(4) the expected long term deformations causing the rock to squeeze into the tunnel;

(5) the possibility of encountering combustible gas; and

(6) the tendency for the shale to 'slake' or deteriorate over time when exposed to air.

To preserve the stability of the roof of the tunnel and minimize surface settlement due to shallow tunneling through soils, as a minimum measure the soil and rock cover above the tunnel obvert should be maintained at 1.5 times the tunnel diameter and the rock between the 10 and 2:00 o'clock positions should receive full support within the tail shield immediately following tunnelling.

The temporary tunnel lining system for soil and bedrock condition must be designed to accommodate and protect tunnel personnel from:

(1) Stress induced instability due to yielding in the rock surrounding the tunnel, given the high in-situ horizontal to vertical stress ratio;

(2) Structurally controlled instability given the sub horizontal bedding plane partings and sub vertical fractures set in the rock mass releasing blocks or wedge of rock and falling cobbles/boulders or bedrock slabs in glacial till and till/slab complex; and

(3) Control of physical rock deterioration by slaking.

The soil and rock parameters proposed in **Table 2.1** and **Table 2.2** can be used for the design of the temporary tunnel lining systems.

The selection of the equipment and method to remove the soil and bedrock, advance the tunnel and type of temporary ground support should be the contractor's choice. In the selection of these, the contractor should consider the soil and bedrock condition including mixed face condition, cobbles/boulders or bedrock slabs in glacial till and till/shale complex, the strength range of the intact shale rock which was measured to range from 1 to 48 MPa, on the basis of which the rock is classified as "very weak" to "medium strong". Consideration should also be given to the fact that the rock mass contains numerous limestone/siltstone layers which could range in thickness between 50 and 310mm, and that the strength of these layers is in the 19 to 141 MPa range ("medium strong" to "very strong"). For the advancement of the tunnel, a wide range of methods could be considered:

(1) Microtunnelling with a Microtunnel Boring Machine (MTBM);

(2) Roadheader Tunnelling;

(3) Tunnel Boring Machine (TBM);

(4) Tunnelling with Small Boring Unit (SBU); or

(5) Horizontal drilling and Back Reaming;

MICROTUNNELLING WITH A MTBM

Microtunneling is a method whereby a jacking pipe is pushed into a bore, mined remotely using a tunnel boring machine head fixed to the lead pipe segment. The MTBM is launched from the entry shaft by pushing a jacking pipe using hydraulic jacks and a reaction frame that are installed in the entry shaft as the cutterhead rotates. The jacking pipe is installed in segments by cyclic pipe jacking. It is recommended that an intermediate jacking station be installed every 75m to reduce the jacking forces on the main jack while pushing the pipe forward. Bentonite lubrication of the annular spaces will be also needed to reduce frictional resistance to jacking and to close up the overcut.

Spoils removal is performed by mixing the excavated material in the front chamber of the MTBM head to the consistency of a slurry with conditioning using bentonite, water and other conditioning agents and then pumping this slurry back to the launching shaft where it is de-sanded and thickened. The rate of slurry removal from the chamber is carefully controlled and matched to the advance rate of pipe jacking such that the predicted lateral earth pressures are balanced with the slurry pressure. A large laydown area is needed for the de-sanding support plant.

This method is suitable for jacking pipes with inner diameters ranging from 600mm to 2400mm. Considering the proposed sanitary sewer will have uncased diameters of 900mm to 1500 mm, the microtunneling with a MTBM could be considered for the sanitary sewer installation. Intermediate jacking stations will be required.

The MTBM must be capable of dealing with a “mixed face” of very weak to medium strong shale and limy shale interbedded with harder layers of medium strong to very strong limestone/siltstone. Local experience with tunnelling systems that rely only on picks or harden cutter teeth to cut through bedrock suggest that cutting ability is poor and such work has failed and replacement systems that use disc cutters are required. Access to the face is not provided for MTBM less than 1.5m in diameter. Therefore, the disc cutters and other face tools must be selected to perform without excessive wear for the entire drive since wore tools will stop advance of the MTBM and require extraction with a rescue shaft. Particular attention must be paid to the section of the perimeter gage disc cutters since they are subject to higher abrasion wear or bearing failure compared to the other disc cutters.

Some zones of shale will produce more clay-rich ‘muck’. This clayey muck can be ‘sticky’ and can foul up some cutterheads and conveyance equipment. The use of polymers may be needed to reduce the stickiness.

During microtunnelling in bedrock, debris can get trapped in the overcut resulting in increasing friction, spikes in jacking load and increased thrust load on the MTBM cutters. Jacking pipes (steel or concrete) specifically designed for use in rock should be used on this project. To minimize the potential for debris to obstruct the MTBM, use of an overcut, or distance between the outside diameter of the jacking pipe and excavation, that is larger than normally used for soft ground tunnelling is recommended. The overcut must be sufficient to accommodate long term squeeze of the shale bedrock. The gap between rock cut surface and casing should not be less than 40mm for the tunnel greater than 1500 mm diameter and 30mm for the 1.1m tunnel along Burnhamthorpe Road using a bigger cutter than the outside diameter of jacking pipe. The overcut should be immediately filled with bentonite slurry of an appropriate viscosity in order to stop groundwater flow along the pipe string. A seal will be required to close the annulus between the wall of the entry/exit shaft and the shield and jacking pipes to retain groundwater behind the temporary shoring and stop backfill of the slurry into the shafts.

Overall, microtunnelling is considered to be the method that minimizes the risk of ground losses and the ground surface settlement and would not require dewatering along the tunnel alignment at this site. The jacking pipe can be used as temporary ground support as well as the permanent pipe. However, it is relatively expensive to mobilize this type of machine.

The primary risks associated with this trenchless method are encountering resistance limestone/siltstone layers along the tunnel invert that would impact the vertical and horizontal alignment, reduce production rates or halt advance if the MTBM is not properly selected or outfitted. Groundwater flow along the excavated annulus is also a concern.

ROADHEADER TUNNELLING

Roadheader tunnelling may be considered for the tunnel diameter of 1500 mm or greater. A small size of roadheader is required for the proposed approximately 1500 mm to 1800 mm dia. tunnel. The selected roadheader should be able to excavate the harder layers of the medium strong to very strong limestone/siltstone as well as be able to deal with the very weak to medium strong shale and limy shale.

Primary temporary tunnel support should be provided immediately following tunnelling. The tunnel face must be fully supported when the excavation is hauled. The choice of the primary temporary tunnel support should be the responsibility of the contractor. Traditionally, rock support in the Georgian Bay Formation has taken the form of full steel ribs and timber lagging and/or partial ribs and lagging spanning only the crown supported by rock bolts. Pattern rock bolting with wire mesh was also successfully used on a number of projects. There has been some successful experience with rock bolt and fibre-reinforced shotcrete lining systems in GTA as well, although skilled nozzlemen and shotcrete additives are required to minimize rebound and maximize the compaction of the applied shotcrete product. Provision for drainage behind the shotcrete is also necessary. Past experiences with tunnelling in this rock formation indicate that in order to prevent cone shaped rock pieces falling out at the obvert, the tunnel support must be installed immediately behind the tunnel face.

Groundwater infiltration into the tunnel should be manageable using conventional sumps and pumps within the shafts. An uphill drive is typical in order to maintain the tunnel self-draining. If zones of fractural bedrock are encountered, bedrock grouting/reinforcement is required. The collected water will be laden with rock 'flour' and will require clarification. The rock 'flour' or 'slimes' will also have a tendency to clog sumps and pumps, thus frequent flushing and maintenance will be needed.

The Georgian Bay Formation is known to contain pockets of combustible gas and as such flame/explosion proof equipment should be utilized for underground construction and excavation work. The monitoring of the gas in the tunnel should be a mandatory part of the contract and proper ventilation systems will be required.

TUNNEL BORING MACHINE (TBM)

TBM is used to excavate tunnels with a circular cross-section through a variety of ground conditions. Typical bedrock TBM consists of "open face" machines which do not provide positive support at the tunnel face. At the TBM moves forward, the round cutter head cut into the tunnel face and splits off rock fragments. Conveyor belts carry the rock shavings (tunnel muck) through the TBM and out the back of the machine.

The selected TBM must be able to excavate through the harder layers of the medium strong to very strong limestone/siltstone as well as be able to deal with the very weak to medium strong shale and limy shale. Primary temporary tunnel support should be provided for immediately following tunnelling. The control of the groundwater and gas is similar to that using Roadheader Tunnelling.

TBM is not suitable for the relatively smaller dia. tunnel proposed along Tomken Road. Therefore, TBM is considered as a suitable method for larger tunnel diameter, such as 1500 mm to 1800 mm.

TUNNELLING WITH SMALL BORING UNIT (SBU)

The SBU often consist of a rotating cutterhead system that is temporarily mounted to the lead end of a steel casing or a concrete jacking pipe and are effective in penetrating the rock type in this area. The ground is cut using a variety of face tools (similar to MTBM described before), but the spoil is generally transported to the working pit/shaft using auger system, much like conventional jack and bore systems. Face openings on the SBU are typically much smaller than the auger opening on conventional jack and bore system and the risk of uncontrolled ingress of ground into the lead end of the casing/jacking pipe is lower for this system as compared for jack and bore methods. These systems do not, however, provide consistent and positive support to the face openings with any slurry or cuttings, unlike the slurry-based MTBMs described before. Construction dewatering is required if saturated fractural zones of bedrock is encountered.

HORIZONTAL DRILLING AND BACK REAMING

This method is similar to horizontal directional drilling (HDD), which involves drilling a small holes (i.e. a pilot bore) from the initial working location (pit/shaft/surface), back-reaming the bore to enlarger it, pulling a casing/linear and finally pushing the pipe through the lined bore, except this particular method involves drilling and reaming from inside a shaft as compared to conventional HDD processes, which enter and existing the ground surface at ground surface or in shallow pit. This particular trenchless method involves the drilling of a pilot bore along the proposed tunnel alignment, typically in the order of 0.2 to 0.3m in diameter, and then enlarging the pilot bore to the required size (i.e., final diameter) by one or more reaming passes. Cuttings from the reaming are blown and transported back to the entry shaft using compressed air. The selection of the reaming equipment is dependent on the type and strength of the bedrock. Frequently, this approach relies upon the ground (soil or rock) through which the pilot and reamed hole pass to remain stable and open without the support of drilling fluids, casings, pipes or other lining systems.

The downhole tolling must be adapted for rock and should include mud motors equipped with milltooth tri-cone bits, drag bits or the like. It may be necessary to use a hole opener or percussive reamer instead of a conventional reamer to enlarge the bore within the rock, particularly if more resistant bands of limestone/siltstone are encountered.

The shale bedrock is susceptible to loss of circulation and built up of excess pressure since it relatively resistant to absorption of the drilling fluid. Line and grade problems may occur if "hard layers" of limestone/siltstone are encountered along the bore.

Overall, horizontal drilling and back reaming, as described above, is capable of creating a bore/tunnel through bedrock and the set-up times associated with this method are generally shorter compared to MTBM and SBU; however, depending on the specific equipment, there may be little alignment control during drilling and there is a greater chance of misalignment during back reaming compared to the other methods described above. This method should be feasible for the sanitary sewer installation along Tomken Road due to its relatively small size, though there remains a greater risk of rock falls and large inflows of water through fracture zones during reaming and pull-back as compared to either the microtunnelling or small boring unit methods as described above.

2.3.2 ASSESSMENT OF SANITARY SEWER INSTALLATION BY TRENCHLESS METHODS

The advantages, disadvantages, and risks/consequences associated with these trenchless construction methods are compared in **Table 2.5** on the basis of anticipated ground conditions, depth of cover, vertical and horizontal alignment, length of pipe installation, availability of equipment, and level of risk of successfully completing the installation. The method with the highest degree of suitability for use at this site is Microtunnelling with a MTBM.

Table 2.5 Evaluation of Trenchless Sanitary Sewer Installation

INSTALLATION METHOD	ADVANTAGE	DISADVANTAGES	RISKS /CONSEQUENCES
Microtunnelling with a MTBM	• Slurry machine is able to counterbalance earth and groundwater pressure in a controlled manner, providing continuous face support and eliminating need to dewatering. • Can be steered continuously, providing precise control over line and grade. • Concrete protection casing can be installed simultaneously as excavation progresses and used as permanent pipe. • No compressible grout between the concrete pipe and bedrock is required if the overcut is sufficient to accommodate long term squeeze of the shale bedrock and the overcut is filled with bentonite slurry. • Potential effects on structures and underground utilities next to the tunnel alignment can be better controlled than other method.	• Requires largest working space since slurry processing systems/ separation plants are required along with additional working area at shaft/pit locations for some systems. • Small diameter MTBM (<1.5m) does not allow cutter changes during the drive since there is no access to the head. • Potential for steering difficulties if harder limestone/siltstone layers encountered. • Debris in overcut could lock up or wedge machine.	• Relatively low risk of ground loss during tunnelling when a counterbalancing viscous slurry pressure is used. • Greater risk of fluid losses to surface compared to other methods which do not utilize slurry, but the potential of fluid losses to the surface depends on slurry composition, viscosity, pressure and the existence of available pathways (old boreholes or wells, utilities, etc.).

INSTALLATION METHOD	ADVANTAGE	DISADVANTAGES	RISKS /CONSEQUENCES
Roadheader Tunnelling	• Provides access to the tunnel face. • Relative short set-up times compared to MTBM, TBM, and SBU. • Relatively low costs compared to MTBM and TBM.	• Requires large working space at shaft locations. • Potential difficulties if harder limestones/siltstone encountered. • Requires primary temporary tunnel support. • Not applicable for 1.1m diameter tunnel along Tomken Road.	• Greater risk of settlement induced damage to nearby infrastructure and underground utilities in present of highly weathered rock. • Highly fractured bedrock zones, where encountered, may require additional groundwater control and tunnel support.
Tunnel Boring Machine (TBM)	• Can be steered continuously, providing precise control over line and grade • Produces smooth tunnel walls which facilitates tunnel support/lining. • Provides access to the tunnel face.	• Requires large working space at shaft locations. • Potential difficulties if harder limestones/siltstone encountered. • Requires primary temporary tunnel support. • Not applicable to 1.1m diameter tunnel along Tomken Road.	• Greater risk of settlement induced damage to nearby infrastructure and underground utilities in present of highly weathered rock. • Highly fractured bedrock zones, where encountered, may require additional groundwater control and tunnel support.
Tunnelling with Small Boring Unit (SBU)	• Steerable to a limited degree as compared to MTBM, provides a degree of control; over line and grade (however, SBU is better than traditional jack and bore methods). • Concrete protection casing can be installed simultaneously as excavation progresses and used as permanent pipe. • Least affected by differences in rock strength within the tunnel face.	• Does not provide positive support pressure to excavation face. • Potential difficulties if harder limestone/siltstone layers encountered. • Not applicable to larger diameter tunnel, such as 1500 mm to 1800 mm.	• Greater risk of settlement induced damage to nearby infrastructure and underground utilities in present of highly weathered rock.
Horizontal Drilling and Back Reaming	• Relative short set-up times compared to MTBM and SBU. • Relatively low costs compared to MTBM. • Pilot hole can provide information on rock behaviour prior to back reaming operation.	• Little directional control during drilling. • No lining during pilot hole drilling and remaining rock must be stable so that the pipe can be installed after reaming. • Potential for steering difficulties if harder limestone/siltstone layers encountered. • Loss of circulation due to hydrolock (increased pressure behind the bit due to the instability of the shale to absorb the drilling fluid) may occur. • Potential for settlement which may affect adjacent buried utilities. • Not applicable to larger diameter tunnel, such as 1500 mm to 1800 mm.	• Greater risk of misalignment during back reaming compared to MTBM and SBU. • Greater risk of settlement induced damage to nearby infrastructure and underground utilities in present of highly weathered rock.

The contractor should be fully responsibility for the selection of the tunnelling technology, which best fits the contractor requirements and his equipment, experience and staff availability. All tunnelling work should be carried out by an experienced specialist contractor employing only qualified workers skilled in their trade under the direction of an experienced foreman.

Contractors bidding on this project should be required to submit their tunneling plan, methodology, specifications which are critical to successful undertaking of the project. Tunneling must not commence until these documents have been approved by the Geotechnical Engineer and all technical requirements of the Geotechnical Engineer in that regard have been met.

2.3.3 FINAL LINING DESIGN AND GROUT

The rock parameters proposed in **Table 2.2** can be used for the design of the permanent pipes.

In recognition that the shale surrounding the tunnel has long-term time-dependent deformation properties and that it will "squeeze" into the tunnel opening, it is recommended that the annular void around the permanent pipes be grouted only after a minimum delay time of 3 to 4 months following excavation using a compressible low strength, low modulus grout material (such as foamed or cellular grout).

For the permanent concrete pipe installed using microtunnelling with MTBM, the gap between rock cut surface and pipe should not be less than 40mm for the tunnel greater than 1500 mm diameter and 30mm for the 1.1m tunnel and gap is filled by bentonite slurry using a bigger cutter than the concrete pipe, for which grouting is not required.

2.3.4 SHAFTS

We understand that the excavation to the base of the shafts will be vertical due to the limited space at this site. Vertical cuts in the overburden soils should be supported with shoring. Circular and rectangular shafts are often constructed in the shale bedrock. The upper overburden and weathered rock portion of the shafts (i.e. to depths of 2 to 8m) might be constructed using cast-in-place concrete ring segments sunk down from the top of the excavation (caisson sinking method), or alternatively, using continuous caisson (secant pile wall) method. Shafts constructed in this manner would be essentially watertight. Use of a watertight shaft construction is preferred in order to mitigate seepage from perched water in the fill and along the overburden to bedrock interface. The shaft walls need to be designed for hydrostatic water pressure for the groundwater level, taken as the higher of 1m higher than the measured from nearby monitoring wells.

The earth pressure distribution shown on **Drawing No. D14** should be utilized for multiple strut or tie-back shored system. The earth pressure distribution for shafts constructed using the caisson sinking method needs to be calculated using the coefficient of lateral earth pressure at rest, K_o as provided in **Table 2.1**. All shoring designs should be in accordance with the 4th Edition of the Canadian Foundation Engineering Manual and must be reviewed by this office. Allowable bond stress for rock anchors is 400 kPa (at least 3m into sound rock). An allowable bearing capacity of 3000 kPa can be used for caissons which are installed at least 1m into sound bedrock. The anticipated ground surface surcharge must account for construction machinery.

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

No loss of ground should be permitted during augering for caisson piles and the drilling contractor should be warned of the potential for encountering overburden obstructions within the till and till/shale complex such as cobbles, boulders and/or rock slabs. The use of liners will be required to keep the augered holes open before concrete placement. In the event that loss of ground occurs during lagging installation, the depth of excavated lifts must be reduced. In critical areas, the lost soils must be replaced by grouting. Monitoring of shored wall deflections by means of survey targets is recommended in areas where settlements could damage existing utilities, infrastructures, or buildings.

The excavation into fresh, sound bedrock can be done using near-vertical sidewalls provided that:

- All OHSA requirements regarding worker safety are met during the course of the work;
- The bedrock excavation is stepped-in from the inside face of the secant pile wall by at least 1.2m and the rock in the pile toe zone (which is offering passive restraint to the piles) is continuously reinforced at each pile, or closer spacing, by means of a row of inclined rock anchors fitted with steel bearing plates designed to resist the pile toe reactions. Alternatively, the contractor may opt to extend the caisson piles to minimum 1m below the base of the excavation in order to avoid the 1.2m benching at the bedrock surface.
- The rock face is scaled of all loose and potentially spalling material (including slaked rock as the excavation faces dry out over time) and is fully covered with a welded wire mesh (or at least 60mm of fibre-reinforced shotcrete) plus rock bolts. The rock bolt embedment length can be 2m at 1m spacing. Provision of drainage behind the shotcrete is needed by means of Delta membrane or similar. The shotcrete facing is recommended in light of the significant depth of the shaft, the safety hazard of falling rock, potential for ice build-up and the relatively high slaking potential of the shale under repeated cycles of wetting and drying (the 2nd Cycle Slake Durability Index of twelve test shale samples was found to be in the range of 41% to 85%).

Refer to **Section 2.2.1** for detailed discussions for the excavation.

The rate of groundwater seepage through the bedrock is expected to be slow to medium and can be handled by gravity drainage and pumping from filtered sumps established at the base of the shaft. The fractured bedrock may need to be grouted to stop groundwater seepage in the worst case, but in the best case it may be dry. Based on the groundwater measurements in three deep monitoring wells installed deep into bedrock (BH2-43, and BH2-46), the observed groundwater levels are relatively shallow; ranged from 0.7 m to 4.3 m or at Elev. 132.2 m to Elev. 142.1 m. Regular cleaning of the sumps will be needed given the potential of fouling by rock fines.

In order to minimize any long term drainage effects caused by the shaft excavation, it is recommended that a 300mm thick clay/silty clay layer be constructed when the backfill is reached to the bedrock surface level. The clay/silty clay is not required, if u-fill is used as backfilling material.

2.4 CHAMBERS/MANHOLES

2.4.1 FOUNDATION

The proposed chambers/manholes are expected to be founded within bedrock or native soils. The bedrock possesses relatively high strength and low compressibility. Consequently, generally good to excellent foundation conditions are expected.

Chambers/manholes founded on very stiff to hard or dense to very dense native soils, can be designed for an allowable (SLS) bearing resistance of 200 kPa and factored ULS bearing resistance of 300 kPa. For the design bearing pressure, total and differential settlements are estimated to be less than 20mm and 15mm, respectively. Chambers/manholes should not be founded on fill soils. A minimum 100mm thick lean mix concrete mud slab is required. Inspection of prepared foundation by qualified geotechnical personnel is required, prior to placement of a mud slab.

Where boulders protrude into the subgrade, they should be removed and replaced with low strength concrete (10 MPa).

Footings established within the sound bedrock (minimum 1.5 m below bedrock surface), can be designed for an allowable (SLS) bearing resistance of 3000 kPa and factored ULS bearing resistance of 4500 kPa. For the design bearing pressure, total and differential settlements in the sound bedrock are estimated to be less than 5mm. A lean mix concrete mud slab must be placed on the prepared shale base after inspection and approval by qualified geotechnical personnel, to minimize slaking deterioration.

In order to minimize the development of cracks due to “rock squeeze” in the base slab of the chambers/manholes, it is proposed to incorporate a layer of granular material, such as clear crushed limestone, between the underside of the slab and the surface of the bedrock after inspection and approval by the geotechnical engineer. We recommend that the thickness of the granular layer should not be more than 200mm. It should be OPSS Granular ‘A’ or clear crushed limestone, compacted to 100% of SPMDD at placement water content within ±2% of the materials optimum.

The chambers/manholes should be designed as water-tight structures. The floor-to-wall slab intersections may be fitted with conventional water stops. The floor slabs and walls may be waterproofed using a membrane or other means.

Refer to **Section 2.2.1** for general discussions on excavation and dewatering.

2.4.2 UPLIFT PRESSURE

The structure of the chambers/manholes will extend below the present groundwater table and will, therefore, be subjected to hydrostatic uplift pressures. The design groundwater level must be taken as the higher of 1m higher than the measured groundwater level in the nearest monitoring wells installed within the overburden.

If the combination of the weight of the structure and the mobilized frictional resistance between the buried portion of the exterior walls and the backfill materials is insufficient to resist the uplift forces during any stage of the construction and/or during the operation of the structure, then a fail-safe system of grouted ground anchors is needed. For anchors installed into hard/dense soils, the allowable (SLS) bond stress between grout and soil and allowable (SLS) shear capacity of soil can be assumed to be 120 kPa. For rock anchor, the allowable (SLS) bond stress between grout and rock can be taken as 400 kPa. In calculating the required depth of embedment, the top 500mm of the surface rock should be neglected and the anchor bond length should not be less than 3m.

For a single anchor, the uplift allowable capacity, P_a should not be more than that the weight of 60º rock cone, W as shown in the standard drawing for rock anchor, **Appendix B**. For a group of anchors, the total uplift allowable (SLS) capacity, ΣPa should not be more than the weight of adjoining 60º cones, W as shown in the standard drawing for rock anchor. Below the groundwater table, the submerged weight of rock must be used.

The actual capacity of the anchors should be established by at least two (2) full scale pull-out tests (“performance test”) in accordance with Post-Tensioning Institute (PTI) guidelines but taken to 200% of working load. In the field, each installed anchor must be proof loaded to 1.33 times the design working load for the anchor, in accordance with PTI guidelines. The ground anchors should be double-corrosion protected (i.e. PTI Class I).

Friction between the exterior walls of chambers/manholes and the granular backfill materials should only be taken into account if it is absolutely certain that no excavations will be undertaken around the exterior walls any time in the future. In this case, an ultimate friction factor of 0.4 applied to the horizontal earth pressure on the wall could be used, the average coefficient of earth pressure of 0.5, and average unit weight of 21.0 kN/m^3 above the groundwater table and 11.2 kN/m^3 below the groundwater table can be used in the calculation of horizontal earth pressure. When checking the overall stability of the structure, the design should incorporate a minimum safety factor of 1.1 when using only the dead weight of the structures. The safety factor to be used for the frictional resistance should not be less than 2.0.

2.4.3 LATERAL EARTH PRESSURE

LATERAL EARTH PRESSURE IN OVERBURDEN SOILS

The earth pressure distribution on chambers/manholes backfilled with granular “B” fill can be taken as hydrostatic, i.e. which is increasing linearly with depth according to the following expression:

$$P_h = K_o.\gamma.h + K_o.q$$

where

P_h = horizontal pressure at depth h (kN/m²)

γ = unit weight of soil as shown in **Table 2.1**

h = depth below ground surface (m)

q = surcharge load at ground surface (kPa)

K_o = coefficient of lateral earth pressure at rest for a horizontal ground surface condition as shown in **Table 2.1.**

Passive earth pressure coefficients for calculation of manhole thrust resistance can be taken from Table 2.1. The K_p value for the native soil will govern.

Below the water table, the submerged unit weight of the soil should be used, and the full hydrostatic water pressure should be added. If the ground surface is not horizontal, the uneven portion can be treated as an equivalent surcharge load.

Where the structure is installed in a wide open excavation, the values of γ and K_o should be those of the backfill material. Elsewhere, where the thickness of the column of backfill material behind the wall is less than half of the buried height of the wall, the values of native soils given in Table 2.1 can be used.

LATERAL EARTH PRESSURE IN BEDROCK

Structures which extend below the surface of the bedrock and the walls of which are poured in direct contact with the bedrock will be subject to “rock squeeze”. Such practice is not recommended without installation of a compressible buffer material.

Based on in-situ stress measurements and local experience, it is known that bedrock belonging to the Georgian Bay Formation contains high horizontal stresses (Table 2.2, and [3, 4, 5, 6, 7]). As a result of the relief of this high horizontal stress, significant elastic displacements occur during and after the excavation. Of these, the long term, time dependant displacements are of greater importance. These are estimated to be of the order of 0.05% of the height of the excavation per log cycle of time (e.g. 5mm per log cycle of time (in days) for a 10m deep excavation or about a total of 21mm over a period of 50 years) [8, 9, 10, 11]. Approximately 50% of the displacement (i.e. 10mm) is expected to occur during the first 100 days following excavation.

The chambers/manholes should not be designed to resist these displacements. Rather a layer of compressible material must be placed between the structure and the rock. This compressible layer could be either a synthetic material (e.g. EPS GeoSpan Compressible Fill) or foamed “cellular grout”. Properties and proposed thicknesses of the compressible material should be submitted to a qualified engineer to evaluate its stiffness and assess its suitability to accommodate the strain without transferring unacceptably high loads onto the structure. Certain rigid polystyrene insulation products are considered to be excessively stiff for this application.

Provided that rock squeeze is allowed to dissipate by delaying construction of permanent concrete walls or by applying a compressible void former, the lateral earth pressures acting on this bedrock portion of the chambers/manholes below the overburden can be assumed to be a uniform pressure equal to the maximum overburden lateral earth pressure calculated at the overburden to rock interface, plus the hydrostatic forces even if lean concrete is used as backfill material.

2.4.4 SEISMIC SITE CLASSIFICATION

Based on the borehole information and according to Table 4.1.8.4.A of OBC 2012, the subject site for the proposed manhole/chamber structures can be classified as 'Class C' for seismic site response.

2.5 GEOTECHNICAL QUALITY OF EXCAVATED SOILS

Reference to the borehole logs suggests that the excavated materials with respect to their compaction characteristics can be divided into four groups:

Group 1 comprises the granular road-base materials encountered near the surface. These materials are expected to have good compaction characteristics and could be reused as trench backfill provided that they are carefully segregated from the more silty or clayey soil strata. Some drying of the sand excavated from below the groundwater level will likely be required. There are limited quantities of these materials available.

Group 2 comprises mainly the native silty clay to silty clay (till) but locally silty sand (till) and till/shale complex and have moisture content very close to or above its optimum water content. This material will excavate in clods and would thus require a heavy pad footed compactor or hoe pack to break it down and adequately compact it. Given the water content of the till, it may not be possible to obtain a degree of compaction of this fill much above 95% of SPMDD. This degree of compaction might be acceptable within landscaped areas above which pavements or infrastructure are not expected to be built in the future.

Group 3 comprises the excavated shale and limestone/sandstone. These materials could be used as backfill provided they are crushed to the sizes similar to Granular 'A' or 'B'. Ripped or mechanically excavated bedrock may be too coarsely graded and open graded for reuse as compacted fill.

Group 4 soils consist of unsuitable materials because of their high moisture or organic inclusions, and some of the existing fill materials especially soft to firm clayey fill. These soils should be either disposed off-site or should be used only in "soft" landscaping areas where they can be placed with nominal compaction, and where surface settlements are tolerable to the Region.

As a general requirement, all backfill material should be placed in 200 to 300mm thick loose lifts and compacted to at least 98% of SPMDD, at a placement moisture content within ±2% of the optimum. Below existing/future roads, the backfill must be Granular 'A' or 'B' material, and the top 1.5m of subgrade backfill below the underside of the pavement structure should be compacted to 100% of SPMDD. The existing road pavement structure should be reinstated. New granular must match in thickness to the underside of existing to ensure unimpeded cross drainage. Where a free-draining backfill is needed or where the backfill is needed for structural support of overlying structures, the site soils will not be suitable and OPSS Granular 'B' or 'A' sand and gravel will be required. Similarly, during work in the autumn, winter and spring months, re-use of the excavated soils as compacted fill may not be practical and imported OPSS Granular 'B' should be used.

2.6 CORROSIVITY POTENTIAL

Six (6) soil samples were analyzed for corrosivity parameters, including the resistivity of the soil, pH, Redox potential and sulphide concentration for corrosion protection to the proposed sewers. **Table 2.6** summarizes the ANSI and the American Water Works Association (AWWA) rating for the tested soil samples for their potential for corrosion towards buried grey or ductile cast iron pipe. A score of ten (10) points or more indicates potential for corrosion. The analytical data are provided in the "Geotechnical Data Report for Cawthra Road Sanitary Replacement – Phase 3" [1].

According to the ANSI/AWWA rating system, the tested samples pose relatively low to medium potential for corrosion of grey or ductile iron pipe.

Note that there may be other overriding factors in the assessment of corrosion potential, such as the application of de-icing salts on the roadway and subsequent leaching into the sub soils, stray currents, contact with sewage, etc.

Table 2.6 ANSI/AWWA Soil Corrosivity Potential

PARAMETER CONCENTRATION (ANSI/AWWA POINT RATING)								
BOREHOLE NO.	SAMPLE NO.	DEPTH, m (FROM - TO)	RESISTIVITY (ohm·cm)	PH	REDOX POT. (mV)	SULPHIDE (*mg/kg)	MOISTURE CONT. (%)	TOTAL POINTS
BH2-16	SS3	1.5 – 2.1	3140 (0)	7.89 (0)	300 (0)	<0.20 (2)	11.2 (2)	4
BH2-18	SS4	2.3 – 2.9	2500 (2)	7.91 (0)	315 (0)	<0.20 (2)	9.45 (2)	6
BH2-20	SS4	2.3 – 2.7	4370 (0)	8.12 (0)	241 (0)	<0.20 (2)	7.18 (2)	4
BH2-43	SS5	3.1 – 3.7	4930 (0)	7.86 (0)	297 (0)	<0.20 (2)	14.4 (2)	4
BH2-46	SS5	3.1 – 3.3	4550 (0)	7.89 (0)	295 (0)	<0.20 (2)	7.16 (2)	7
BH2-49	SS5	3.1 – 3.2	2100 (5)	7.98 (0)	260 (0)	<0.20 (2)	9.27 (2)	9

2.7 CONCRETE EXPOSED TO SULPHATE ATTACK

The sulphate (SO4) resistance of the concrete in contact with the soils was evaluated by performing water-soluble sulphate tests on six (6) soil samples taken from selected boreholes at depths from 1.5m to 3.7m below grade. The test results are shown on the following **Table 2.7**. The analytical data are presented in "Geotechnical Data Report for Cawthra Road Sanitary Replacement – Phase 3 [1].

Table 2.7 Sulphate Test Results

BOREHOLE NO.	SAMPLE NO.	SAMPLE DEPTH (m)	WATER SOLUBLE SULPHATE CONTENT (mg/kg*)
BH2-16	SS3	1.5 – 2.1	61
BH2-18	SS4	2.3 – 2.9	24
BH2-20	SS4	2.3 – 2.7	66
BH2-43	SS5	3.1 – 3.7	26
BH2-46	SS5	3.1 – 3.3	27
BH2-49	SS5	3.1 – 3.2	30

* 1mg/kg = 1µg/g = 1ppm = 0.0001%

The tests revealed that the sulphate concentration in the tested soil samples was 24 to 66 mg/kg or 24 to 66 µg/g or 0.0024 to 0.0064%. The category of severity of attack is "negligible" based on CSA Standard CAN/CSA-A23.1, Concrete Materials and Methods of Concrete Construction. The final selection of the type of concrete should be made by the Engineer taking into account all aspects of design considerations.

2.8 ROAD PAVEMENT REINSTATEMENT

The existing road pavement structure should be reinstated. New granular must match in to the underside of existing to ensure unimpeded cross drainage.

At the borehole locations, the existing pavement structure consisted of 80 to 230mm of asphaltic concrete layer underlain by 280 to 510mm of granular base/subbase. The existing pavement structure are presented in "Geotechnical Data Report for Cawthra Road Sanitary Replacement – Phase 3 [1]. Based on the existing pavement structure data, the mean thickness of asphalt and the underlying base/sub-base course material is 144 mm and 421 mm, respectively. The range of existing pavement structure thickness is 460 mm to 740 mm with an average pavement structure of 565 mm.

Based on the asphalt thickness encountered during the borehole drilling and to satisfy the minimum requirements of the City of Mississauga (Standard No. 2220.010), the thickness of new asphalt concrete should not be less than 140mm, consisting of 40mm (Superpave 12.5FC1) and 100mm (Superpave 19.0) using subgrade condition having 55% of maximum silt.

The new granular base should consist of a minimum thickness of 150mm of Region of Peel Granular 'A' compacted to 100% of the SPMDD, overlying the required remaining thickness of Region of Peel Granular 'B' (to match-in with existing) compacted to 98% of the SPMDD. The subgrade should be prepared as discussed in Section 2.5.

As mentioned previously, if ravelling of the existing granular base is not controlled, then it will be necessary to cut back the existing asphalt by say 0.5m to 1m in order to replace and compact this material prior to placing the asphalt.

2.8.1 PAVEMENT BUTT JOINT

In order to minimize maintenance of the longitudinal asphalt joint between the existing pavement and the new asphalt placed above the sewers, we recommend milling the existing asphalt to a depth of 50mm and providing butt joint. The HL-1 (Superpave 12.5FC1) surface course asphalt should be placed over the milled surface and the new binder course asphalt in one pass.

An alternative to the above recommended method is to rout the joint between the existing asphalt and new asphalt to a depth of 40mm and seal this joint with joint sealant. Periodic maintenance of the joint will be required in either case.

If a surface course asphalt overlay is proposed for the full pavement width, shoulder to shoulder, then the above steps are not necessary.

3 LIMITATIONS OF REPORT

The Statement of Limitations, as provided in **Appendix D**, forms an integral part of this report.

LIST OF REFERENCES

[1] WSP Canada Inc., Geotechnical Data Report - Cawthra Road Sanitary Sewer Replacement (Phase 3) Mississauga, Ontario, 2020.

[2] WSP Canada Inc., Hydrogeological Data Report and Impact Assessment, Cawthra Road Sanitary Sewer and Watermain Project 17-24525, Phase 3, Mississauga, ON, April 2020.

[3] Morton, J.D., Lo, K.Y. and Belshaw, D.: "Rock performance consideration for shallow tunnels in bedded shales with high lateral Stresses", Proceedings, 12th Canadian Rock Mechanics Symposium, Kingston, Ontario, 1975.

[4] Lo, K.Y. and Morton, J.D.: "Tunnels in bedded rock with high horizontal stresses", Canadian Geotechnical Journal, Vol. 13, 1976.

[5] Franklin, J.A. and Hungr, O.: "Rock Stresses in Canada, their relevance in engineering projects", Rock Mechanics, by Springer-Verlag, 1978.

[6] Thompson, P.M.: "In situ rock stress determinations for the Burloak Water Intake Tunnel", Underground Research Laboratory, Prepared for Geo-Canada Ltd., dated December 22, 2004.

[7] Golder Associates, "Report on In Situ Stress Measurements – Hanlan Feedermain North, Mississauga, ON", Prepared for Regional Municipality of Peel, dated August 2013.

[8] Lo, K.Y., Palmer, J.H.L. and Quigley, R.M.: "Time-dependent deformation of shaly rocks in southern Ontario", Canadian Geotechnical Journal, Vol. 15, 1978.

[9] Lo, K.Y., Cooke, B.H. and Dunbar, D.D.: "Design of buried structures in squeezing rock in Toronto, Canada", Canadian Geotechnical Journal, Vol. 24, 1987.

[10] Lieszkowszky, I.P., Ng, J. and Dullerud, E.P.: "Simcoe Street sewer tunnel in squeezing shale rock in Toronto", Canadian Tunneling, 1994.

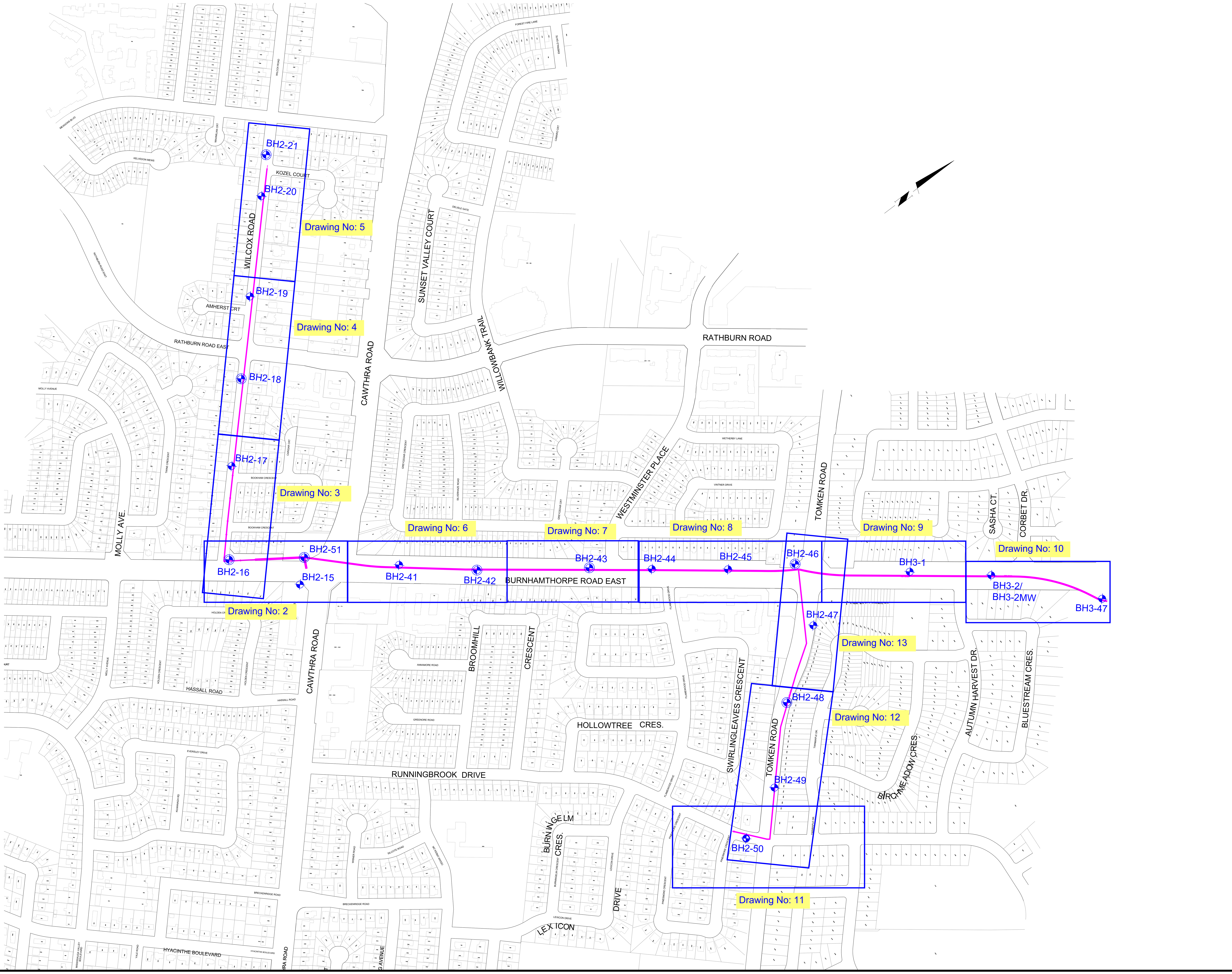
[11] Lieszkowszky, I.P., Ng, J. and Davidson, R.: "Urban tunnel construction in shale rock", 13th Annual Canadian Tunneling Conference, Montreal, 1995.

[12] Trow, W.A. and Lo, K.Y.: "Horizontal Displacement Induced by Rock Excavations: Scotia Plaza, Toronto, Ontario", Canadian Geotechnical Journal. Vol. 26, 1989.

DRAWINGS

BOREHOLE LOCATION PLAN AND GEOTECHNICAL SECTIONS (DRAWING NOS. D1 TO D13)
EARTH PRESSURE DISTRIBUTION ON BRACED EXCAVATIONS (DRAWING NO. D14)
GUIDELINES FOR UNDERPINNING IN SOIL AND EXCAVATION (DRAWING NO. D15)

2020-May-5 6:42:25 PM D:\WSP\171-08406-04 Cawthra\171-08406-04-Plan-Mar10-2020.dwg



General Notes

All Driveways Are ASPHALT Unless Otherwise Noted

All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field

All Horizontal And Vertical Bends Are In Degrees

All Pipes Size In mm

20C Existing Water Service, Size In mm

WS20 Proposed Water Service, Size In mm

B.M. No. Elev.
Description
Location

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

wsp

2 International Boulevard, Suite 201
Toronto, Ontario | M9W 1A2 | Canada
Tel.: 416-798-0065 | Fax: 416-798-0518 | www.wsp.com

Designed by Chkd.

Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

50m 0 50 100 150m

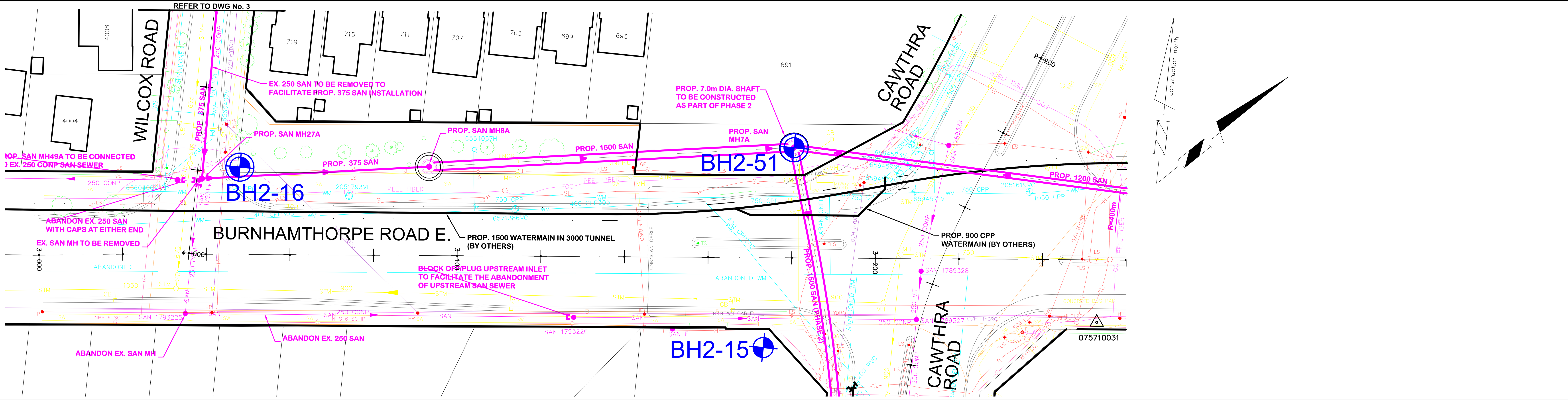
HORIZONTAL SCALE

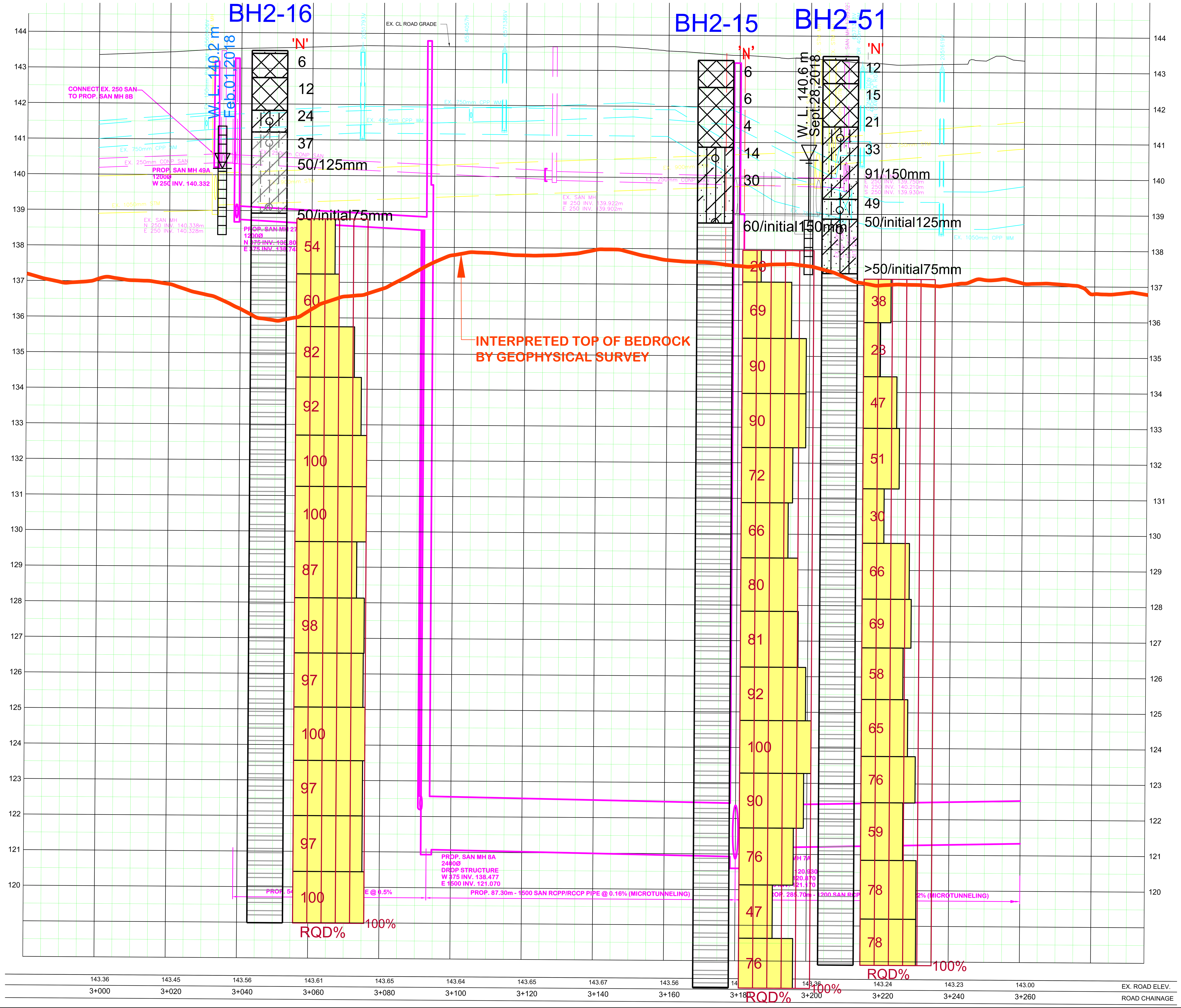
Region of Peel
Working for you

BOREHOLE LOCATION PLAN

GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

CAD Area	Area	Project No.	171-08406-04
Checked by LC	Drawn by ZMO		
Date March 10, 2020	Sheet of	Drawing No.	D1



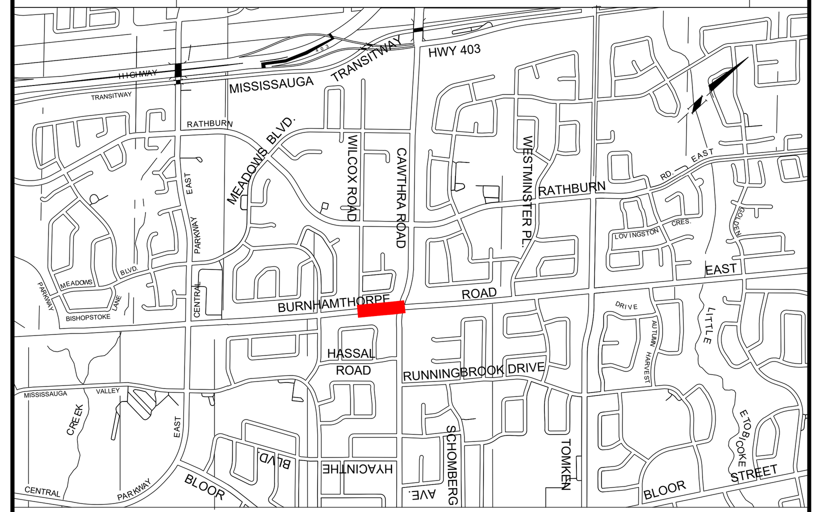


SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
MAY 16, 2017	ISSUED FOR 30% REVIEW	F.S.
JUNE 25, 2018	ISSUED FOR 50% REVIEW	JSD
PENDING	ISSUED FOR 90% REVIEW	F.S.



KEY PLAN (N.T.S.)

LEGEND

- Asphalt
- Topsoil
- Fill
- Fill gravely sand
- Sand and Gravel
- Sand
- Silt
- Silt Till
- Silty Sand
- Sandy Silt
- Silty Clay
- Clayey Silt
- Silty Sand Till
- Sandy Silt Till
- Silty Clay Till
- Clayey Silt Till
- Sand and Silt Till
- Silty Clay Till/ Shale Complex
- Clayey Silt Till/ Shale Complex
- Shale (Georgian Bay Formation)
- Borehole drilled by WSP
- Borehole with Monitoring Well drilled by WSP
- Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted
All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field
All Horizontal And Vertical Bends Are In Degrees
All Pipes Size In mm
Existing Water Service, Size In mm
WS20 Proposed Water Service, Size In mm
B.M. No. Elev.
Description
Location
The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

Designed by
Chkd.
Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE
1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Working for you

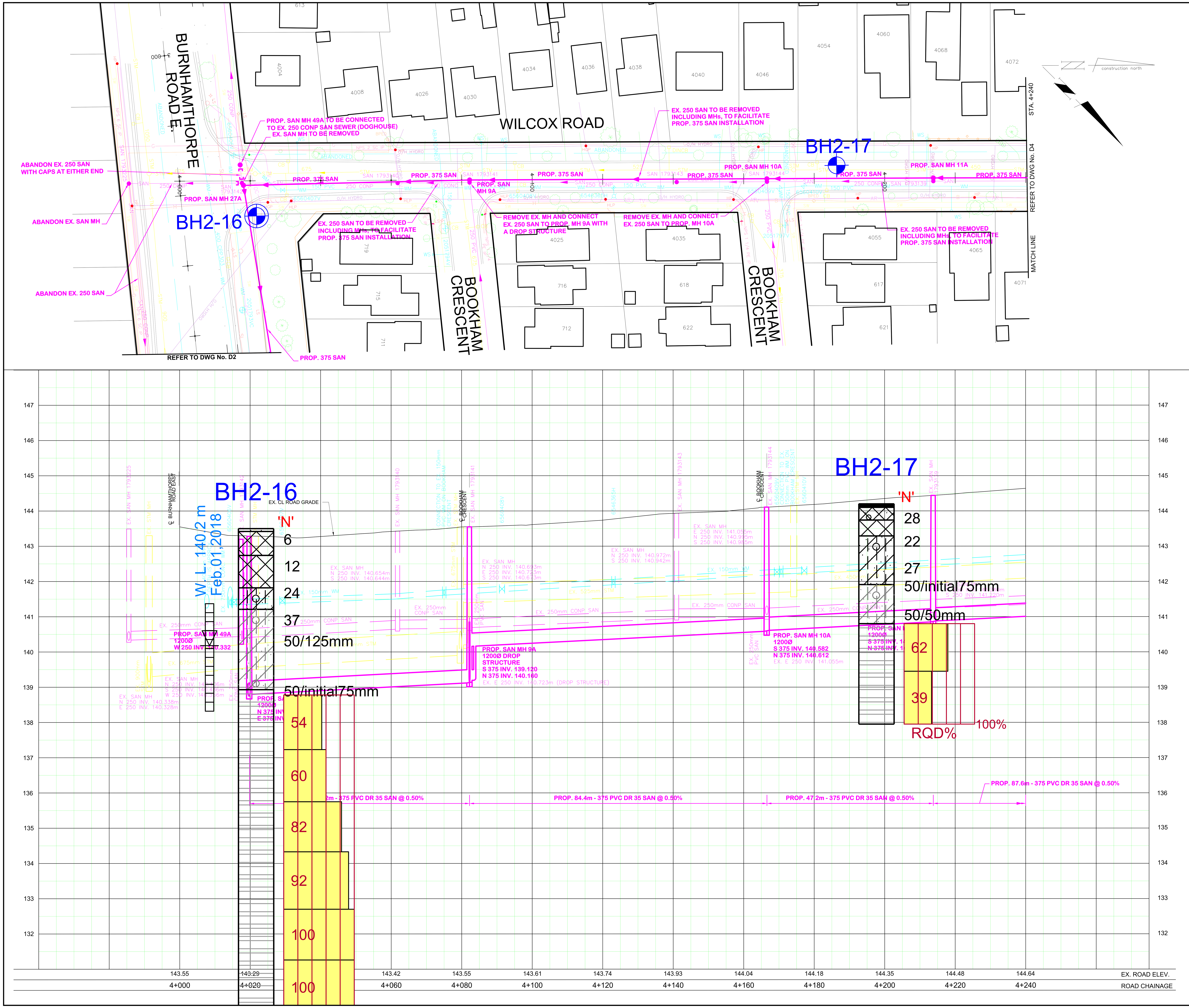
BURNHAMTHORPE ROAD E.

BOREHOLE LOCATION PLAN AND GEOLOGICAL SECTIONS
GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 3+000 TO STA. 3+280

CAD Area		Area		Project No.	171-08406-04
Checked by	LC	Drawn by	ZMO		
Date	March 5, 2020	Sheet	of	Drawing No.	D2

D:\WSP\171-08406-04 Cawthra\D2-171-08406-04-PP9-Mar5-2020.dwg



D:\WSP\171-08406-04 Cawthra\D3-171-08406-04-PP10-Mar5-2020.dwg

SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
MAY 16, 2017	ISSUED FOR 30% REVIEW	F.S.
JUNE 25, 2018	ISSUED FOR 50% REVIEW	JSD
PENDING	ISSUED FOR 90% REVIEW	F.S.

KEY PLAN (N.T.S.)

LEGEND

Asphalt
Topsoil
Fill
Fill gravely sand
Sand and Gravel
Sand
Silt
Silt Till
Silty Sand
Sandy Silt
Silty Clay
Clayey Silt
Silty Sand Till
Sandy Silt Till
Silty Clay Till
Clayey Silt Till
Sand and Silt Till
Silty Clay Till/ Shale Complex
Clayey Silt Till/ Shale Complex
Shale (Georgian Bay Formation)
Borehole drilled by WSP
Borehole with Monitoring Well drilled by WSP
Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted
All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field
All Horizontal And Vertical Bends Are In Degrees
All Pipes Size In mm
20C Existing Water Service, Size In mm
WS20 Proposed Water Service, Size In mm

B.M. No. Elev.
Description
Location

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

wsp
2 International Boulevard, Suite 201
Toronto, Ontario | M9W 1A2 | Canada
Tel.: 416-798-0065 | Fax: 416-798-0518 | www.wsp.com

Designed by ______ Chkd. ______
Approved by ______

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE
1m 0 1 2 3m VERTICAL SCALE

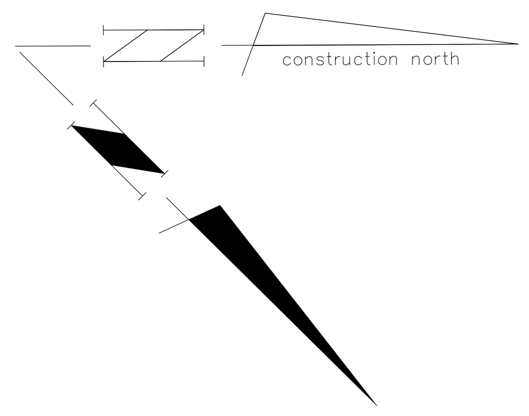
Region of Peel
Working for you

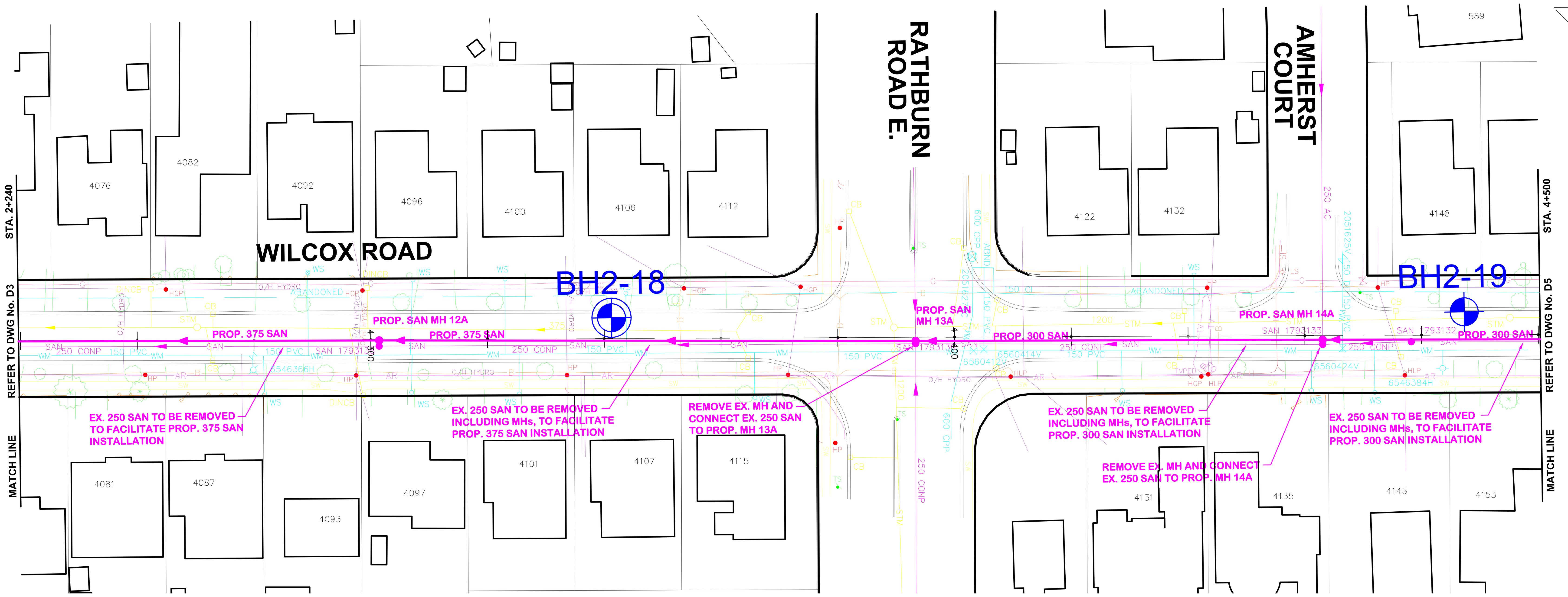
WILCOX ROAD

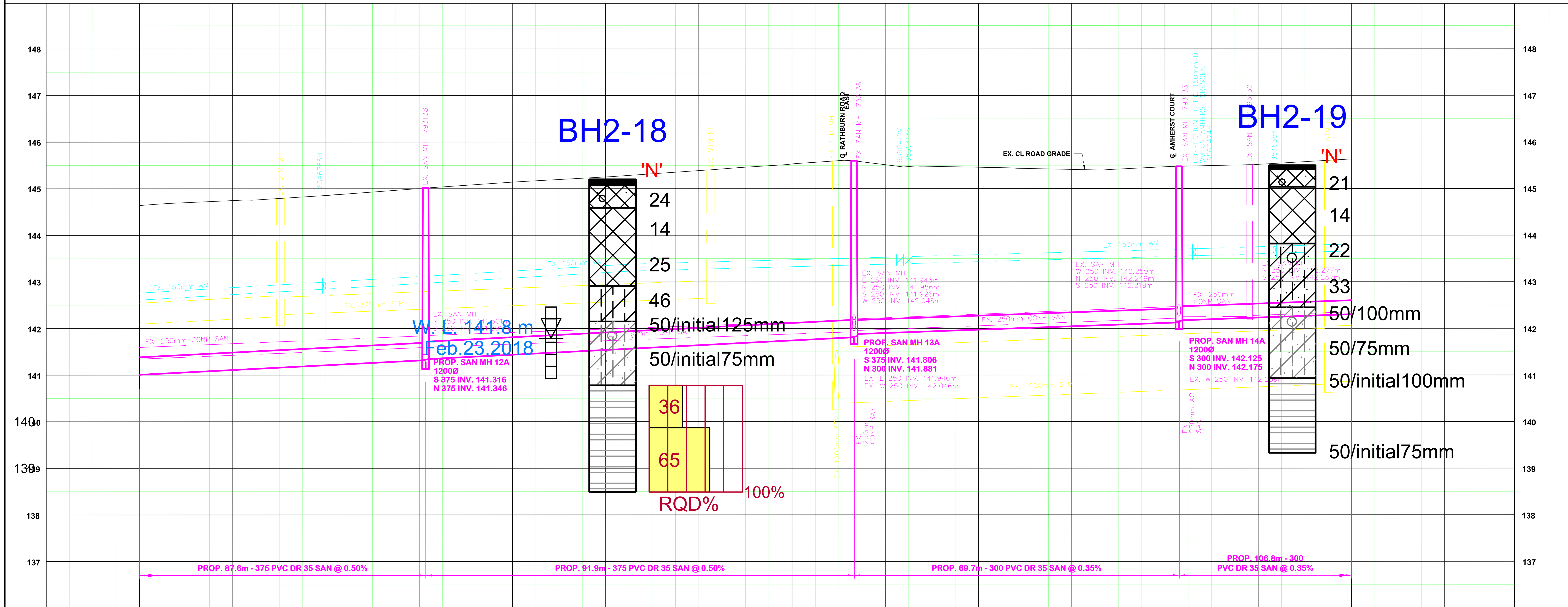
GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 4+000 TO STA. 4+240

CAD Area	Area	Project No. 171-08406-04
Checked by LC	Drawn by ZMO	
Date March 5, 2020	Sheet of	Drawing No. D3







144.64	144.75	144.85	145.00	145.14	145.25	145.39	145.53	145.51	145.46	145.41	145.47	145.52	145.63	EX. ROAD ELEV.
4+240	4+260	4+280	4+300	4+320	4+340	4+360	4+380	4+400	4+420	4+440	4+460	4+480	4+500	ROAD CHAINAGE

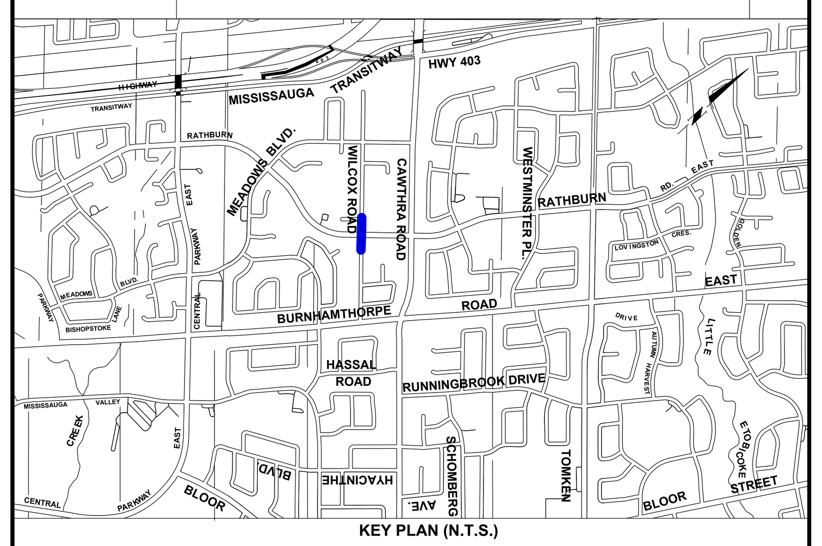
D:\WSP\171-08406-04 Cawthra\D4-171-08406-04-PP11-Mar5-2020.dwg

SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
MAY 16, 2017	ISSUED FOR 30% REVIEW	F.S.
JUNE 25, 2018	ISSUED FOR 50% REVIEW	JSD
PENDING	ISSUED FOR 90% REVIEW	F.S.



LEGEND

Asphalt
Topsoil
Fill
Fill gravely sand
Sand and Gravel
Sand
Silt
Silt Till
Silty Sand
Sandy Silt
Silty Clay
Clayey Silt
Silty Sand Till
Sandy Silt Till
Silty Clay Till
Clayey Silt Till
Sand and Silt Till
Silty Clay Till/ Shale Complex
Clayey Silt Till/ Shale Complex
Shale (Georgian Bay Formation)
Borehole drilled by WSP
Borehole with Monitoring Well drilled by WSP
Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted
All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field
All Horizontal And Vertical Bends Are In Degrees
All Pipes Size In mm
20C Existing Water Service, Size In mm
WS20 Proposed Water Service, Size In mm
B.M. No. Elev.
Description
Location

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

wsp
2 International Boulevard, Suite 201
Toronto, Ontario | M9W 1A2 | Canada
Tel.: 416-798-0065 | Fax: 416-798-0518 | www.wsp.com

Designed by
Chkd.
Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE
1m 0 1 2 3m VERTICAL SCALE

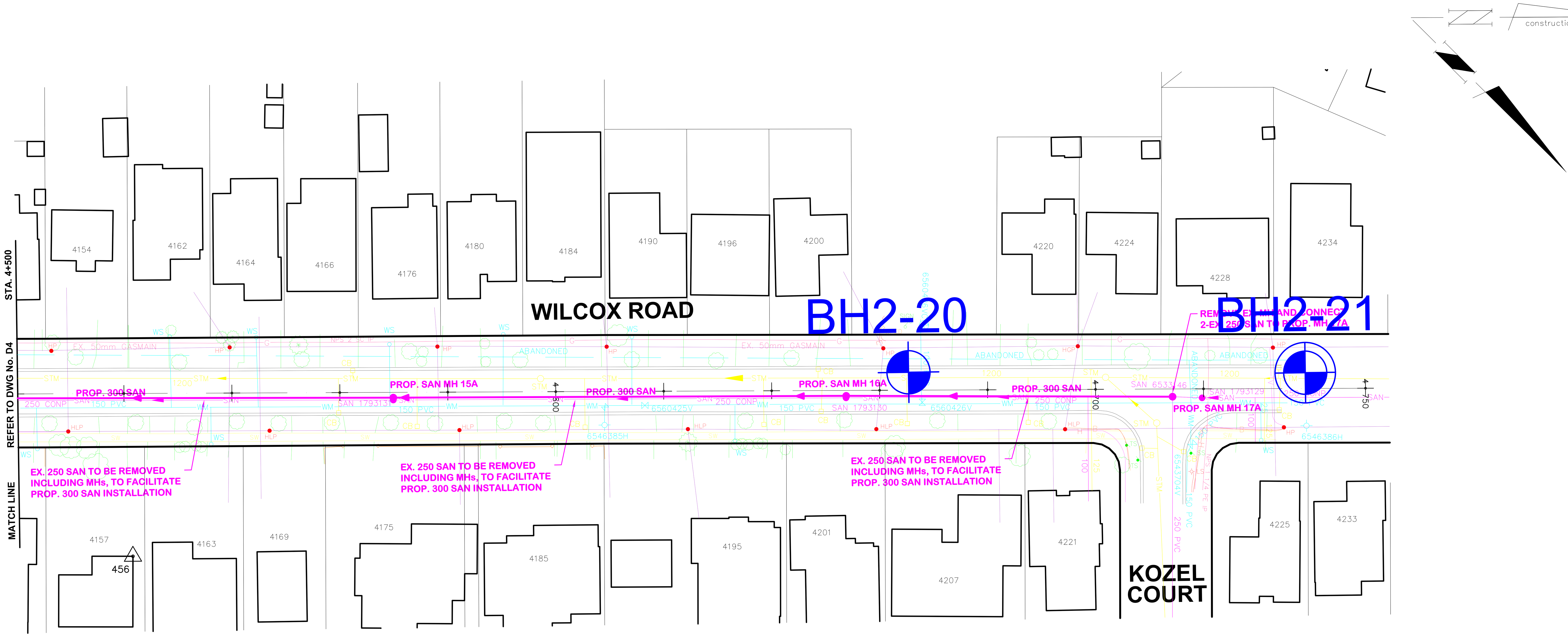
Region of Peel
Working for you

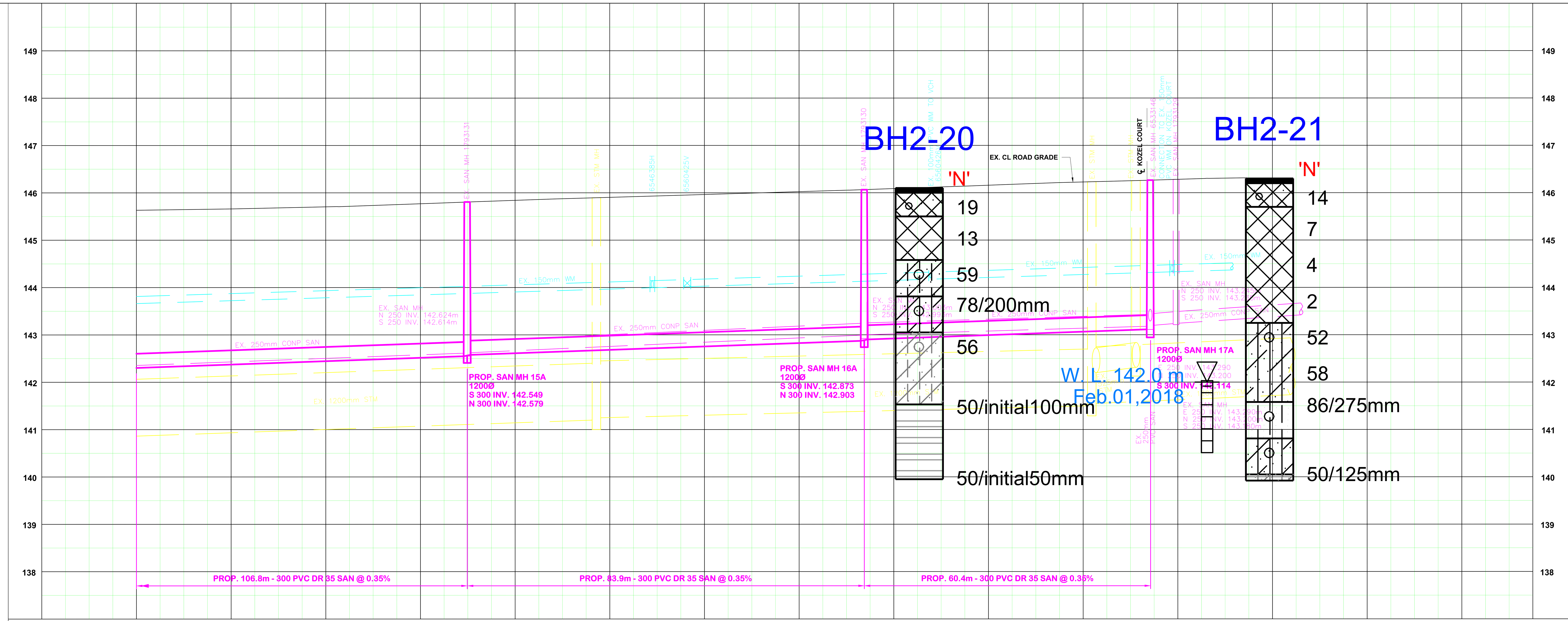
WILCOX ROAD

GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 4+240 TO STA. 4+500

CAD Area	Area	Project No. 171-08406-04
Checked by LC	Drawn by ZMO	
Date March 5, 2020	Sheet of	Drawing No. D4





145.63	145.66	145.71	145.77	145.84	145.90	145.96	146.02	146.09	146.16	146.23	146.28		EX. ROAD ELEV.
4+500	4+520	4+540	4+560	4+580	4+600	4+620	4+640	4+660	4+680	4+700	4+720	4+740	ROAD CHAINAGE

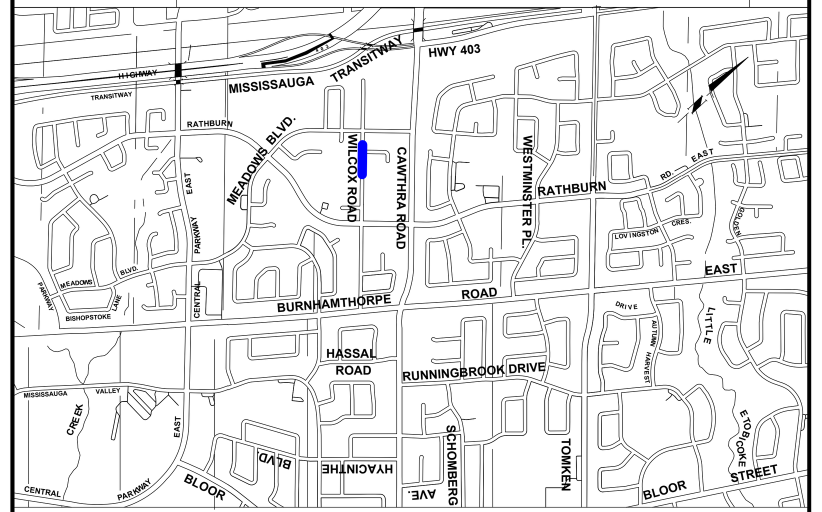
D:\WSP\171-08406-04 Cawthra\D5-171-08406-04-PP12-Mar5-2020.dwg

SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
MAY 16, 2017	ISSUED FOR 30% REVIEW	F.S.
JUNE 25, 2018	ISSUED FOR 50% REVIEW	JSD
PENDING	ISSUED FOR 90% REVIEW	F.S.



KEY PLAN (N.T.S.)

LEGEND

- Asphalt
- Topsoil
- Fill
- Fill gravelly sand
- Sand and Gravel
- Sand
- Silt
- Silt Till
- Silty Sand
- Sandy Silt
- Silty Clay
- Clayey Silt
- Silty Sand Till
- Sandy Silt Till
- Silty Clay Till
- Clayey Silt Till
- Sand and Silt Till
- Silty Clay Till/ Shale Complex
- Clayey Silt Till/ Shale Complex
- Shale (Georgian Bay Formation)
- Borehole drilled by WSP
- Borehole with Monitoring Well drilled by WSP
- Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted

All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field

All Horizontal And Vertical Bends Are In Degrees

All Pipes Size In mm

20C Existing Water Service, Size In mm

WS20 Proposed Water Service, Size In mm

B.M. No. Elev.
Description
Location

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

wsp

2 International Boulevard, Suite 201
Toronto, Ontario | M9W 1A2 | Canada
Tel.: 416-798-0065 | Fax: 416-798-0518 | www.wsp.com

Designed by
Chkd.

Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE

1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Working for you

WILCOX ROAD

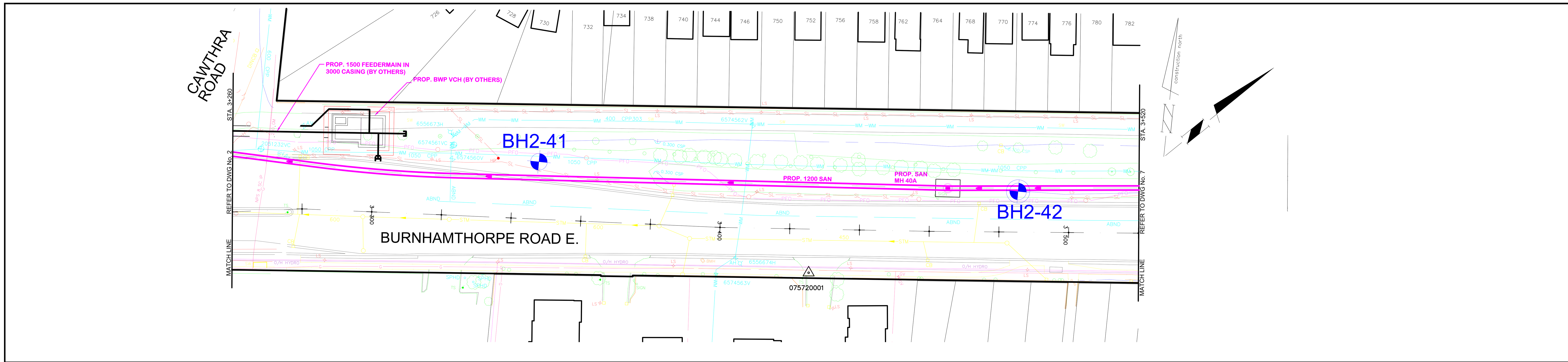
BOREHOLE LOCATION PLAN AND GEOLOGICAL SECTIONS

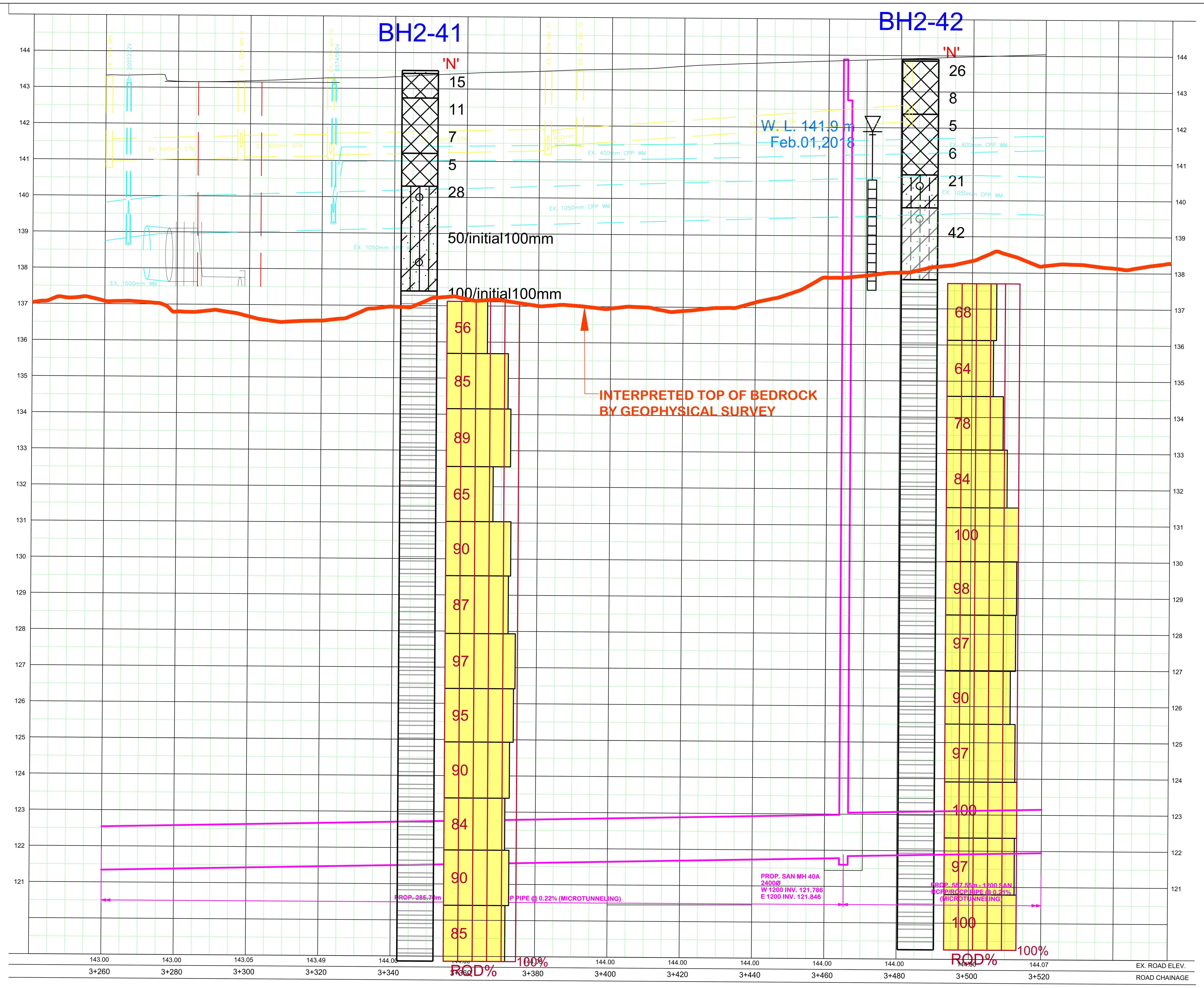
GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 4+500 TO STA. 4+740

CAD Area	Area	Project No. 171-08406-04
Checked by LC	Drawn by ZMO	
Date March 5, 2020	Sheet of	Drawing No. D5

D:\WSP\171-08406-04 Cawthra\D6-171-08406-04-PP44-Mar5-2020.dwg



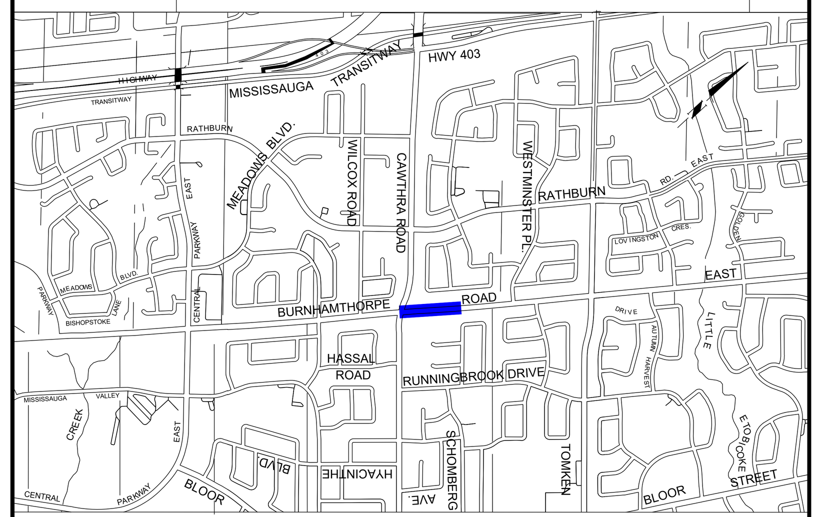


SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
MAY 16, 2017	ISSUED FOR 30% REVIEW	F.S.
JUNE 25, 2018	ISSUED FOR 50% REVIEW	JSD
PENDING	ISSUED FOR 90% REVIEW	F.S.



KEY PLAN (N.T.S.)

LEGEND

- Asphalt
- Topsoil
- Fill
- Fill gravelly sand
- Sand and Gravel
- Sand
- Silt
- Silt Till
- Silty Sand
- Sandy Silt
- Silty Clay
- Clayey Silt
- Silty Sand Till
- Sandy Silt Till
- Silty Clay Till
- Clayey Silt Till
- Sand and Silt Till
- Silty Clay Till/ Shale Complex
- Clayey Silt Till/ Shale Complex
- Shale (Georgian Bay Formation)
- Borehole drilled by WSP
- Borehole with Monitoring Well drilled by WSP
- Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted

All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field

All Horizontal And Vertical Bends Are In Degrees

All Pipes Size In mm

20C Existing Water Service, Size In mm

WS20 Proposed Water Service, Size In mm

B.M. No. Elev.
Description
Location

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

Designed by

Chkd.

Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE

1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Working for you

BURNHAMTHORPE ROAD E.

BOREHOLE LOCATION PLAN AND GEOLOGICAL SECTIONS

GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 3+280 TO STA. 3+520

CAD Area		Area		Project No. 171-08406-04
Checked by	LC	Drawn by	ZMO	
Date	March 5, 2020	Sheet	of	Drawing No. D6

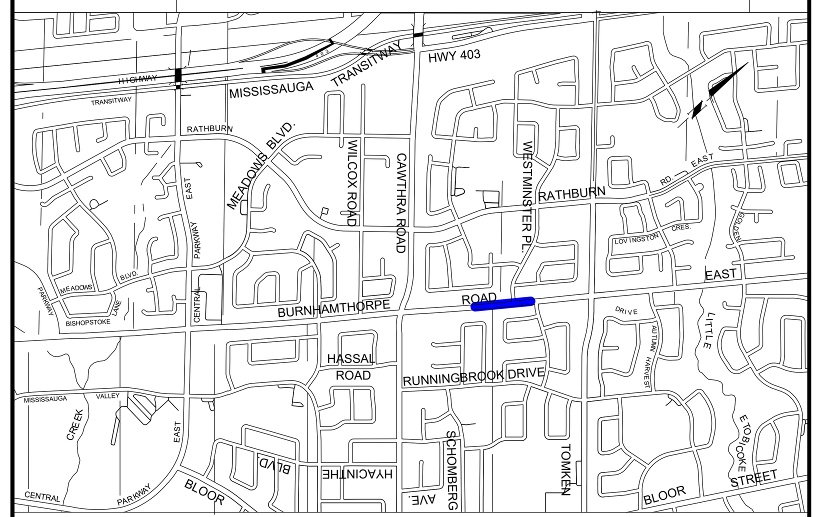
D:\WSP\171-08406-04 Cawthra\D7-171-08406-04-PP45-Mar5-2020.dwg

SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
MAY 16, 2017	ISSUED FOR 30% REVIEW	F.S.
JUNE 25, 2018	ISSUED FOR 50% REVIEW	JSD
PENDING	ISSUED FOR 90% REVIEW	F.S.



KEY PLAN (N.T.S.)

LEGEND

- Asphalt
- Topsoil
- Fill
- Fill gravely sand
- Sand and Gravel
- Sand
- Silt
- Silt Till
- Silty Sand
- Sandy Silt
- Silty Clay
- Clayey Silt
- Silty Sand Till
- Sandy Silt Till
- Silty Clay Till
- Clayey Silt Till
- Sand and Silt Till
- Silty Clay Till/ Shale Complex
- Clayey Silt Till/ Shale Complex
- Shale (Georgian Bay Formation)
- Borehole drilled by WSP
- Borehole with Monitoring Well drilled by WSP
- Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted

All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field

All Horizontal And Vertical Bends Are In Degrees

All Pipes Size In mm

20C Existing Water Service, Size In mm

WS20 Proposed Water Service, Size In mm

B.M. No. Elev.
Description
Location

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

Designed by

Chkd.

Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

1Om 0 10 20 30m HORIZONTAL SCALE

1m 0 1 2 3m VERTICAL SCALE

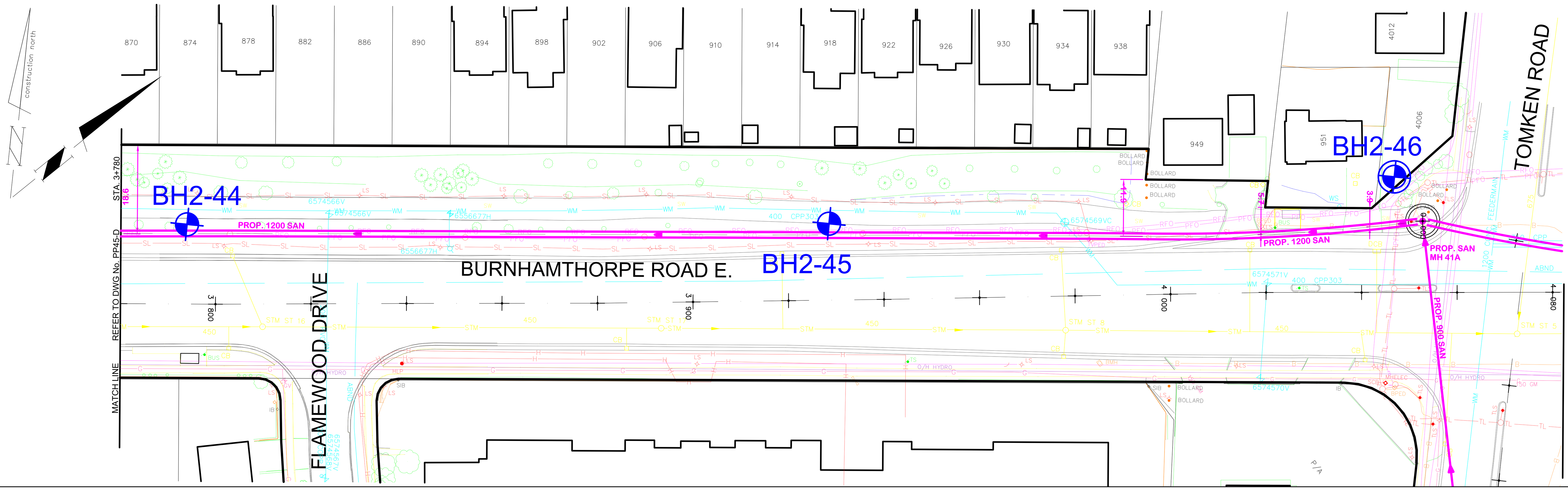
Region of Peel
Working for you

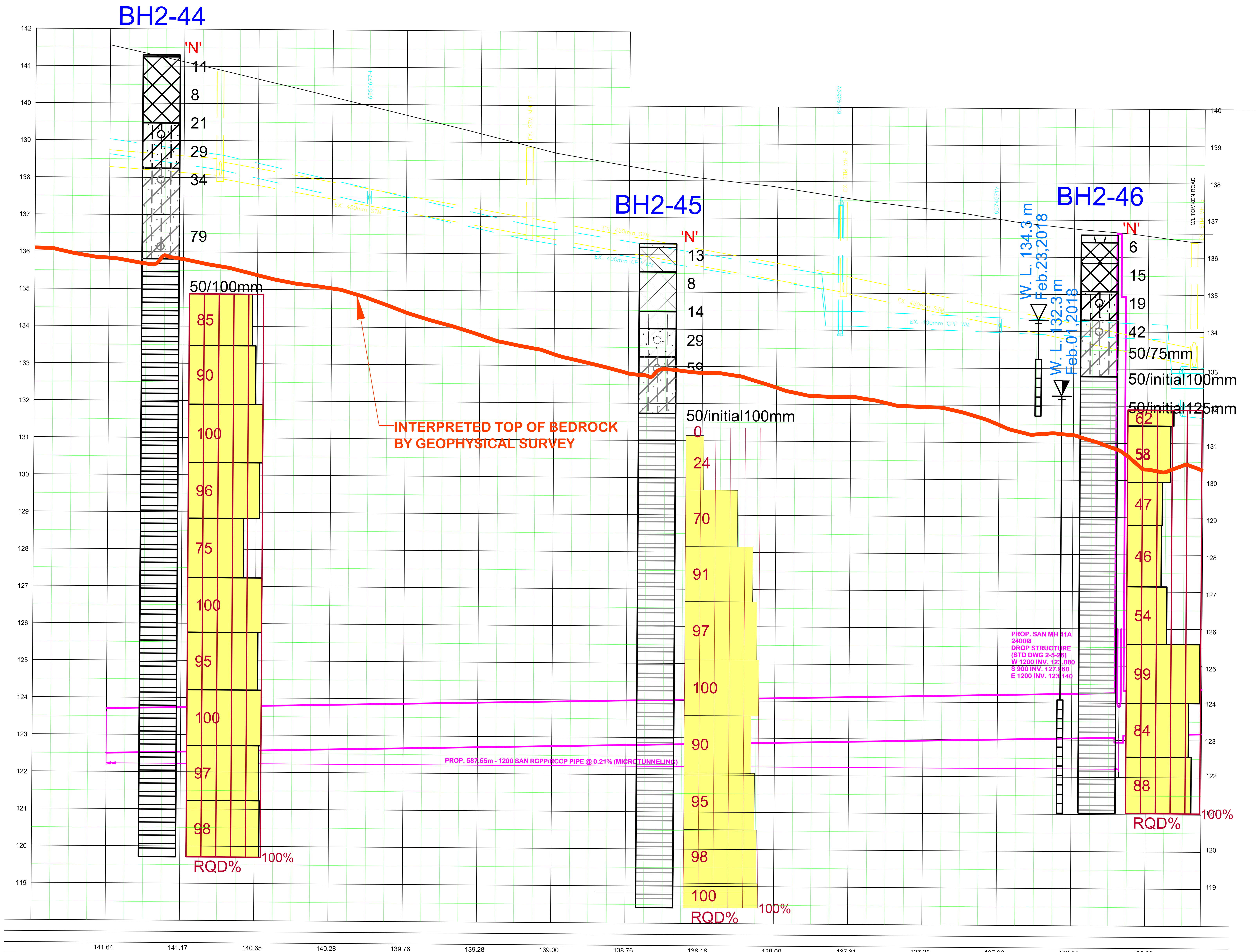
BURNHAMTHORPE ROAD E.

BOREHOLE LOCATION PLAN AND GEOLOGICAL SECTIONS
GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 3+520 TO STA. 3+780

CAD Area		Area		Project No. 171-08406-04
Checked by	LC	Drawn by	ZMO	
Date	March 5, 2020	Sheet	of	Drawing No. **D7**





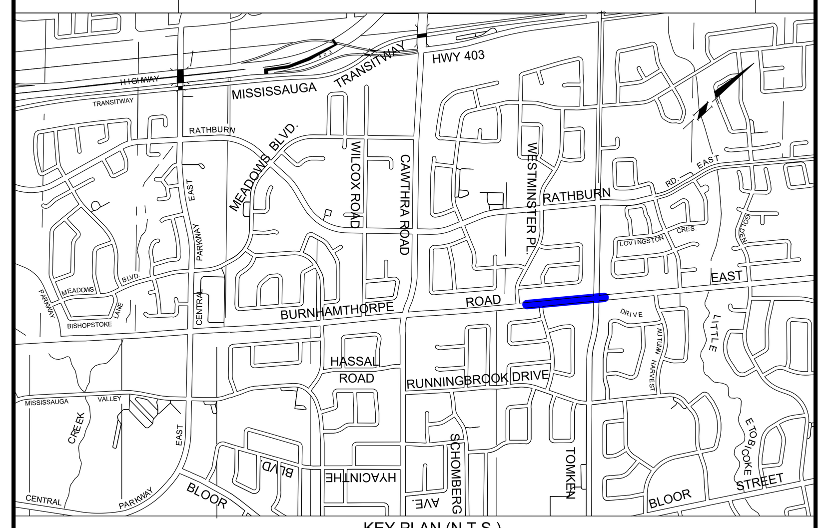
EX. ROAD ELEV.	141.64	141.17	140.65	140.28	139.76	139.28	139.00	138.76	138.18	138.00	137.81	137.28	137.00	136.54	136.00
ROAD CHAINAGE	3+780	3+800	3+820	3+840	3+860	3+880	3+900	3+920	3+940	3+960	3+980	4+000	4+020	4+040	4+060

SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
MAY 16, 2017	ISSUED FOR 30% REVIEW	F.S.
JUNE 25, 2018	ISSUED FOR 50% REVIEW	JSD
PENDING	ISSUED FOR 90% REVIEW	F.S.


KEY PLAN (N.T.S.)

LEGEND

- Asphalt
- Topsoil
- Fill
- Fill gravely sand
- Sand and Gravel
- Sand
- Silt
- Silt Till
- Silty Sand
- Sandy Silt
- Silty Clay
- Clayey Silt
- Silty Sand Till
- Sandy Silt Till
- Silty Clay Till
- Clayey Silt Till
- Sand and Silt Till
- Silty Clay Till/ Shale Complex
- Clayey Silt Till/ Shale Complex
- Shale (Georgian Bay Formation)
- Borehole drilled by WSP
- Borehole with Monitoring Well drilled by WSP
- Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted
All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field
All Horizontal And Vertical Bends Are In Degrees
All Pipes Size In mm
Existing Water Service, Size In mm
WS20 Proposed Water Service, Size In mm
B.M. No. Elev.
Description
Location
The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

Designed by
Chkd.
Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE
1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Working for you

BURNHAMTHORPE ROAD E.

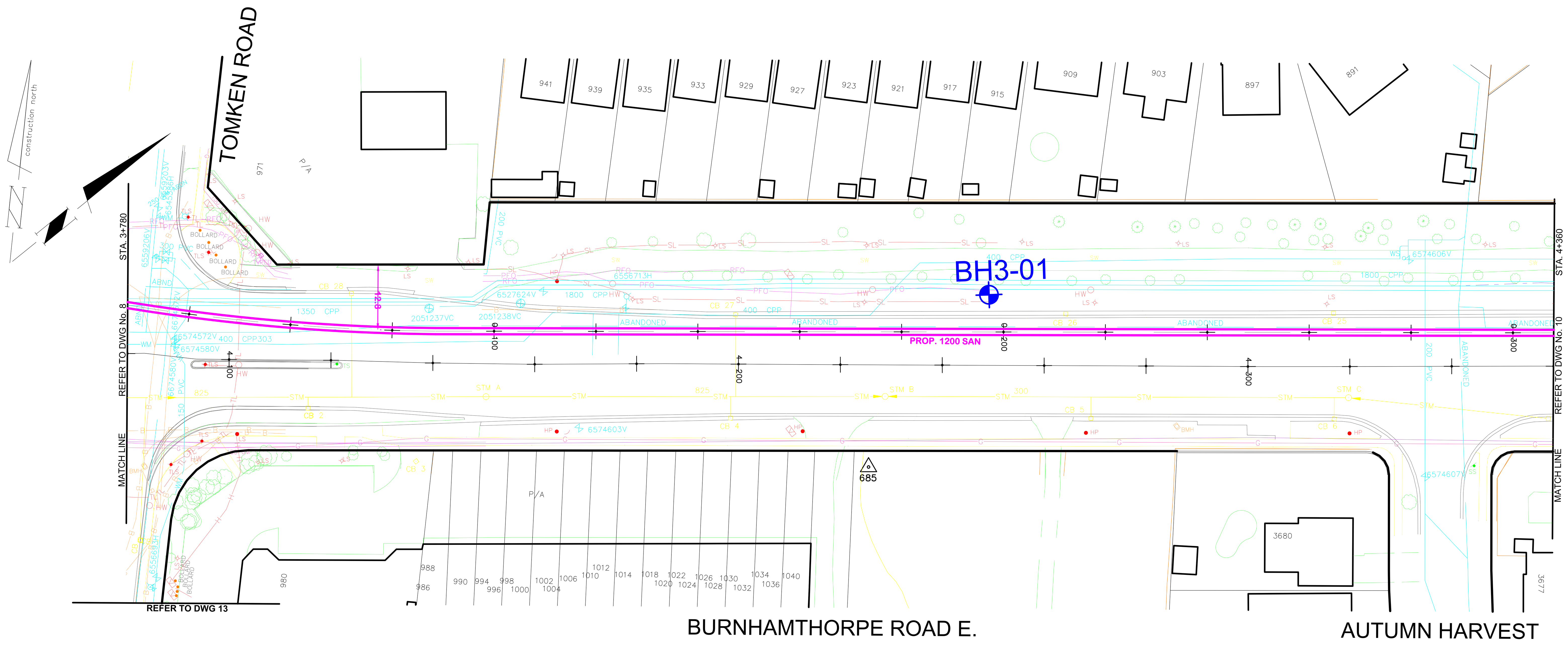
BOREHOLE LOCATION PLAN AND GEOLOGICAL SECTIONS

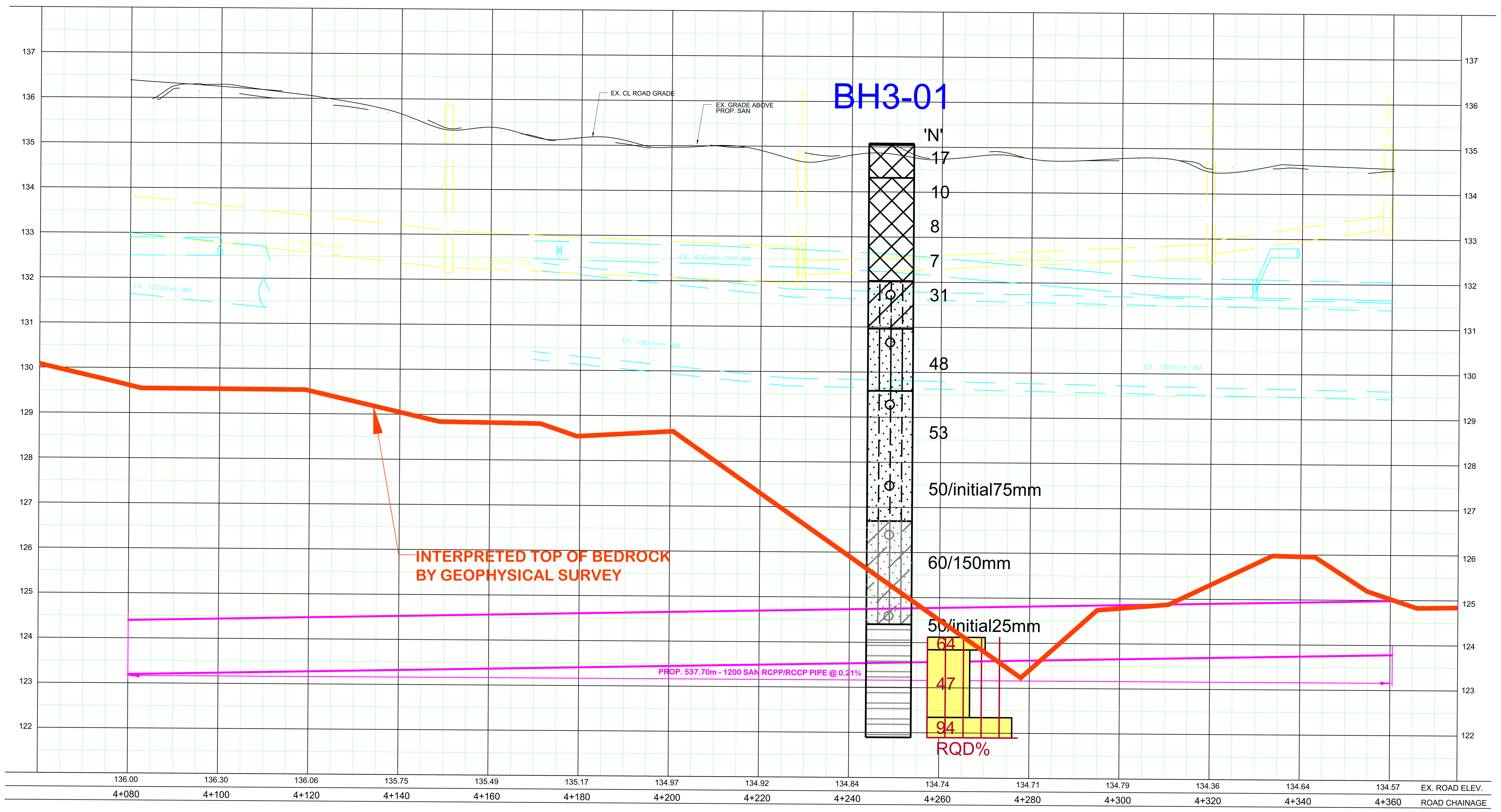
GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 3+780 TO STA. 4+060

CAD Area	Area	Project No.	171-08406-04
Checked by LC	Drawn by ZMO		
Date March 5, 2020	Sheet of	Drawing No.	D8

D:\WSP\171-08406-04 Cawthra\D8-171-08406-04-PP46-Mar5-2020.dwg



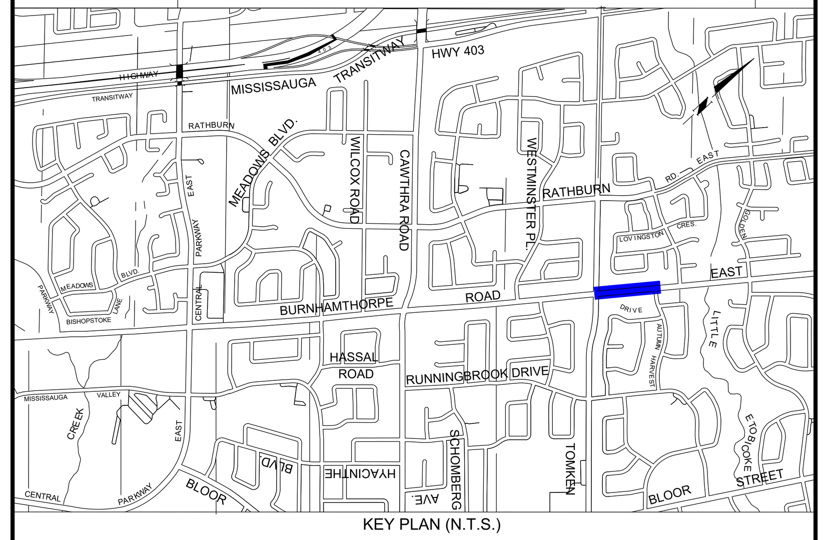


SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
PENDING	ISSUED FOR 90% REVIEW	F.S.



LEGEND

Asphalt, Topsoil, Fill, Fill gravelly sand, Sand and Gravel, Sand, Silt, Silt Till, Silty Sand, Sandy Silt, Silty Clay, Clayey Silt, Silty Sand Till, Sandy Silt Till, Silty Clay Till, Clayey Silt Till, Sand and Silt Till, Silty Clay Till/ Shale Complex, Clayey Silt Till/ Shale Complex, Shale (Georgian Bay Formation)

Borehole drilled by WSP

Borehole with Monitoring Well drilled by WSP

Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted

All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field

All Horizontal And Vertical Bends Are In Degrees

All Pipes Size In mm

Existing Water Service, Size In mm

W920 Proposed Water Service, Size In mm

B.M. No. Elev.
Description
Location

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

Designed by | Chkd. | Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE

1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Working for you

BURNHAMTHORPE ROAD E.

BOREHOLE LOCATION PLAN AND GEOLOGICAL SECTIONS

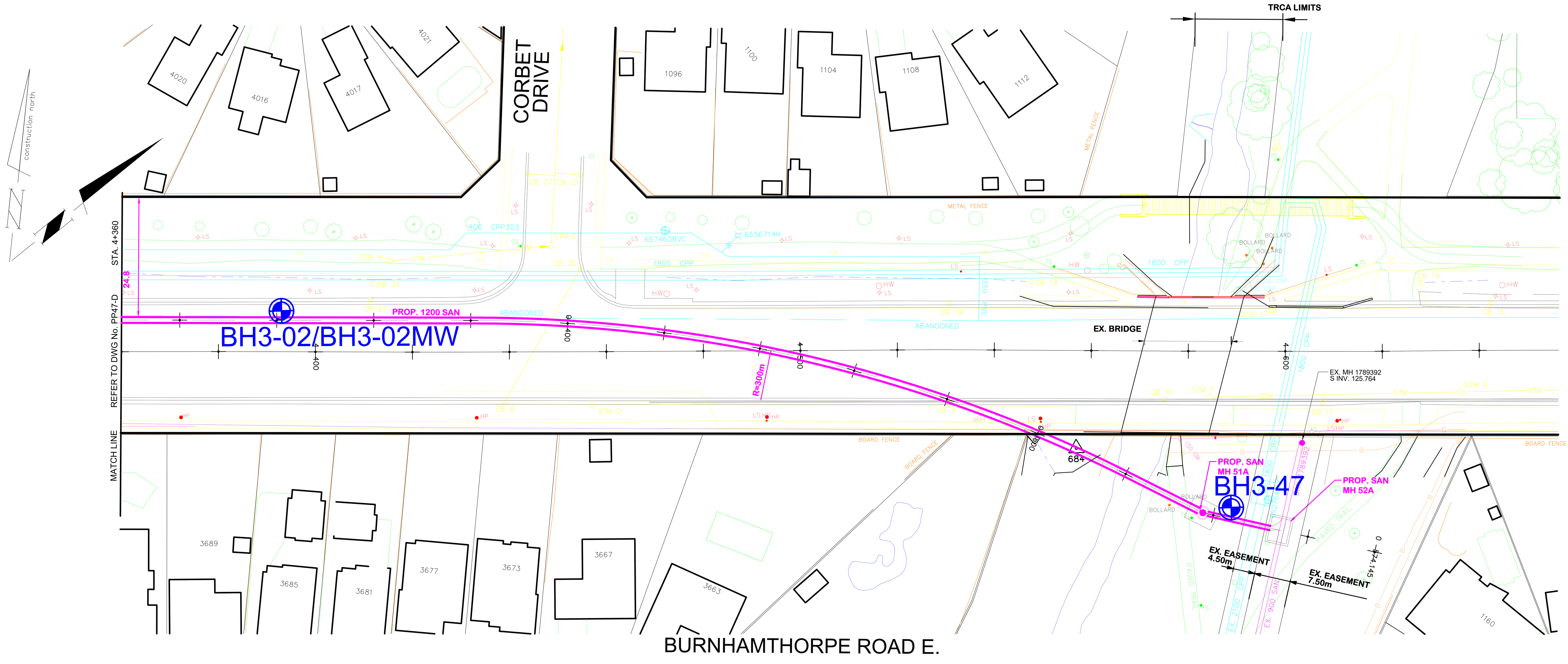
GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

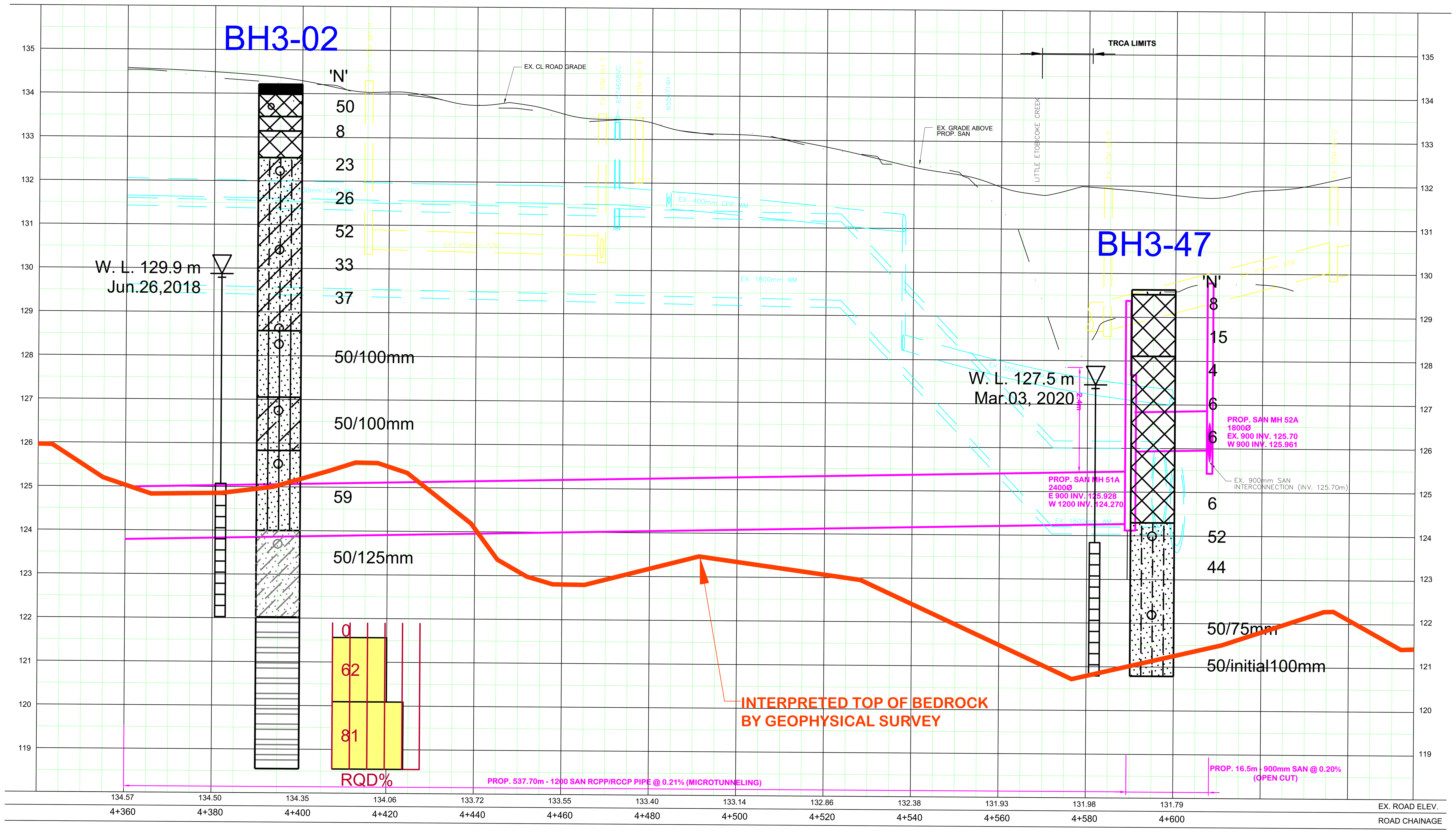
STA. 4+060 TO STA. 4+360

CAD Area	Area	Project No.	171-08406-04
Checked by LC	Drawn by ZMO		
Date March 5, 2020	Sheet of	Drawing No.	D9

D:\WSP\171-08406-04 Cawthra\D9-171-08406-04-PP47-Mar5-2020.dwg

D:\WSP\171-08406-04 Cawthra\D10-171-08406-04-PP48-Mar5-2020.dwg



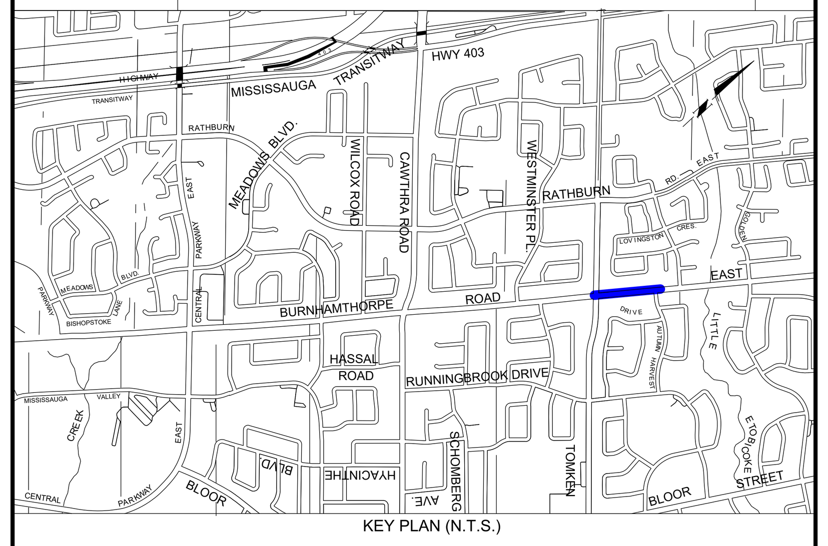


SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
PENDING	ISSUED FOR 90% REVIEW	F.S.



LEGEND

Asphalt
Topsoil
Fill
Fill gravelly sand
Sand and Gravel
Sand
Silt
Silt Till
Silty Sand
Sandy Silt
Silty Clay
Clayey Silt
Silty Sand Till
Sandy Silt Till
Silty Clay Till
Clayey Silt Till
Sand and Silt Till
Silty Clay Till/ Shale Complex
Clayey Silt Till/ Shale Complex
Shale (Georgian Bay Formation)
Borehole drilled by WSP
Borehole with Monitoring Well drilled by WSP
Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted
All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field
All Horizontal And Vertical Bends Are In Degrees
All Pipes Size In mm
20C Existing Water Service, Size In mm
WS20 Proposed Water Service, Size In mm
B.M. No. Elev.
Description
Location
The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

Designed by
Chkd.
Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE
1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Working for you

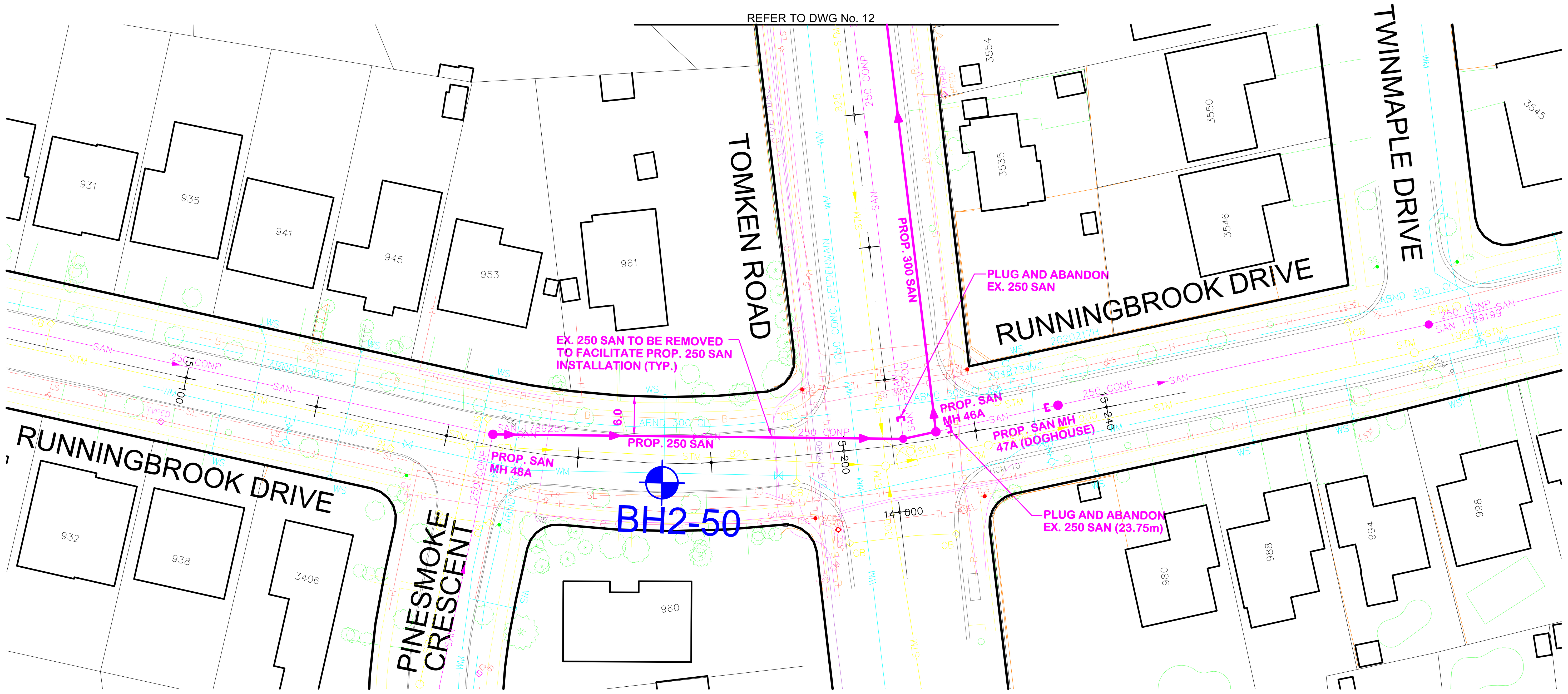
BURNHAMTHORPE ROAD E.

BOREHOLE LOCATION PLAN AND GEOLOGICAL SECTIONS

GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 4+360 TO STA. 4+600

CAD Area	Area	Project No. 171-08406-04
Checked by LC	Drawn by ZMO	
Date March 5, 2020	Sheet of	Drawing No. **D10**

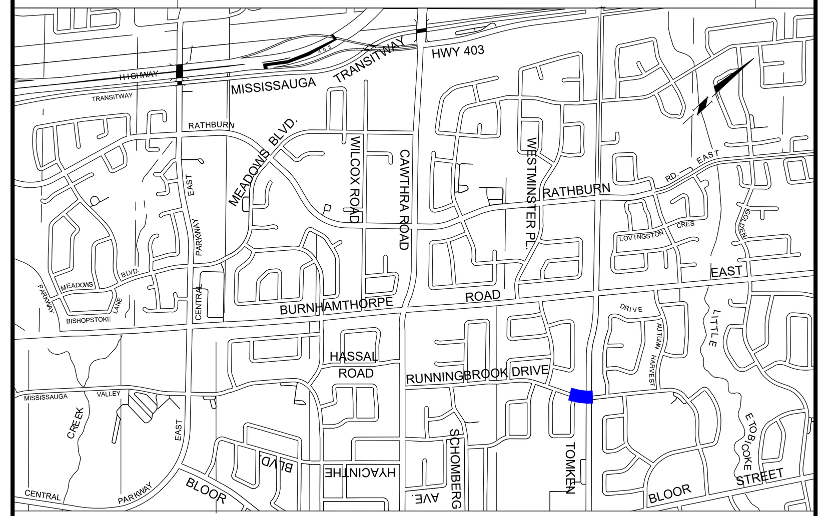


SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

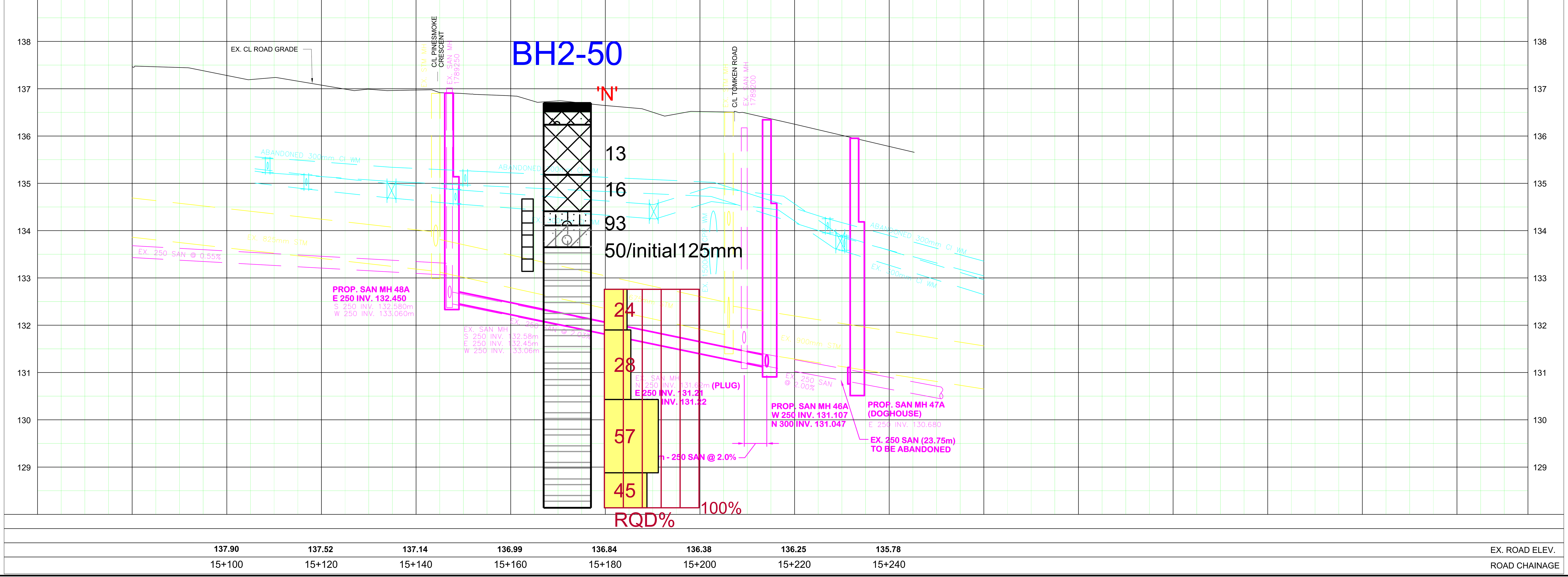
DATE	DETAILS	INIT.
MAY 16, 2017	ISSUED FOR 30% REVIEW	F.S.
JUNE 25, 2018	ISSUED FOR 50% REVIEW	JSD
PENDING	ISSUED FOR 90% REVIEW	F.S.



KEY PLAN (N.T.S.)

LEGEND

- Asphalt
- Topsoil
- Fill
- Fill gravely sand
- Sand and Gravel
- Sand
- Silt
- Silt Till
- Silty Sand
- Sandy Silt
- Silty Clay
- Clayey Silt
- Silty Sand Till
- Sandy Silt Till
- Silty Clay Till
- Clayey Silt Till
- Sand and Silt Till
- Silty Clay Till/ Shale Complex
- Clayey Silt Till/ Shale Complex
- Shale (Georgian Bay Formation)
- Borehole drilled by WSP
- Borehole with Monitoring Well drilled by WSP
- Ground Water Level



General Notes

All Driveways Are ASPHALT Unless Otherwise Noted

All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field

All Horizontal And Vertical Bends Are In Degrees

All Pipes Size In mm

20C Existing Water Service, Size In mm

WS20 Proposed Water Service, Size In mm

B.M. No. Elev.
Description
Location

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

Designed by
Chkd.
Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE

1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Working for you

RUNNINGBROOK DRIVE

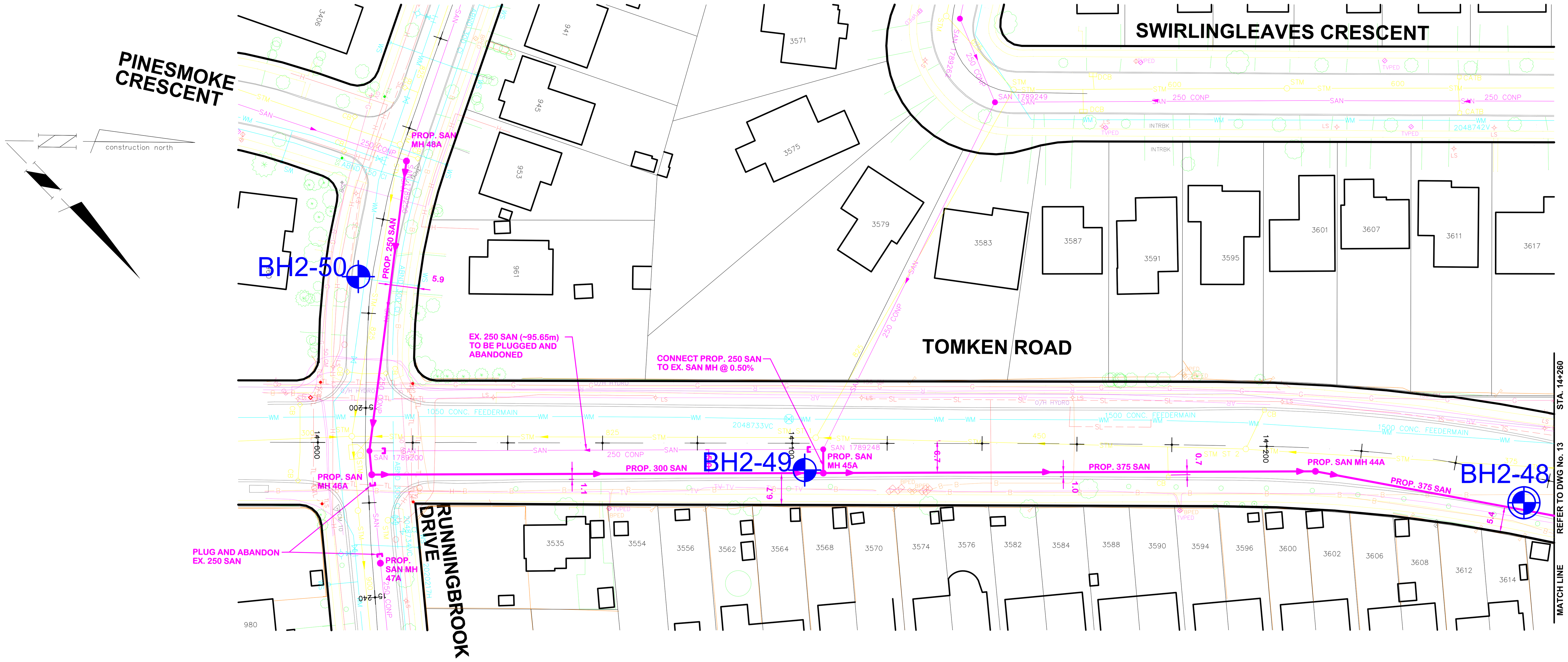
BOREHOLE LOCATION PLAN AND GEOLOGICAL SECTIONS

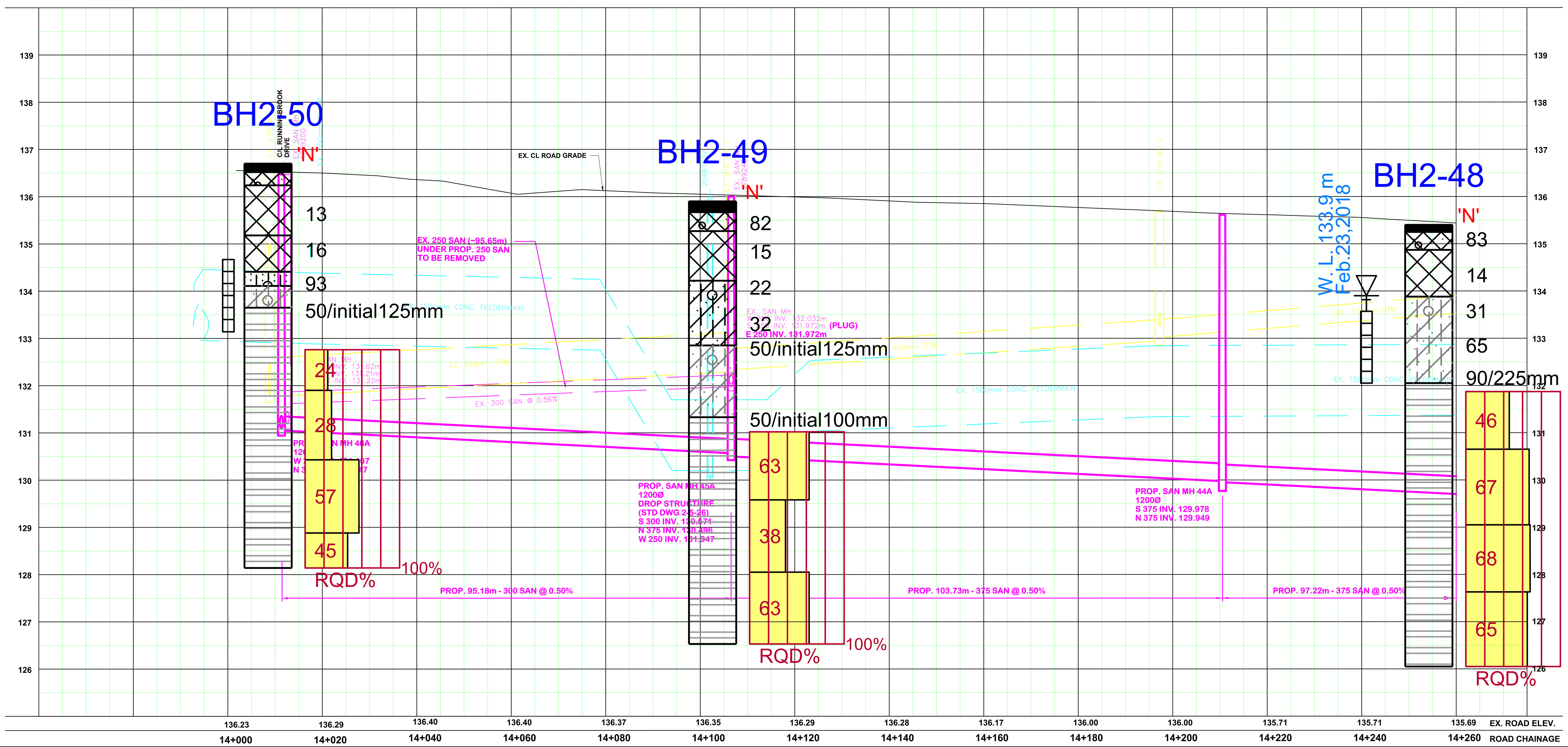
GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 15+100 TO STA. 15+240

CAD Area	Area	Project No. 171-08406-04
Checked by LC	Drawn by ZMO	
Date March 5, 2020	Sheet of	Drawing No. D11

D:\WSP\171-08406-04 Cawthra\D11-171-08406-04-PP49-Mar5-2020.dwg



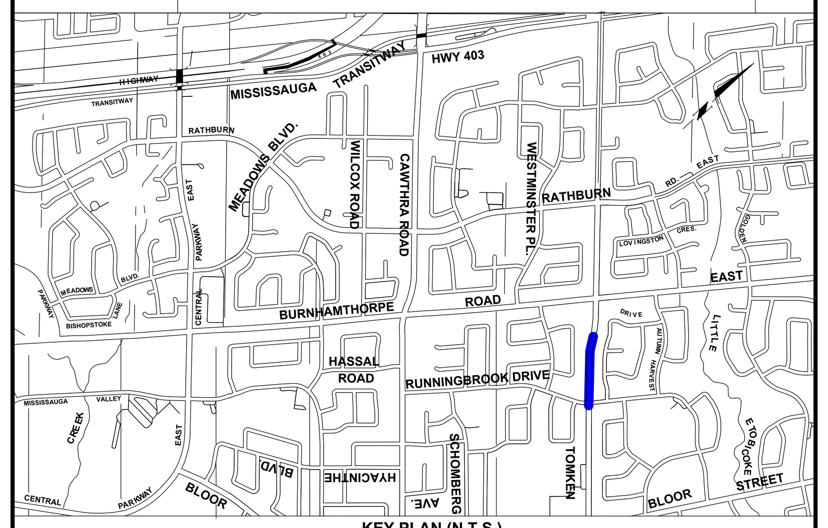


SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
MAY 16, 2017	ISSUED FOR 30% REVIEW	F.S.
JUNE 25, 2018	ISSUED FOR 50% REVIEW	JSD
PENDING	ISSUED FOR 90% REVIEW	F.S.



KEY PLAN (N.T.S.)

LEGEND

- Asphalt
- Topsoil
- Fill
- Fill gravelly sand
- Sand and Gravel
- Sand
- Silt
- Silt Till
- Silty Sand
- Sandy Silt
- Silty Clay
- Clayey Silt
- Silty Sand Till
- Sandy Silt Till
- Silty Clay Till
- Clayey Silt Till
- Sand and Silt Till
- Silty Clay Till/ Shale Complex
- Clayey Silt Till/ Shale Complex
- Shale (Georgian Bay Formation)
- Borehole drilled by WSP
- Borehole with Monitoring Well drilled by WSP
- Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted

All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field

All Horizontal And Vertical Bends Are In Degrees

All Pipes Size In mm

20C Existing Water Service, Size In mm

WS20 Proposed Water Service, Size In mm

B.M. No. Elev.
Description
Location

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

2 International Boulevard, Suite 201
Toronto, Ontario | M9W 1A2 | Canada
Tel.: 416-798-0065 | Fax: 416-798-0518 | www.wsp.com

Designed by Chkd.

Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE

1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Working for you

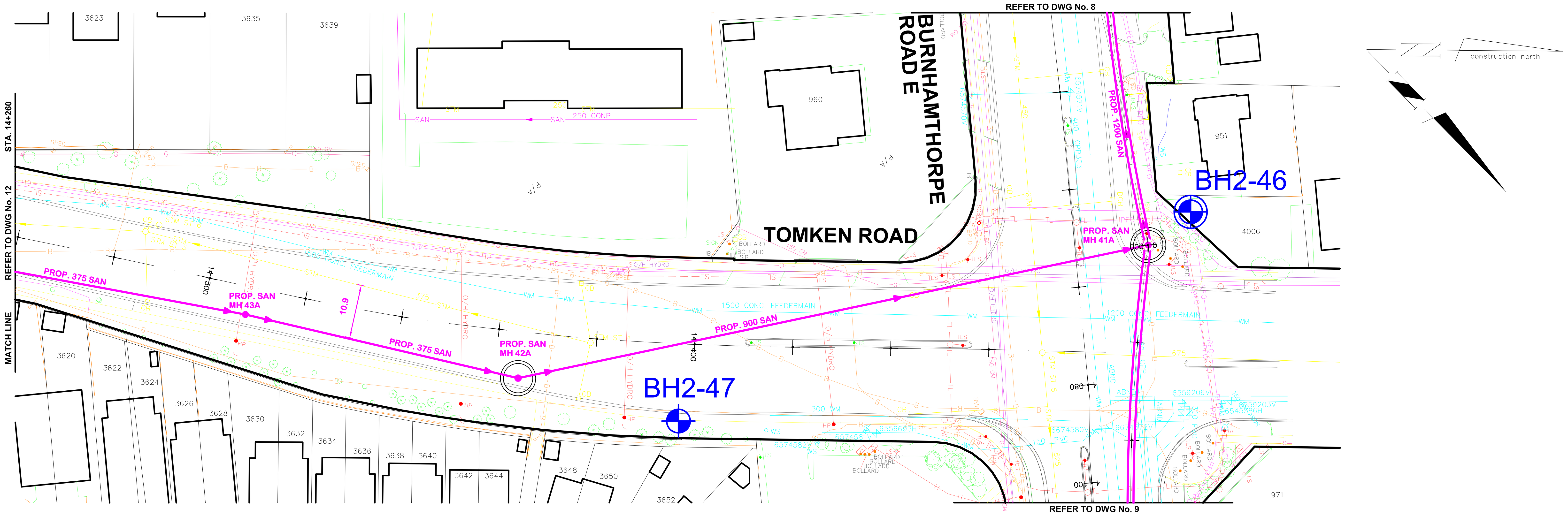
TOMKEN ROAD

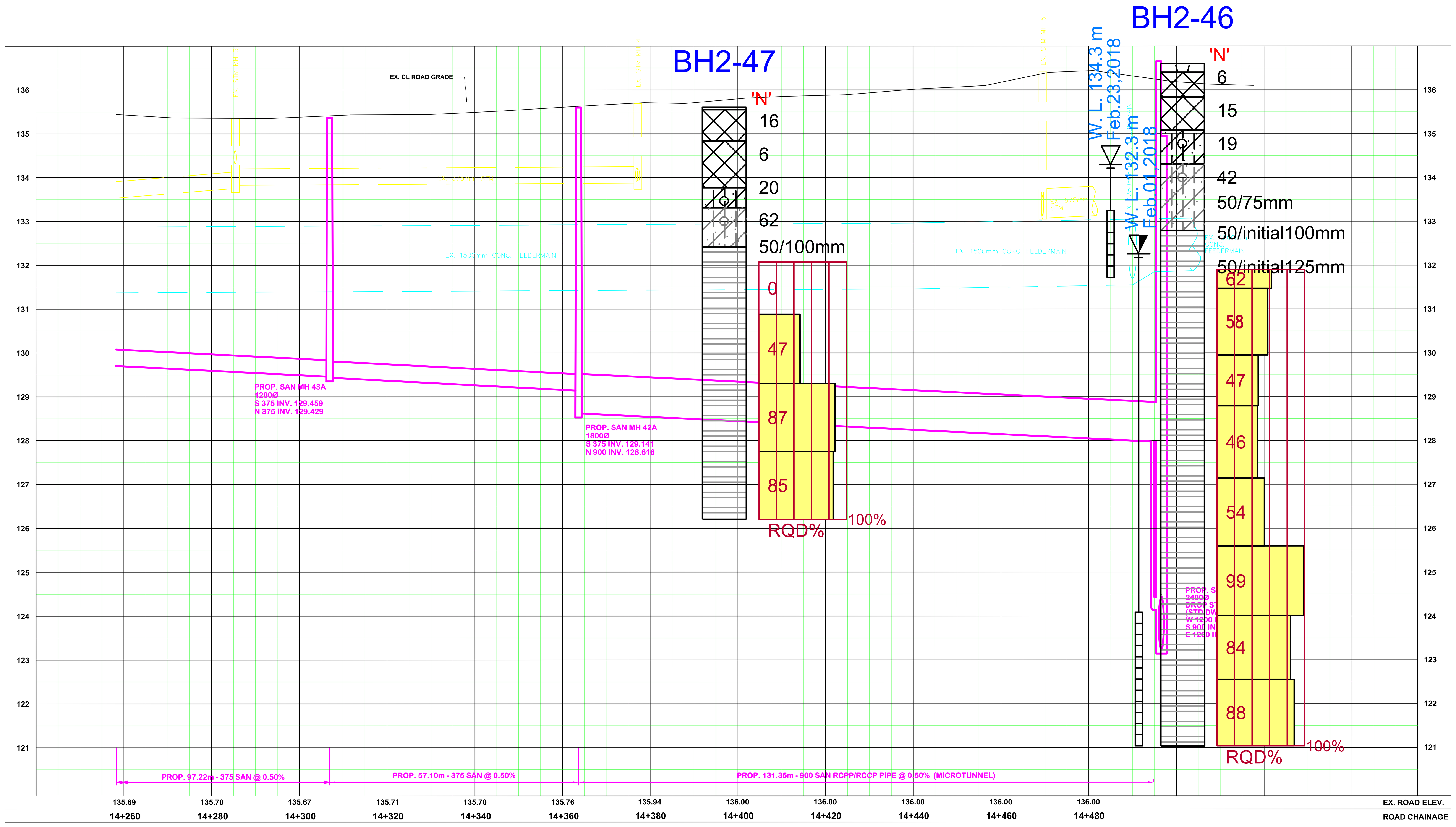
BOREHOLE LOCATION PLAN AND GEOLOGICAL SECTIONS

GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 14+000 TO STA. 14+260

CAD Area	Area	Project No. 171-08406-04
Checked by LC	Drawn by ZMO	
Date March 5, 2020	Sheet of	Drawing No. D12





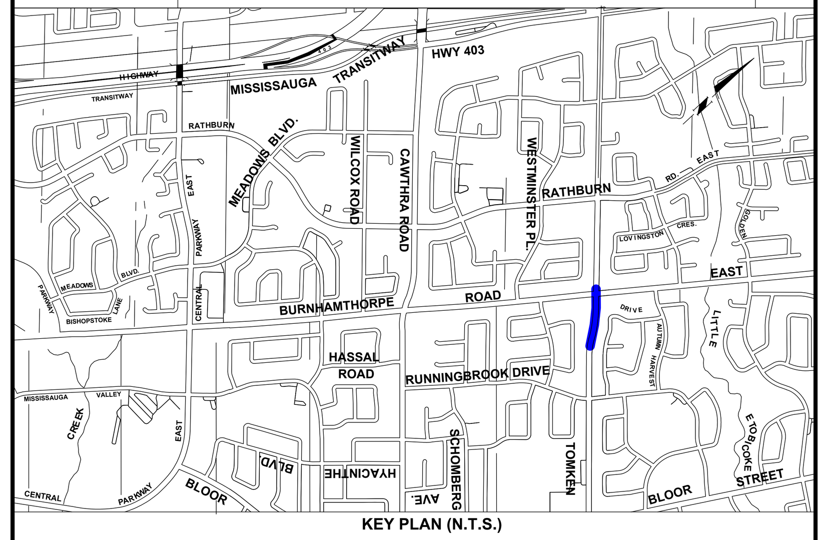
D:\WSP\171-08406-04 Cawthra\D13-171-08406-04-PP51-Mar5-2020.dwg

SERVICE DATA

SERVICE	DATE	INIT.	SERVICE	DATE	INIT.
SAN SEWERS	MARCH 2019	F.S.	GAS MAINS	NOV., 2018	F.S.
STORM SEWERS	MARCH 2019	F.S.	BELL U/G CABLE	JAN., 2019	F.S.
WATERMAINS	MARCH 2019	F.S.	HYDRO U/G CABLE	NOV., 2018	F.S.
TRANSIT	-	-	HYDRO ONE	NOV., 2018	F.S.
PARKS & REC.	FEB., 2019	F.S.	CTV	OCT., 2018	F.S.
ONT. CLEAN WATER	-	-	COMMUNIC. CABLES	NOV., 2018	F.S.

REVISIONS

DATE	DETAILS	INIT.
MAY 16, 2017	ISSUED FOR 30% REVIEW	F.S.
JUNE 25, 2018	ISSUED FOR 50% REVIEW	JSD
PENDING	ISSUED FOR 90% REVIEW	F.S.



LEGEND

Asphalt
Topsoil
Fill
Fill gravelly sand
Sand and Gravel
Sand
Silt
Silt Till
Silty Sand
Sandy Silt
Silty Clay
Clayey Silt
Silty Sand Till
Sandy Silt Till
Silty Clay Till
Clayey Silt Till
Sand and Silt Till
Silty Clay Till/ Shale Complex
Clayey Silt Till/ Shale Complex
Shale (Georgian Bay Formation)

Borehole drilled by WSP

Borehole with Monitoring Well drilled by WSP

Ground Water Level

General Notes

All Driveways Are ASPHALT Unless Otherwise Noted

All Water And Sanitary Service Locations Are Approximate And Must Be Located Accurately In The Field

All Horizontal And Vertical Bends Are In Degrees

All Pipes Size In mm

20C Existing Water Service, Size In mm

WS20 Proposed Water Service, Size In mm

B.M. No. Elev.
Description
Location

The Contractor Is Responsible For Locating And Protecting All Existing Utilities Prior To And During Construction. Location Of Existing Utilities Approximate Only, To Be Verified In Field By Contractor.

wsp
2 International Boulevard, Suite 201
Toronto, Ontario | M9W 1A2 | Canada
Tel.: 416-798-0065 | Fax: 416-798-0518 | www.wsp.com

Designed by

Chkd.

Approved by

NOTICE TO CONTRACTOR

48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING

THE REGIONAL MUNICIPALITY OF PEEL
CITY OF MISSISSAUGA WORKS DEPT.
CITY OF BRAMPTON WORKS DEPT.
TOWN OF CALEDON WORKS DEPT.
BELL CANADA
ENBRIDGE INCORPORATED-GAS DISTRIBUTION
ONTARIO MINISTRY OF TRANSPORTATION
ONTARIO CLEAN WATER AGENCY
HYDRO ONE NETWORKS
ENERSOURCE, HYDRO MISSISSAUGA
HYDRO ONE BRAMPTON

CABLE TELEVISION/FIBREOPTIC PROVIDERS:
BELL CANADA
ENERSOURCE TELECOM
HYDRO ONE TELECOM
ROGERS CABLE
ALLSTREAM
PSN (PUBLIC SECTOR NETWORK)
FUTUREWAY (FCI BROADBAND)

10m 0 10 20 30m HORIZONTAL SCALE

1m 0 1 2 3m VERTICAL SCALE

Region of Peel
Working for you

TOMKEN ROAD

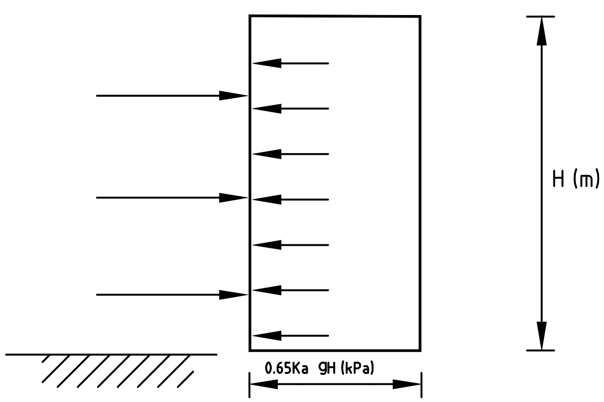
BOREHOLE LOCATION PLAN AND GEOLOGICAL SECTIONS

GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD)
CAWTHRA ROAD SANITARY DETAILED DESIGN
PHASE 3, MISSISSAUGA, ON

STA. 14+260 TO STA. 14+480

CAD Area	Area		Project No. 171-08406-04
Checked by LC	Drawn by ZMO		
Date March 5, 2020	Sheet of		Drawing No. D13

T:\Geotech\171 WSP Projects\171-08406-01 Cawthra Road Sanitary Sewer_ROP (Phase 2 & 3)\Drawings\GMD\D16-EARTH PRESSURE DISTRIBUTION ON BRACED EXCAVATIONS-Mar15-2018.dwg

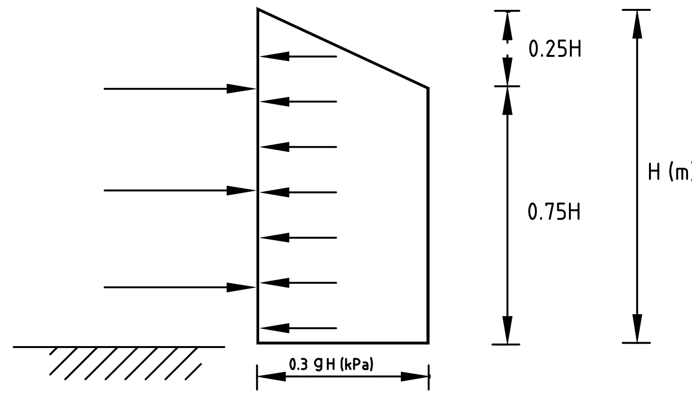


g = unit weight of soil = 21.0 kN/m^3

g' = submerged unit weight of soil (i.e. below ground water level)= 11.2 kN/m^3

Ka = 0.3

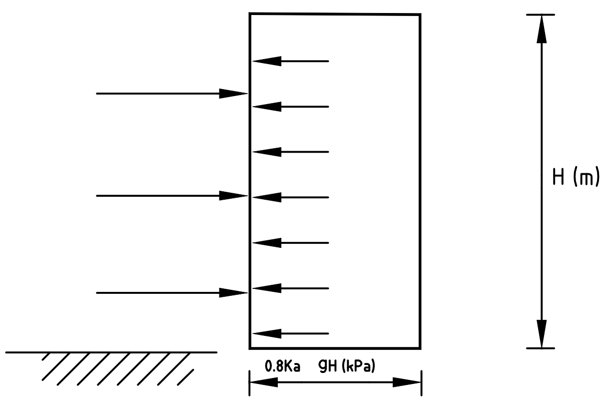
IN COMPACT TO VERY DENSE NON-COHESIVE SOILS (SANDS AND SILTS)



g = unit weight of soil = 21.5 kN/m^3

g' = submerged unit weight of soil (i.e. below ground water level)= 11.7 kN/m^3

IN COHESIVE CLAYS OR CLAYEY SOILS

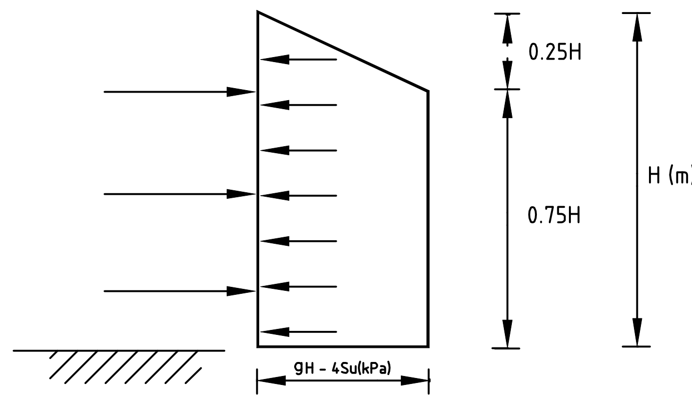


g = unit weight of soil = 19.0 kN/m^3

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

Ka = 0.36

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



g = unit weight of soil = 19.0 kN/m^3

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

Su = 10 KPa

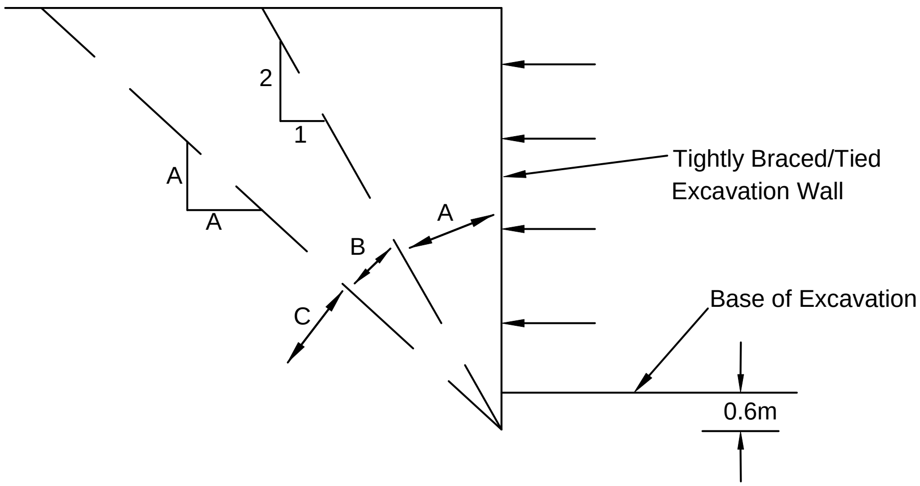
IN VERY SOFT TO FIRM COHESIVE CLAYS OR CLAYEY SOILS

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

wsp
51 CONSTELLATION COURT
TORONTO, ONTARIO CANADA M9W 1K4
TEL.: 416-798-0065 | FAX: 416-798-0518 | WWW.WSP.COM

CLIENT: REGION OF PEEL		PROJECT NO: 171-08406-04	DRAWING NO: D14
TITLE: EARTH PRESSURE DISTRIBUTION ON BRACED EXCAVATIONS		DRAWN BY: ZMO	CHECKED BY: LC
PROJECT: GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD) CAWTHRA ROAD SANITARY AND WATERMAIN DETAILED DESIGN PHASE 2, MISSISSAUGA, ON		DATE: MAR 15, 2018	SCALE: N.T.S
		ORIGINAL SIZE: LETTER	REV. #: N/A

Existing foundations located within Zone A normally require underpinning, especially for heavy structures. For some foundations in Zone A, it may be possible to eliminate underpinning and control foundation movement by tightly braced excavation walls, such as caisson walls.



Zone A Foundations located within this zone normally require underpinning. Horizontal and vertical pressures on the excavation wall of non underpinned foundations must be considered

Zone B Foundations located within this zone normally do not require underpinning. Horizontal and vertical pressures on the excavation wall of non underpinned foundations must be considered

Zone C Underpinning to structures is normally founded in this zone. Lateral pressure from underpinning is not normally considered

(Reference: Figure 26.27 from Canadian Foundation Engineering Manual, 4th Edition)

51 CONSTELLATION COURT
TORONTO, ONTARIO CANADA M9W 1K4
TEL.: 416-798-0065 | FAX: 416-798-0518 | WWW.WSP.COM

CLIENT: REGION OF PEEL		PROJECT NO: 171-08406-04	DRAWING NO: D15
TITLE: GUIDELINES FOR UNDERPINNING IN SOIL AND EXCAVATION SUPPORT		DRAWN BY: ZMO	CHECKED BY: LC
PROJECT: GEOTECHNICAL MEMORANDUM FOR DESIGN (GMD) CAWTHRA ROAD SANITARY AND WATERMAIN DETAILED DESIGN PHASE 2, MISSISSAUGA, ON		DATE: MAR 15, 2018	SCALE: N.T.S
		ORIGINAL SIZE: LETTER	REV. #: N/A

APPENDIX

A

SUMMARY OF THE PROPOSED ALIGNMENTS AND PROJECT DETAILS

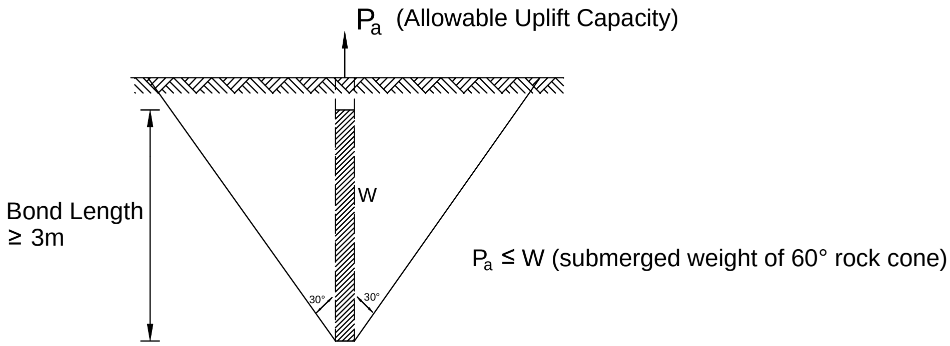
Table A1 Summary of Proposed Alignments and Project Details

ALIGNMENT	LOCATION	CHAINAGE	TYPE/SIZE (mm)	LENGTH (m)	INVERT ELEVATION masl	CONSTRUCTION METHOD
BURNHAMTHORPE ROAD E	WILCOX ROAD TO LITTLE ETOBICOKE CREEK	3+035 TO 3+093	SANITARY/375	54.4	138.75 – 138.48	OPEN CUT
		3+093 TO 3+180	SANITARY/1500	87.3	121.07 – 120.93	TRENCHLESS
		3+180 TO 4+560	SANITARY/1200	1411.0	121.17 – 124.27	TRENCHLESS
WILCOX ROAD	BURNHAMTHORPE ROAD TO KOZEL COURT	4+020 TO 4+395	SANITARY/375	375.3	138.80 – 141.81	OPEN CUT
		4+395 TO 4+715	SANITARY/300	320.8	141.88 – 143.11	OPEN CUT
TOMKEN ROAD	RUNNINGBROOK DRIVE TO BURNHAMTHORPE ROAD EAST	14+011 TO 14+107	SANITARY/300	95.2	131.05 – 130.57	OPEN CUT
		14+107 TO 14+364	SANITARY/375	258.1	130.50 – 129.14	OPEN CUT
		14+364 TO 14+495	SANITARY/900	131.4	128.62 – 127.96	TRENCHLESS
RUNNINGBROOK DRIVE	PINESMOKE CRESCENT TO TOMKEN ROAD	15+145 TO 15+210	SANITARY/250	61.6	132.45 – 131.22	OPEN CUT

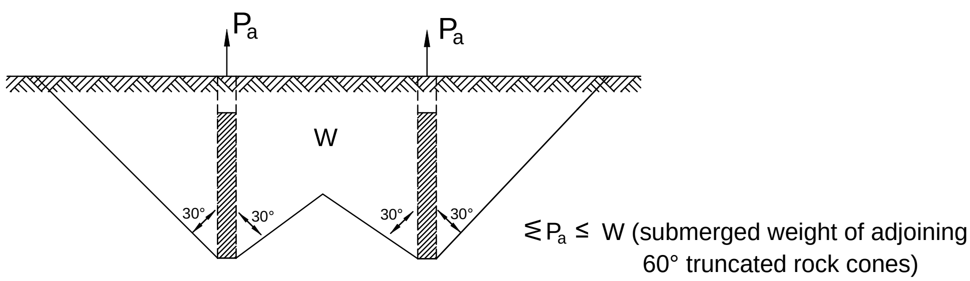
APPENDIX

B

STANDARD DRAWING FOR ROCK ANCHOR



(a) Single Rock Anchor



(a) Group Rock Anchors

Notes

1. The top 500mm of the surface rock should be neglected in calculating the required depth of embedment and the anchor bond length should not be less than 3m.
2. Below the groundwater table, the submerged weight of rock must be used.
3. The actual capacity of the anchors should be established by at least two (2) full scale pull-out tests ("performance test") in accordance with Post-Tensioning Institue (PTI) guidelines but taken to 200% of working load. In the field, each installed anchor must be proof loaded to 1.33 times the design working load for the anchor, in accordance with PTI guidelines.
4. The ground anchors should be double-corrosion protected (i.e. PTI Class I).
5. Using the above figure a factor of safety of unity (1.0) can be utilized.
6. Consult with geotechnical engineer if rockmass has defined discontinuity (i.e. fracture) sets.
7. Two stage grouting is required for fracture rockmasses (i.e. post-grouting).

<u>ALLOWABLE (SLS) UPLIFT CAPACITY FOR PRESSURE - GROUTED ROCK ANCHOR</u>

51 CONSTELLATION COURT
TORONTO, ONTARIO CANADA M9W 1K4
TEL.: 416-798-0065 | FAX: 416-798-0518 | WWW.WSP.COM

APPENDIX

C

TUNNELMAN'S GROUND CLASSIFICATION AND PROBABLE WORKING CONDITIONS

Tunnelman's Ground Classification and Probable Working Conditions

Soil Classification	Representative Soil Samples	Tunnel Working Conditions
Hard	Very hard calcareous clay; Cemented sand and gravel	Tunnel heading may be advanced without roof support.
Firm	Loess above GWT; Various calcareous clay with low plasticity	Tunnel heading may be advanced without roof support. Permanent support can be constructed before the ground will start to move.
Slow Ravelling and Fast Ravelling	Fast ravelling occurs in residual soils or in sand with clay binder below the GWT. Above the GWT, the same soils may be Slow Ravelling or even Firm.	Chunks of material may drop out of the crown or the sides some time after the ground has been exposed. In Fast Ravelling ground, the process starts within a few minutes; otherwise, it is classed as Slow Ravelling.
Squeezing	Soft or medium-soft clay	Ground slowly advances into tunnel without fracturing and without perceptible increase of water content in ground surrounding the tunnel.
Swelling	Heavily pre-compressed clays with a plasticity index greater than 30. Sedimentary formations containing layers of anhydrite.	Like squeezing ground, moves slowly into tunnel, but the movement is associated with a very considerable volume increase in the ground surrounding the tunnel.
Cohesive Running and Running	Occurs in clean, fine moist sand Occurs in clean, coarse or medium sand above the GWT	Removal of the lateral support of any surface rising at an angle of more than about 34° to the horizontal is followed by a 'run', whereby the material flows like granulated sugar until the slope angle is approx. 34°. If the 'run' is preceded by a brief period of ravelling, the ground is called Cohesive Running.
Very Soft Squeezing	Clays and silts with high plasticity indices	Ground advances rapidly into the tunnel in a plastic flow
Flowing	Any ground below the GWT that has an effective grain size in excess of about 0.00mm	Flowing ground moves like a viscous liquid. It can invade the tunnel not only through the roof and the sides, but also through the invert. If the flow is not stopped, it will eventually completely fill the tunnel.
Bouldery	Boulder glacial till; riprap fill; some land slide deposits, some residual soils. The matrix between boulders may be gravel, sand, silt, clay and in any combination.	Problems incurred in advancing shield or in forepoling; blasting or hand mining ahead of machine may become necessary.

APPENDIX

D

STATEMENT OF LIMITATIONS

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to WSP Canada Inc. at the time of preparation. Unless otherwise agreed in writing by WSP Canada Inc., it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the test hole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the test holes may differ from those encountered at the test hole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of test holes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. WSP Canada Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We accept no responsibility for any decisions made or actions taken as a result of this report unless we are specifically advised of and participate in such action, in which case our responsibility will be as agreed to at that time.