PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

PREPARED FOR:
Edenshaw Queen Developments Limited
20-129 Lakeshore Road East
Mississauga, Ontario

ATTENTION:
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Part Queen Street Plan PC-1 (300) E of Credit River btn Ann St and Hurontario St Parts 2-4 43R-39134 Mississauga, Ontario

Grounded Engineering Inc.
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1 Introduction

Grounded Engineering Inc. (Grounded) was retained by Edenshaw Queen Developments Limited to provide preliminary geotechnical engineering design advice for the proposed development of the subject site that consists of a strip of land located north adjacent to 88 Park Street East, with the legal description of Part Queen Street Plan PC-1 (300) E of Credit River btn Ann St and Hurontario St Parts 2-4 43R-39134, Mississauga, Ontario. The subject site is located at the northeast corner of the intersection of Queen Street East and Ann Street. The proposed development will consist of the subject site as well as the rest of the property at 88 Park Street East, extending south from the subject site. The present report has been requested for the subject site only.

The proposed project includes constructing a two new high-rise towers with four underground parking levels beneath the entire site set at a lowest (P4) Finished Floor Elevation (FFE) of 69.65± m. The proposed development will be situated across the subject property and 88 Park Street East, Mississauga.

Grounded has been provided with the following reports and drawings to assist in our geotechnical scope of work:

- Site survey, prepared by R. Avis Surveying Inc. (Dec 13, 2021).
- Building Section, “30 Queen Street East, Mississauga, Ontario”; Project 21-231, Drawing A500, dated December 2021, prepared by CORE Architects Inc.

Grounded’s subsurface investigation of the site to date includes three (3) boreholes (Boreholes 101 to 103) which were advanced from January 5th to 6th, 2022.

Based on the borehole findings, preliminary geotechnical engineering advice for the proposed development is provided for foundations, seismic site classification, earth pressure design, slab on grade design, and basement drainage. Construction considerations including excavation, groundwater control, and geostructural engineering design advice are also provided.

This preliminary geotechnical engineering report is appropriate for due diligence and planning purposes only. A detailed geotechnical engineering report, potentially including additional site-specific boreholes and wells, will be required for the entire development site which includes the subject site plus the lands to the south.

2 Ground Conditions

The borehole results are detailed on the attached borehole logs. Our assessment of the relevant stratigraphic units is intended to highlight the strata as they relate to geotechnical engineering. The ground conditions reported here will vary between and beyond the borehole locations.

The stratigraphic boundary lines shown on the borehole logs are assessed from non-continuous samples supplemented by drilling observations. These stratigraphic boundary lines represent transitions between soil types and should be regarded as approximate and gradual. They are not exact points of stratigraphic change.

Elevations are measured relative to the survey by R. Avis referenced above. The horizontal coordinates are provided relative to the Universal Transverse Mercator (UTM) geographic coordinate system.

2.1 Stratigraphy

The following stratigraphy summary is based on the borehole results and the geotechnical laboratory testing.

A subsurface profile showing stratigraphy and engineering units is appended.

2.1.1 Surficial and Earth Fill

Surficial fill (pavements, aggregate, etc.) thicknesses were observed in individual borehole locations through the top of the open borehole. Thicknesses may vary between and beyond each borehole location.

All boreholes encountered an asphalt pavement structure at ground surface. Borehole 101 encountered a 100 mm thick aggregate layer underlying the asphalt pavement.

Underlying the surficial materials, all boreholes observed a layer of earth fill that extends to 1.5 m below grade (Elev. 83.3 to 83.0 m). The earth fill varies in composition but generally consists of sand and silty sand with traces of gravel and clay. It contains traces of asphalt, cinder, and brick fragments. The earth fill varies in color (grey, brown, and black). The earth fill is typically moist (occasionally wet). Due to inconsistent placement and the inherent heterogeneity of earth fill materials, the relative density of the earth fill varies.

2.1.2 Native Till

Underlying the fill materials, all the Grounded boreholes encounter an undisturbed native glacial till deposit with a matrix of cohesive clayey silts (silt and clay to clayey silt). These soils are grouped together as the "**native till unit**". This unit was encountered at 1.5 m below grade (Elev. 83.3 to 83.0 m) and extends down to 9.1 m below grade (Elev. 75.7 to 75.4 m).

The native till is generally brown, moist, and transitions to grey at around 3 m below grade. This unit contains trace gravel, and trace to some sand. Traces of shale and limestone fragments were also observed deeper in this unit above the inferred bedrock. There were also occasional silt and sand seams observed within the till.

Standard Penetration Test (SPT) results (N-Values) measured in the native till range from 13 to over 50 blows per 300 mm of penetration ("bpf"), indicating a consistency ranging from stiff to hard (on average, hard).

2.1.3 Inferred Bedrock

All the boreholes indirectly inferred the top of weathered bedrock through auger cuttings, split spoon samples, and auger grinding/resistance observations, at 9.1 m below grade (Elev. 75.7 to 75.4 m). Each of these boreholes was terminated due to auger and sampler refusal (at target investigation depth) at elevations ranging from Elev. 75.0 to 73.9 m.

2.2 Groundwater

Monitoring wells were installed in each of the boreholes, and stabilized groundwater levels were measured in each of the monitoring wells one week after the completion of drilling. The boreholes were cased by hollow stem augers on completion, and cave measurement was not practical.

The groundwater observations are shown on the Borehole Logs and are summarized as follows.

Borehole No.	Borehole depth (m)	Upon completion of drilling		Strata Screened	Water Level in Well on Jan 27, 2023 (m)	
		Depth to cave (m)	Unstabilized water level (m)		Depth	Elevation
101	10.7	N/A	Not Measured	Native Till	7.7	77.1
102	9.5	N/A	Not Measured	Native Till	7.9	76.6
103	10.9	N/A	Dry	Native Till	8.2	76.6

Groundwater levels fluctuate with time depending on the amount of precipitation and surface runoff, and may be influenced by known or unknown dewatering activities at nearby sites.

The design groundwater table for engineering purposes is at Elev. 77.1 m. The groundwater table is in the native till deposit, which has a very low permeability and will yield only minor seepage in the long-term. There is also groundwater present in the weathered bedrock. It can be expected that fractures in the weathered and sound bedrock will produce seepage below the groundwater table. There is also perched stormwater in the earth fill, infiltrating down to the groundwater table.

Grounded has prepared a hydrogeological report for this site (File No. 22-302).

3 Geotechnical Engineering Recommendations

Based on the factual data summarized above, preliminary geotechnical engineering recommendations are provided. These preliminary recommendations are for due diligence purposes only. They must be supplemented and confirmed by additional boreholes, wells, and a detailed geotechnical engineering report at the detailed design stage.

This report assumes that the design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards, and guidelines of practice. If there are any changes to the site development features, or there is any additional information relevant to the interpretations made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Grounded should be retained to review the implications of these changes with respect to the contents of this report.

The proposed project includes constructing a two new high-rise towers with four underground parking levels beneath the entire site set at a lowest (P4) Finished Floor Elevation (FFE) of 69.65± m. The proposed development will be situated within the subject property as well as within the property at 88 Park Street East to the south.

3.1 Preliminary Foundation Design Parameters

Foundations made for the proposed P4 level will bear on sound bedrock of the Georgian Bay Formation. Conventional spread footings made to bear on sound bedrock may be designed using a maximum factored geotechnical resistance at ULS of 10 MPa. The net geotechnical reaction at SLS is 6 MPa, for an estimated total settlement of 10 mm. **A detailed investigation scope across the entire development site including coring of the bedrock is required for the above capacities to be utilized.**

Individual spread footing foundations designed to these capacities must be at least 1000 mm wide and must be embedded a minimum of 600 mm below FFE. These minimum requirements apply in conjunction with the above recommended geotechnical resistance regardless of loading considerations. The geotechnical reaction at SLS refers to a settlement which for practical purposes is linear and non-recoverable. Differential settlement is related to column spacing, column loads, and footing sizes.

Footings stepped from one level to another should be at a slope not exceeding 1 vertical to 1 horizontal for the above bearing pressures to be applicable. There must be a minimum of 300 mm between the edge of any footing and the top of a sloped 2V:1H sound rock cut down to another footing.

The lowest levels of unheated underground parking structures two or more levels deep are, although unheated, still warmer than typical outdoor winter temperatures in the Greater Toronto Area. Interior foundations (or pile caps) with 900 mm of frost cover perform adequately, as do perimeter foundations with 600 mm of frost cover. Where foundations are next to ventilation shafts or are exposed to typical outdoor temperatures, 1.2 m of earth cover (or equivalent insulation) is required for frost protection.

The founding subgrade must be cleaned of all unacceptable materials and approved by Grounded prior to pouring concrete for the footings. Such unacceptable materials may include disturbed or caved soils, ponded water, or similar as indicated by Grounded during founding subgrade inspection. During the winter, adequate temporary frost protection for the footing bases and concrete must be provided if construction proceeds during freezing weather conditions. The bedrock surface can weather and deteriorate on exposure to the atmosphere or surface water; hence, foundation bases which remain open for an extended period of time should be protected by a skim coat of lean concrete.

3.2 Earthquake Design Parameters

The Ontario Building Code (2012) stipulates the methodology for earthquake design analysis, as set out in Subsection 4.1.8.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration, and the site classification.

The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4A of the Ontario Building Code (2012). The classification is based on the determination of the average shear wave velocity in the 30 metres of the site stratigraphy below spread footing/grade beam elevation, where shear wave velocity (v_s) measurements have been taken. Alternatively, the classification is estimated from the rational analysis of undrained shear strength (s_u) or penetration resistance (N-values) according to the OBC and National Building Code of Canada.

It is assumed that the foundations for the proposed development will be resting on sound bedrock. Based on this information, the site designation for seismic analysis is **Class B**, per Table 4.1.8.4.A of the Ontario Building Code (2012). Tables 4.1.8.4.B and 4.1.8.4.C. of the same code provide the applicable acceleration- and velocity-based site coefficients.

3.3 Earth Pressure Design Parameters

At this site, the design parameters for structures subject to unbalanced earth pressures such as basement walls and retaining walls are shown in the table below.

Stratigraphic Unit	γ	φ	K_a	K_o	K_p
Compact Granular Fill Granular 'B' (OPSS.MUNI 1010)	21	32	0.31	0.47	3.25
Existing Earth Fill	19	29	0.35	0.52	2.88
Native Till	21	34	0.28	0.44	3.54
Sound Bedrock	26	28	n/a		

γ = soil bulk unit weight (kN/m^3)
φ = internal friction angle (degrees)
K_a = active earth pressure coefficient (Rankine, dimensionless)
K_o = at-rest earth pressure coefficient (Rankine, dimensionless)
K_p = passive earth pressure coefficient (Rankine, dimensionless)

These earth pressure parameters assume that grade is horizontal behind the retaining structure. If retained grade is inclined, these parameters do not apply and must be re-evaluated.

The following equation can be used to calculate the unbalanced earth pressure imposed on walls:

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

P	=	horizontal pressure (kPa) at depth h	γ	=	soil bulk unit weight (kN/m^3)
h	=	the depth at which P is calculated (m)	γ'	=	submerged soil unit weight (γ - 9.8 kN/m^3)
K	=	earth pressure coefficient	q	=	total surcharge load (kPa)
h_w	=	height of groundwater (m) above depth h			

If the wall backfill is drained such that hydrostatic pressures on the wall are effectively eliminated, this equation simplifies to:

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

The possible effects of frost on retaining earth structures must be considered. In frost-susceptible soils, pressures induced by freezing pore water are basically irresistible. Insulation typically addresses this issue. Alternatively, non-frost-susceptible backfill may be specified.

Foundation resistance to sliding is proportional to the friction between the rock subgrade and the base of the footing. The factored geotechnical resistance to friction (**R_f**) at ULS provided in the following equation:

$$R_f = \Phi N \tan\varphi$$

R_f	=	frictional resistance (kN)
Φ	=	reduction factor per Canadian Foundation Engineering Manual (CFEM) Ed. 4 (0.8)
N	=	normal load at base of footing (kN)
φ	=	internal friction angle (see table above)

3.4 Rock Swell

The earth pressure design approach for foundation walls below the top of bedrock is empirical and assumes a uniform pressure distribution below the top of bedrock elevation equal to the maximum earth pressure calculated for the lowest level of soil overtop. This approach is conventional and likely conservative, but it is practical insofar as it acknowledges the requirement of having a foundation wall of a consistent width at the lower levels.

However, this approach does not recognize the potential for pressures on the basement wall due to time-dependent rock swell that results when locked in horizontal stresses are released. For structures deeper than 2 m below the top of sound rock, rock swell must also be considered. The simplest approach to dealing with rock swell is scheduling. If there is a 120-day gap between rock excavation and construction of the permanent structure that will restrain the rock, experience on similar structures indicates the rock will de-stress and swell, and no significant stresses are imposed on the structural wall. This requirement typically only impacts the lowest basement level (or two) in bedrock, acknowledging the 120-day window.

If the construction schedule does not allow for a 120-day gap, mitigation measures will be required. For structures subjected to unbalanced rock swell pressure (i.e. lowest exterior foundation walls, sumps, elevators, other features cast directly against the rock face), rock squeeze effects can be addressed by providing a crushable layer between the rock and the concrete, such as 50 mm thick Ethafoam 220 Polyethylene Foam planks. The subject walls are typically designed for the 50% compressive strength resistance of the foam. At 50% compression, a 220 Ethafoam 220 Polyethylene Foam plank provides 124 kPa of resistance. At 10% compression (which allows for concrete placement), this material provides 50 kPa of resistance.

Deeper protrusions (sumps, elevator pits, etc.) can be over-excavated as they are not typically constrained by the property lines or adjacent footings. In this case the rock can be horizontally over-excavated by a minimum 600 mm on all sides. Precast pits and sumps are then placed and backfilled with 19 mm clear stone (OPSS.MUNI 1004). The clear stone backfill then accommodates the rock swell.

Rock squeeze effects are not relevant to foundation excavations as the earth pressures exerted on foundation elements are balanced, and concrete is strong enough to resist the swell pressure and render it null.

3.5 Slab on Grade Design Parameters

The lowest (P4) basement slab of the proposed structure will be set on sound bedrock of the Georgian Bay Formation. The bedrock at this site constitutes an adequate subgrade for support of a slab on grade. The modulus of subgrade reaction appropriate for design of the slab resting on an aggregate drainage layer overlying unweathered (sound) bedrock is 80,000 kPa/m.

If this basement structure is made as a conventional drained structure, a permanent drainage system including subfloor drains is required (see section below). In this case, the slab on grade must be provided with a drainage layer and capillary moisture break, which is achieved by forming the slab on a minimum 300 mm thick layer of 19 mm clear stone (OPSS.MUNI 1004) vibrated to a dense state.

Subfloor drains are typically installed in trenches below the capillary moisture break drainage layer per the typical detail appended. If trenches are to be avoided for whatever reason, the subfloor drainage system can be incorporated into the capillary moisture break and drainage layer. In this case, the subfloor drains are laid directly on the flat subgrade and backfilled with a minimum 300 mm thick layer of 9.5 mm clear stone (OPSS.MUNI 1004), HPB, or approved equivalent, vibrated to a dense state. Any solid collection pipes must be sloped so that they positively discharge to the sumps.

Without proper filtering there may be entry of fines from the surrounding subgrade soils into the bedding. This loss of ground could result in a loss of support of the slab and clogging of the subfloor drainage system.

The use of excavated bedrock spoil to restore subgrade elevations is to be specifically prohibited. This bedrock spoil cannot be adequately compacted to provide support for the slab on grade and is not to be reused below any settlement sensitive areas.

Prior to placement of the capillary moisture break and construction of the slab, the cut subgrade be cut and inspected by Grounded for obvious exposed loose or disturbed areas, or for areas containing excessive deleterious materials or moisture. These areas shall be recompacted in place and retested, or else replaced with Granular B placed as engineered fill (in lifts 150 mm thick or less and compacted to a minimum of 98 percent SPMDD). The slab on grade should not be placed on frozen subgrade, to prevent settlement of the slab as the subgrade thaws. Areas of frozen subgrade should be removed during subgrade preparation.

3.6 Long-Term Groundwater and Seepage Control

To limit seepage to the extent practicable, exterior grades adjacent to foundation walls should be sloped at a minimum 2 percent gradient away from the wall for 1.2 m minimum.

For a conventional drained basement approach, perimeter and subfloor drainage systems are required for the underground structure. Subfloor drainage collects and removes the seepage that infiltrates under the floor. Perimeter drainage collects and removes seepage that infiltrates at the foundation walls. The exterior faces of foundation walls should be provided with a layer of waterproofing to protect interior finishes.

Subfloor drainage pipes are to be spaced at an average 6 m (measured on-centres).

The walls of the substructure are to be fully drained to eliminate hydrostatic pressure. Where drained basement walls are made directly against shoring, prefabricated composite drainage panel covering the blind side of the wall is used to provide drainage. Seepage from the composite drainage panel is collected and discharged through the basement wall in solid ports directly to the sumps. A layer of waterproofing placed between the drain core product and the basement wall should be considered to protect interior finishes from moisture.

In an open cut excavation, basement wall drainage is installed directly against the basement wall from the open cut side. Perimeter foundation drains made in this application comprise perforated pipe (minimum 100 mm diameter) surrounded by a granular filter of OPSS.MUNI HL-8 Coarse Aggregate providing a minimum 300 mm of cover over the drain pipe.

Typical basement drainage details are appended.

The perimeter and subfloor drainage systems are critical structural elements since they eliminate hydrostatic pressure from acting on the basement walls and floor slab. The sumps that ensure the performance of these systems must have a duplexed pump arrangement providing 100% redundancy, and they must be on emergency power. The sumps should be sized by the mechanical engineer to adequately accommodate the estimated volume of water seepage.

The permanent dewatering requirements are provided in Grounded's Hydrogeological Report (File No. 22-302).

If any water is to be discharged to the storm or sanitary sewers, the City of Mississauga and Region of Peel will require Discharge Agreements to be in place.

4 Considerations for Construction

4.1 Excavations

Excavations must be carried out in accordance with the *Occupational Health and Safety Act – Regulation 213/91 – Construction Projects (Part III - Excavations, Section 222 through 242)*. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. For practical purposes:

- The earth fill is a Type 3 soil
- The native till is a Type 2 soil

In accordance with the regulation's requirements, the soil must be suitably sloped and/or braced where workers must enter a trench or excavation deeper than 1.2 m. Safe excavation slopes (of no more than 3 m in height) by soil type are stipulated as follows:

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

Minimum support system requirements for steeper excavations are stipulated in Sections 235 through 238 and 241 of the Act and Regulations and include provisions for timbering, shoring and moveable trench boxes. Any excavation slopes greater than 3 m in height should be checked by Grounded for global stability issues.

Bedrock is not considered a soil under the Act. Vertical excavations made in sound bedrock are generally self-supporting provided the rock bedding is horizontally oriented. If deemed necessary, rock bolts can be used to anchor a layer of protective mesh that will protect workers from loose rock spalling from the face of excavation. The rock face must be inspected by Grounded to determine that no other support system is required to prevent the spalling of loose rock, and to confirm that all loose spall material at risk of falling upon a worker is removed (Section 233 of the above noted regulations).

Larger obstructions (e.g. buried concrete debris, other obstructions) not directly observed in the boreholes are likely present in the earth fill. Similarly, larger inclusions (e.g. cobbles and boulders) may be encountered in the native soils. The size and distribution of these obstructions cannot be predicted with boreholes, as the split spoon sampler is not large enough to capture particles of this size. Provision must be made in excavation contracts to allocate risks associated with the time spent and equipment utilized to remove or penetrate such obstructions when encountered.

Excess soil is now governed by Ontario Regulation 406/19: On-Site and Excess Soil Management. As of January 1, 2023, the Project Leader (typically the owner) may be required to file a notice in the excess soil registry and a Qualified Person (within the meaning of O.Reg. 153/04) may be required to prepare the associated planning documents and/or develop and implement a tracking system in accordance with the Soil Rules, to track each load of excess soil during its transportation and deposit before removing excess soil from the project area.

Excavations will penetrate weathered and sound bedrock. Georgian Bay Formation bedrock is a rippable rock that can be removed with conventional excavation equipment once it has been broken by ripper tooth or hoe ram. Creating detailed excavation shapes for foundations etc. is normally accomplished by hoe ram. The removal of rock from a vertical face without over-excavation, which can happen inadvertently by dislodging additional rock, is largely dependent on machine operator skill. The contractor shall exercise caution and implement the appropriate techniques to reduce the amount of disturbance to the rock mass (rock fracturing) with the excavator.

If excavation faces must be made neat (such as beside an existing footing), a line of excavation can be provided by line drilling the rock a series of closely-spaced vertical holes (100 mm diameter, spaced at 300 mm on centre) to provide a preferential vertical break path for the excavation face.

Georgian Bay Formation bedrock contains beds of harder calcareous beds (e.g. limestone). When excavating this bedrock, it should be expected that these harder layers will be encountered. Hard layers interbedded within the shale are normally broken with hoe mounted hydraulic rams before excavation.

Limestone beds may also be found to straddle the founding elevation, in which case the entire thickness of the hard limestone layer must be removed to expose founding subgrade as it is not possible to remove part of one of these layers. This will in turn result in excess rock removal not intrinsic to the project requirements. The risk and responsibility for the excess rock removal under these circumstances, and the supply and placement of the extra concrete to restore the foundation grade, must be addressed in the contract documents for foundations, excavation, and shoring contractors.

4.2 Short-Term Groundwater Control

Considerations pertaining to groundwater discharge quantities and quality are discussed in Grounded's preliminary hydrogeological report for the site, under separate cover (File No: 22-302).

The groundwater table at Elev. 77.1± m is above the bulk excavation level for P4. Excavations will generally be made above the groundwater table, in relatively low permeability native soils that preclude the free flow of water into excavations. Positive dewatering of the bedrock is not required.

Cohesionless wet zones were encountered in several of the boreholes. If these cohesionless zones are penetrated, some seepage from these wet zones should be anticipated. However, these zones are likely of limited extent and are not horizontally continuous layers.

On this basis, seepage into excavations may be allowed to drain into the excavation and then controlled by a conventional sump pump arrangement. Nevertheless, delays in excavation will occur as the seepage is controlled and these delays should be anticipated in the construction schedule.

The City of Mississauga and/or Region of Peel will require a Discharge Agreement in the short term, if any water is to be discharged to the storm or sanitary sewers during construction.

4.3 Earth-Retention Shoring Systems

No excavation shall extend below the foundations of existing adjacent structures without adequate alternative support being provided.

Excavation zone of influence guidelines are appended.

Continuous interlocking caisson wall shoring is to be used where the excavation must be constructed as a rigid shoring system. Caisson wall shoring preserves the support capabilities and integrity of the soil beneath existing foundations of adjacent buildings, in a state akin to the at-rest condition. Otherwise, excavations can be supported using conventional soldier pile and lagging walls.

The railway may have additional design, performance, or monitoring criteria for earth retention support systems next to their lands.

4.3.1 Lateral Earth Pressure Distribution

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

$$\boldsymbol{P = 0.8\,K[\gamma H + q] + \gamma_w h_w}$$

P = maximum horizontal pressure (kPa)
K = earth pressure coefficient (see Section 3.3)
H = total depth of the excavation (m)

h_w = height of groundwater (m) above the base of excavation
γ = soil bulk unit weight (kN/m3)
q = total surcharge loading (kPa)

Where shoring walls are drained to effectively eliminate hydrostatic pressure on the shoring system (e.g. pile and lagging walls), h_w is equal to zero. For the design of impermeable shoring, a design groundwater table at Elev. 77.1 m must be accounted for.

In cohesive soils, the lateral earth pressure distribution is trapezoidal, uniformly increasing from zero to the maximum pressure defined in the equation above over the top and bottom quarter (H/4) of the shoring.

Where the excavation penetrates the bedrock, the rock excavation is nominally self-supporting in a vertical face, provided the rock bedding is horizontally oriented. The requirement for extending lagging into partially weathered rock depends on the quality of the excavation cut and the degree of weathering.

4.3.2 Soldier Pile Toe Embedment

Soldier pile toes will be made in sound bedrock. Soldier pile toes resist horizontal movement due to the passive earth pressure acting on the toe below the base of excavation. The maximum factored vertical geotechnical resistance at ULS for the design of a pile embedded in the sound bedrock is 10 MPa. The maximum factored lateral geotechnical resistance at ULS of the undisturbed rock is 1 MPa.

There are zones of soil in the subgrade that are wet, cohesionless, and permeable. Temporarily cased holes advanced to the bedrock surface are required to prevent borehole caving during installations in drilled holes. To prevent groundwater issues (groundwater inflow, caving and blowback into the drill holes, disturbance to placed concrete, etc.) during drilling and installation, construction methods such as utilizing temporary liners, pre-advancing liners deeper than the augered holes, mud/slurry/polymer drilling techniques, tremie pour concrete, or other methods as deemed necessary by the shoring contractor are required. Concrete for shoring piles and fillers must be placed by tremie method wherever there is more than 300 mm of water or fluid at the base of the drill hole.

Exposed bedrock of the Georgian Bay Formation deteriorates with time. Within 12 months of exposure, excavation faces made within this bedrock flake and recede as much as 300 mm, generally in the form of coin-size shale particles dropping from the face on a constant basis. The deteriorated rock loses internal integrity and bearing capability. Solider piles for the shoring system are typically advanced at least 1 m below the base of the excavation (to be confirmed by the geostructural engineer) to accommodate this weathering and still ensure that the required lateral and vertical bearing resistances can be utilized.

4.3.3 Lateral Bracing Elements

The shoring system at this site will require lateral bracing. If feasible, the shoring system should be supported by pre-stressed soil anchors (tiebacks) extending into the subgrade of the adjacent properties. To limit the movement of the shoring system as much as is practically possible, tiebacks are installed and stressed as excavation proceeds. The use of tiebacks through adjacent properties requires the consent (through encroachment agreements) of the adjacent property owners.

In the very stiff to hard native till, it is expected that post-grouted anchors can be made such that an anchor will safely carry up to 70 kN/m of adhered anchor length (at a nominal borehole diameter of 150 mm). Conventional earth anchors made in Georgian Bay Formation bedrock can be designed using a working adhesion of 620 kPa. A single anchor cannot be supported simultaneously by both the native soil and bedrock due to strain incompatibility.

At least one prototype anchor per tieback level must be performance-tested to 200% of the design load to demonstrate the anchor capacity and validate design assumptions. Given the potential variability in soil conditions or installation quality, all production anchors must also be proof-tested to 133% of the design load.

The bedrock below the proposed FFE is suitable for the placement of raker foundations. Raker footings established on bedrock at an inclination of 45 degrees can be designed using a maximum factored geotechnical resistance at ULS of 2500 kPa.

4.4 Site Work

To better protect wet undisturbed subgrade, excavations exposing wet soils must be cut neat, inspected, and then immediately protected with a skim coat of concrete (i.e. a mud mat). Wet sands are susceptible to degradation and disturbance due to even mild site work, frost, weather, or a combination thereof.

The effects of work on site can greatly impact soil integrity. Care must be taken to prevent this damage. Site work carried out during periods of inclement weather may result in the subgrade becoming disturbed, unless a granular working mat is placed to preserve the subgrade soils in their undisturbed condition. Subgrade preparation activities should not be conducted in wet weather and the project must be scheduled accordingly.

If site work causes disturbance to the subgrade, removal of the disturbed soils and the use of granular fill material for site restoration or underfloor fill will be required at additional cost to the project.

It is construction activity itself that often imparts the most severe loading conditions on the subgrade. Special provisions such as end dumping and forward spreading of earth and aggregate fills, restricted construction lanes, and half-loads during placement of the granular base and other work may be required, especially if construction is carried out during unfavourable weather.

Adequate temporary frost protection for the founding subgrade must be provided if construction proceeds in freezing weather conditions. The subgrade at this site is susceptible to frost damage. The slab on grade should not be placed on frozen subgrade, to prevent settlement of the slab as the subgrade thaws. Areas of frozen subgrade should be removed during subgrade preparation. Depending on the project context, consideration should be given to frost effects (heaving, softening, etc.) on exposed subgrade surfaces.

The exposed Georgian Bay Formation deteriorates with time. Exposed excavation faces have been found to flake and recede as much as 300 mm with 12 months exposure. This recession generally takes the form of coin size shale particles dropping from the face on a constant basis. The deteriorated rock loses internal integrity and bearing capability. If bedrock is to be exposed for prolonged periods of time, it is recommended that a skim coat of concrete be used to protect the surface of bedrock from slaking and other degradation resulting from weathering.

4.5 Engineering Review

By issuing this preliminary report, Grounded Engineering has assumed the role of Geotechnical Engineer of Record for this site. Grounded should be retained to review the structural engineering drawings prior to issue or construction to ensure that the recommendations in this report have been appropriately implemented.

All foundation installations must be reviewed in the field by Grounded, the Geotechnical Engineer of Record, as they are constructed. The on-site review of foundation installations and the condition of the founding subgrade as the foundations are constructed is as much a part of the geotechnical engineering design function as the design itself; it is also required by Section 4.2.2.2 of the Ontario Building Code. If Grounded is not retained to carry out foundation engineering field review during construction, then Grounded accepts no responsibility for the performance or non-performance of the foundations, even if they are constructed in general conformance with the engineering design advice contained in this report.

The long-term performance of a slab on grade is highly dependent upon the subgrade support and drainage conditions. Strict procedures must be maintained during construction to maintain the integrity of the subgrade to the extent possible. The design advice in this report is based on an assessment of the subgrade support capabilities as indicated by the boreholes. These conditions may vary across the site depending on the final design grades and therefore, the preparation of the subgrade and the compaction of all fill should be monitored by Grounded at the time of construction to confirm material quality, thickness, and to ensure adequate compaction.

A visual pre-construction survey of adjacent lands and buildings is recommended to be completed prior to the start of any construction. This documents the baseline condition and can prevent unwarranted damage claims. Any shoring system, regardless of the execution and design, has the potential for movement. Small changes in stress or soil volume can cause cracking in adjacent buildings.

5 Limitations and Restrictions

This preliminary geotechnical engineering feasibility study is intended for due diligence purposes only. At detailed design, site-specific boreholes, groundwater monitoring wells, and updated detailed geotechnical engineering advice are required. Once completed, the future detailed geotechnical engineering report by Grounded Engineering would then supersede this preliminary report.

5.1 Investigation Procedures

The geotechnical engineering analysis and advice provided are based on the factual borehole information observed and recorded by Grounded. The investigation methodology and engineering analysis methods used to carry out this scope of work are consistent with conventional standard practice by Grounded as well as other geotechnical consultants, working under similar conditions and constraints (time, financial and physical).

Borehole drilling services were provided to Grounded by a specialist professional contractor. The drilling was observed and recorded by Grounded's field supervisor on a full-time basis. Drilling was conducted using conventional drilling rigs equipped with hollow stem augers. As drilling proceeded, groundwater observations were made in the boreholes. Based on examination of recovered borehole samples, our field supervisor made a record of borehole and drilling observations. The field samples were secured in air-tight clean jars and bags and taken to the Grounded soil laboratory where they were each logged and reviewed by the geotechnical engineering team and the senior reviewer.

The Split-Barrel Method technique (ASTM D1586) was used to obtain the soils samples. The sampling was conducted at conventional intervals and not continuously. As such, stratigraphic interpolation between samples is required and stratigraphic boundary lines do not represent exact depths of geological change. They should be taken as gradual transition zones between soil or rock types.

A carefully conducted, fully comprehensive investigation and sampling scope of work carried out under the most stringent level of oversight may still fail to detect certain ground conditions. As such, users of this report must be aware of the risks inherent in using engineered field investigations to observe and record subsurface conditions. As a necessary requirement of working with discrete test locations, Grounded has assumed that the conditions between test locations are the same as the test locations themselves, for the purposes of providing geotechnical engineering advice.

It is not possible to design a field investigation with enough test locations that would provide complete subsurface information, nor is it possible to provide geotechnical engineering advice that completely identifies or quantifies every element that could affect construction, scheduling, or tendering. Contractors undertaking work based on this report (in whole or in part) must make their own determination of how they may be affected by the subsurface conditions, based on their own analysis of the factual information provided and based on their own means and methods. Contractors using this report must be aware of the risks implicit in using factual information at discrete test locations to infer subsurface conditions across the site and are directed to conduct their own investigations as needed.

5.2 Site and Scope Changes

Natural occurrences, the passage of time, local construction, and other human activity all have the potential to directly or indirectly alter the subsurface conditions at or near the project site. Contractual obligations related to groundwater or stormwater control, disturbed soils, frost protection, etc. must be considered with attention and care as they relate to potential site alteration.

The geotechnical engineering advice provided in this report is based on the factual observations made from the site investigations as reported. It is intended for use by the owner and their retained design team. If there are changes to the features of the development or to the scope, the interpreted subsurface information, geotechnical engineering design parameters, advice, and discussion on construction considerations may not be relevant or complete for the project. Grounded should be retained to review the implications of such changes with respect to the contents of this report.

This report provides preliminary geotechnical engineering advice intended for use by the owner and their retained design team for due diligence only. These preliminary interpretations, design parameters, advice, and discussion on construction considerations are not complete. A detailed site-specific geotechnical investigation must be conducted by Grounded during detailed design to confirm and update the preliminary recommendations provided here.

5.3 Report Use

The authorized users of this report are Edenshaw Queen Developments Limited and their design team, for whom this report has been prepared. Grounded Engineering Inc. maintains the copyright and ownership of this document. Reproduction of this report in any format or medium requires explicit prior authorization from Grounded Engineering Inc.

The local municipal/regional governing bodies may also make use of and rely upon this report, subject to the limitations as stated.

6 Closure

If the design team has any questions regarding the discussion and advice provided, please do not hesitate to have them contact our office. We trust that this report meets your requirements at present.

For and on behalf of our team,

GROUNDED
ENGINEERING

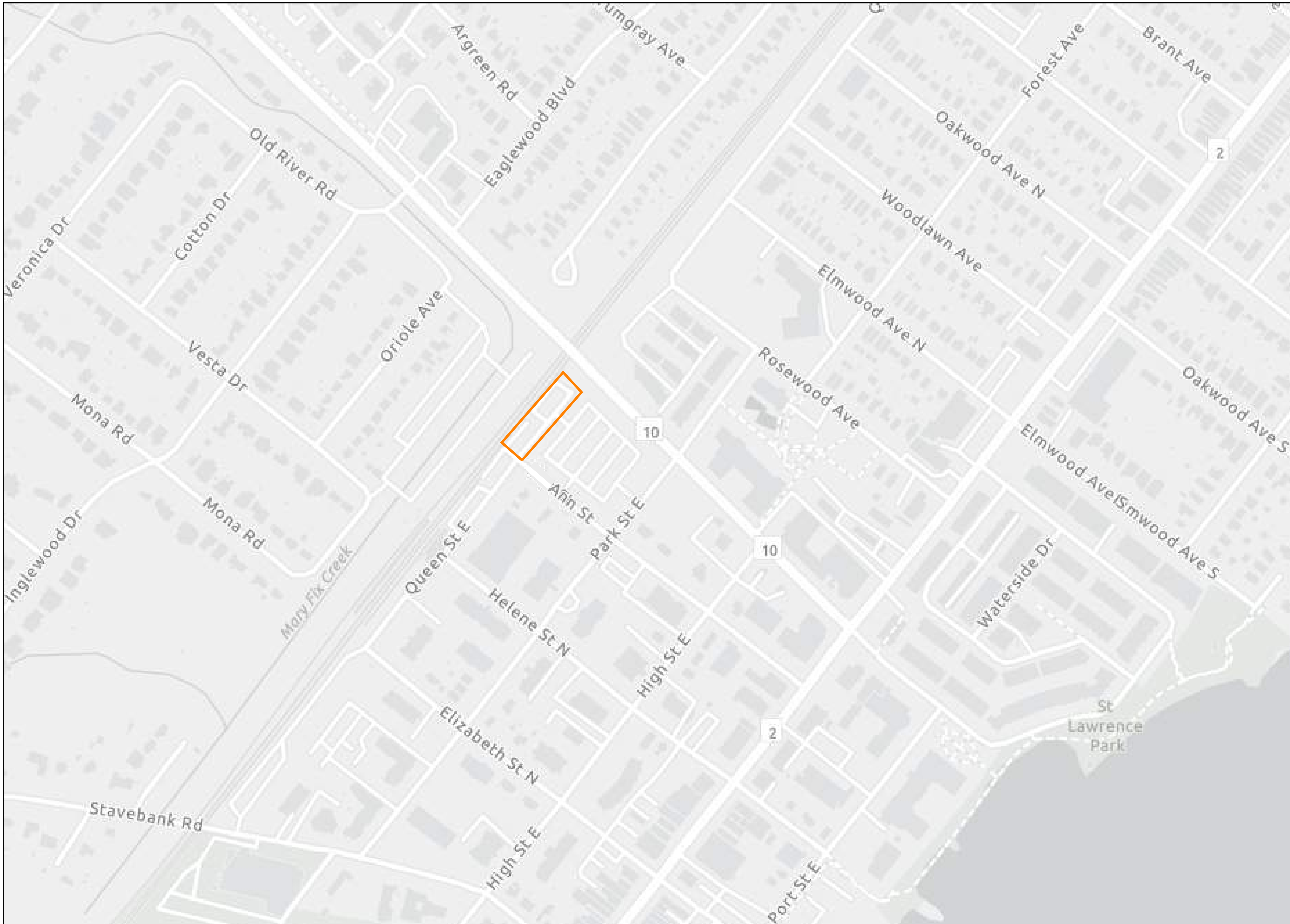
LICENSED PROFESSIONAL ENGINEER
M. DIEZ DE AUX
100159147
10/02/2023
PROVINCE OF ONTARIO

AG

Arman Gelimforoush, MASc, EIT
Project Manager

Michael Diez de Aux, M.A.Sc., P.Geo., P.Eng.
Associate

FIGURES



GROUNDED ENGINEERING

1 BANIGAN DRIVE, TORONTO, ONT., M4H 1G3
www.groundedeng.ca

LEGEND

APPROXIMATE PROPERTY BOUNDARY

Note

Reference

ArcGIS Online Map 2023

Project

88 PARK ST E, MISSSISSAUGA, ONTARIO

Figure Title

SITE LOCATION PLAN

North

TRUE

PROJECT

Date

FEBRUARY 2022

Scale

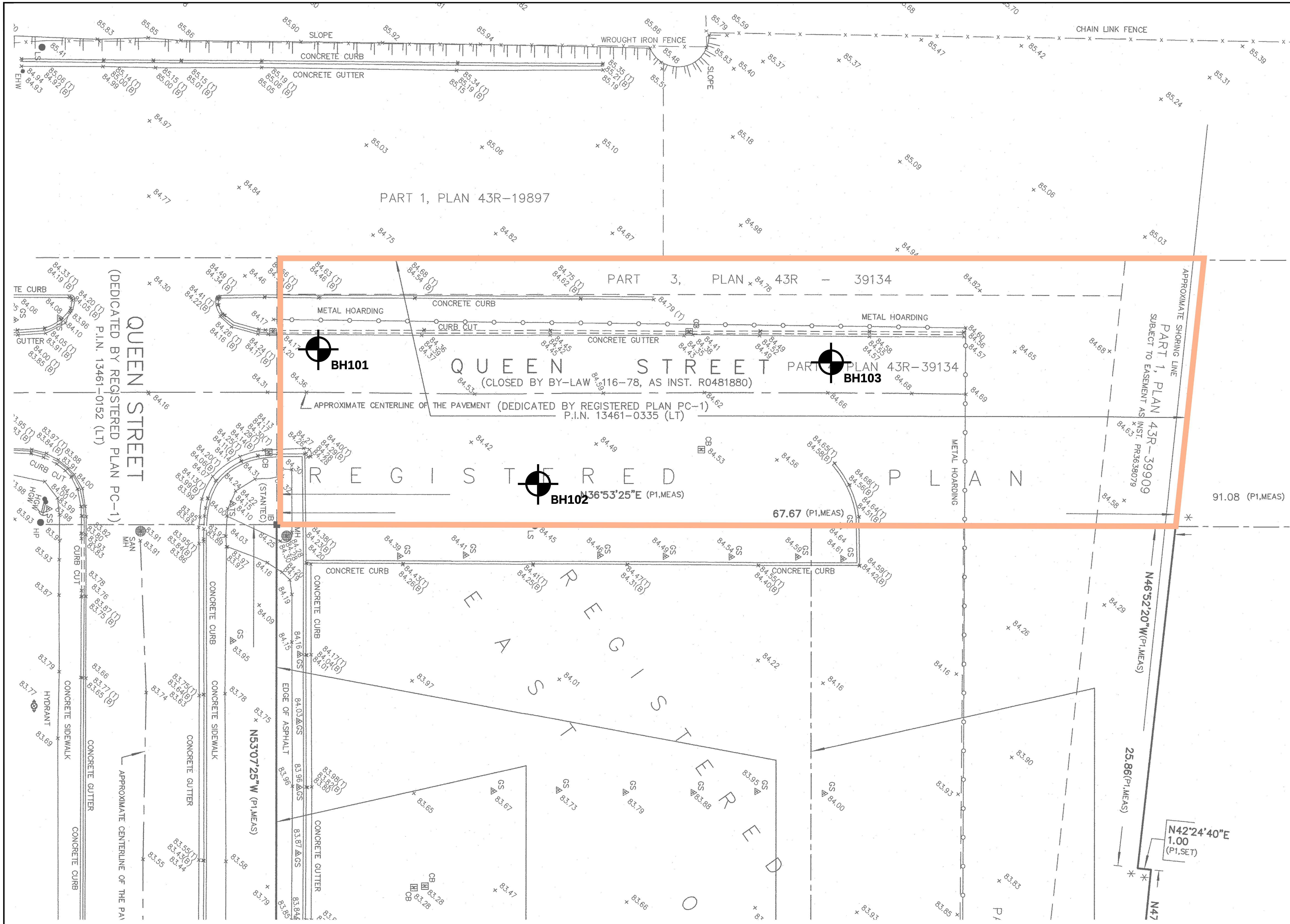
0m 50m 100m

Job No

22-302

Figure No

FIGURE 1



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LEGEND

PROPERTY BOUNDARY
PROPOSED BOREHOLE BY GROUNDED

Note

Reference

Survey Drawing job no. 3508-0.
Reference no. 3508-0T.DWG.
Dated: December 13, 2021.
Prepared by R. Avis Surveying Inc.

Project

88 PARK ST E, MISSISSAUGA, ONTARIO

Figure Title

BOREHOLE LOCATION PLAN

North

PROJECT

Date

FEBRUARY 2023

Scale

0m 10m 20m

Job No

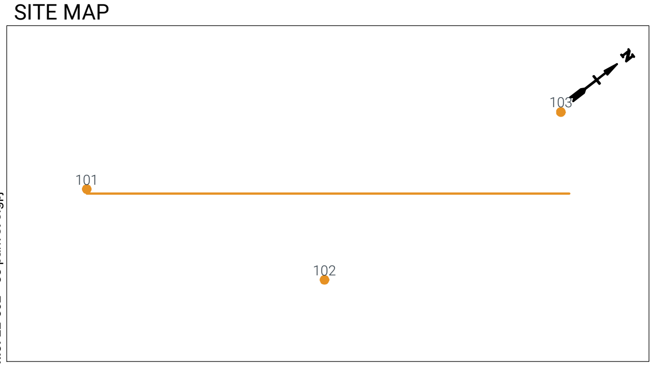
22-302

Figure No

FIGURE 1

Boreholes Equally Spaced

SITE MAP



BOREHOLE STRATIGRAPHY LEGEND

- Asphalt
- Aggregate
- Fill
- Silt and Clay Till
- Sand and Silt
- Clayey Silt Till
- Bedrock (inferred)
- Silt and Sand

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LEGEND

- FILL
- GRAVELS (gravel to gravelly sand)
- SILT TO SAND (not till)
- COHESIONLESS TILLS
- COHESIVE SOILS (clayey silt to clay, incl. tills)
- DISTURBED/REWORKED/ORGANIC

- water level, unstabilized
- water level, stabilized (latest)
- water level, stabilized (highest)

Project
88 PARK ST E MISSISSAUGA

Figure Title
SUBSURFACE PROFILE

Date
FEBRUARY 2023

Scale
AS INDICATED

Job No
22-302

Figure No
FIGURE 3

APPENDIX A

BOREHOLE LOG TERMINOLOGY

GROUNDED ENGINEERING

SAMPLING/TESTING METHODS

SS: split spoon sample

AS: auger sample

GS: grab sample

FV: shear vane

DP: direct push

PMT: pressuremeter test

ST: shelby tube

CORE: soil coring

RUN: rock coring

SYMBOLS & ABBREVIATIONS

MC: moisture content

LL: liquid limit

PL: plastic limit

NP: non-plastic

γ: soil unit weight (bulk)

G_s: specific gravity

S_u: undrained shear strength

unstabilized water level

1st water level measurement

2nd water level measurement most recent

water level measurement

ENVIRONMENTAL SAMPLES

M&I: metals and inorganic parameters

PAH: polycyclic aromatic hydrocarbon

PCB: polychlorinated biphenyl

VOC: volatile organic compound

PHC: petroleum hydrocarbon

BTEX: benzene, toluene, ethylbenzene and xylene

PPM: parts per million

FIELD MOISTURE (based on tactile inspection)

DRY: no observable pore water

MOIST: inferred pore water, not observable (i.e. grey, cool, etc.)

WET: visible pore water

COHESIONLESS

Relative Density	N-Value
Very Loose	<4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very Dense	>50

COHESIVE

Consistency	N-Value	Su (kPa)
Very Soft	<2	<12
Soft	2 - 4	12 - 25
Firm	4 - 8	25 - 50
Stiff	8 - 15	50 - 100
Very Stiff	15 - 30	100 - 200
Hard	>30	>200

COMPOSITION

Term	% by weight
trace silt	<10
some silt	10 - 20
silt***y***	20 - 35
sand ***and*** silt	>35

ASTM STANDARDS

ASTM D1586 Standard Penetration Test (SPT)
Driving a 51 mm O.D. split-barrel sampler ("split spoon") into soil with a 63.5 kg weight free falling 760 mm. The blows required to drive the split spoon 300 mm ("bpf") after an initial penetration of 150 mm is referred to as the N-Value.

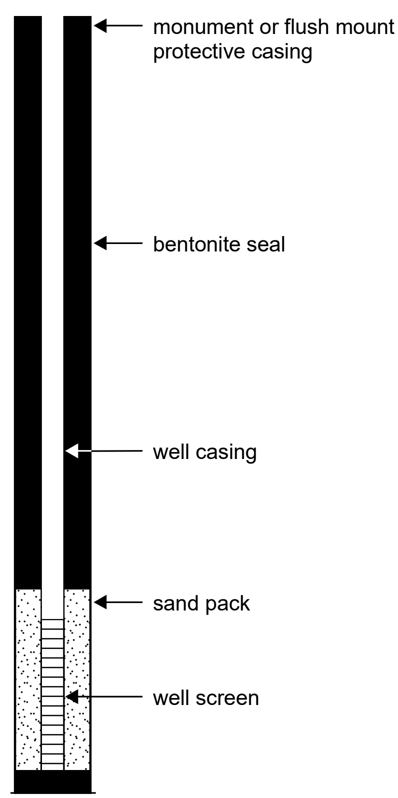
ASTM D3441 Cone Penetration Test (CPT)
Pushing an internal still rod with a outer hollow rod ("sleeve") tipped with a cone with an apex angle of 60° and a cross-sectional area of 1000 mm^2 into soil. The resistance is measured in the sleeve and at the tip to determine the skin friction and the tip resistance.

ASTM D2573 Field Vane Test (FVT)
Pushing a four blade vane into soil and rotating it from the surface to determine the torque required to shear a cylindrical surface with the vane. The torque is converted to the shear strength of the soil using a limit equilibrium analysis.

ASTM D1587 Shelby Tubes (ST)
Pushing a thin-walled metal tube into the in-situ soil at the bottom of a borehole, removing the tube and sealing the ends to prevent soil movement or changes in moisture content for the purposes of extracting a relatively undisturbed sample.

ASTM D4719 Pressuremeter Test (PMT)
Place an inflatable cylindrical probe into a pre-drilled hole and expanding it while measuring the change in volume and pressure in the probe. It is inflated under either equal pressure increments or equal volume increments. This provides the stress-strain response of the soil.

WELL LEGEND



BOREHOLE LOG 101

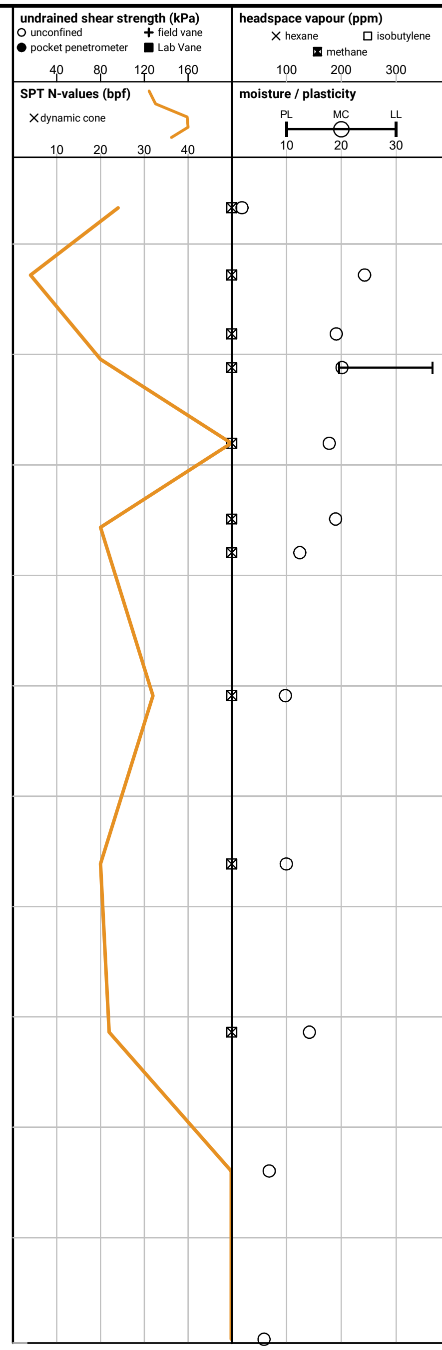
GROUNDED ENGINEERING

Date Started : Jan 6, 2023
Position : E: 614233, N: 4823630 (UTM 17T)
Elev. Datum : Geodetic

File No. : 22-302 **Project** : 88 Park St E, Mississauga **Client** : Edenshaw Queen Developments Limited

drill method : CME 75 — hollow stem augers OD=215 mm

elev depth (m)	description	number	type	SPT N-value	lab data and comments
84.8	**GROUND SURFACE**				
84.6 / 0.2	100mm **ASPHALT**				
	100mm **AGGREGATE**	1	SS	24	SS1: EC/SAR, H-Ms, Metals, ORPs, PCBs
	FILL, sand, trace gravel, trace asphalt, compact, grey, moist ...at 0.8 m, silty clay, trace sand, firm, wet	2	SS	4	SS2: BTEX, PAHs, PHCs, VOCs
83.3 / 1.5	**SILT AND CLAY**, some sand, trace gravel, very stiff, brown, moist (GLACIAL TILL)	3A	SS		
		3B	SS	20	4 20 46 30 SS3B: EC/SAR, H-Ms, Metals, ORPs, PAHs
82.5 / 2.3	**SAND AND SILT**, trace gravel, very dense, brown, moist	4	SS	58	
	...at 3.0 m, compact	5A	SS	20	
81.3 / 3.5	**CLAYEY SILT**, some sand, very stiff to hard, grey, moist (GLACIAL TILL)	5B	SS		
		6	SS	32	SS6: BTEX, PHCs
		7	SS	20	
					7.2m: auger grinding **to 7.3m**
	...at 7.6 m, sandy, trace shale and limestone fragments	8	SS	22	SS8: VOCs
75.7 / 9.1	**INFERRED BEDROCK**, shale and limestone fragments, grey	9	SS	75 / 75mm	**9.1m:** auger grinding
74.1 / 10.7		10	SS	50 / 75mm	



END OF BOREHOLE

Water level and cave not measured upon completion of drilling.

50 mm dia. monitoring well installed.

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jan 9, 2023	7.7	77.1
Jan 12, 2023	7.7	77.1
Jan 27, 2023	7.7	77.1

file: 22-302 - 88 park st e.gpj

Tech : AK | **PM** : AG | **Rev** : MD

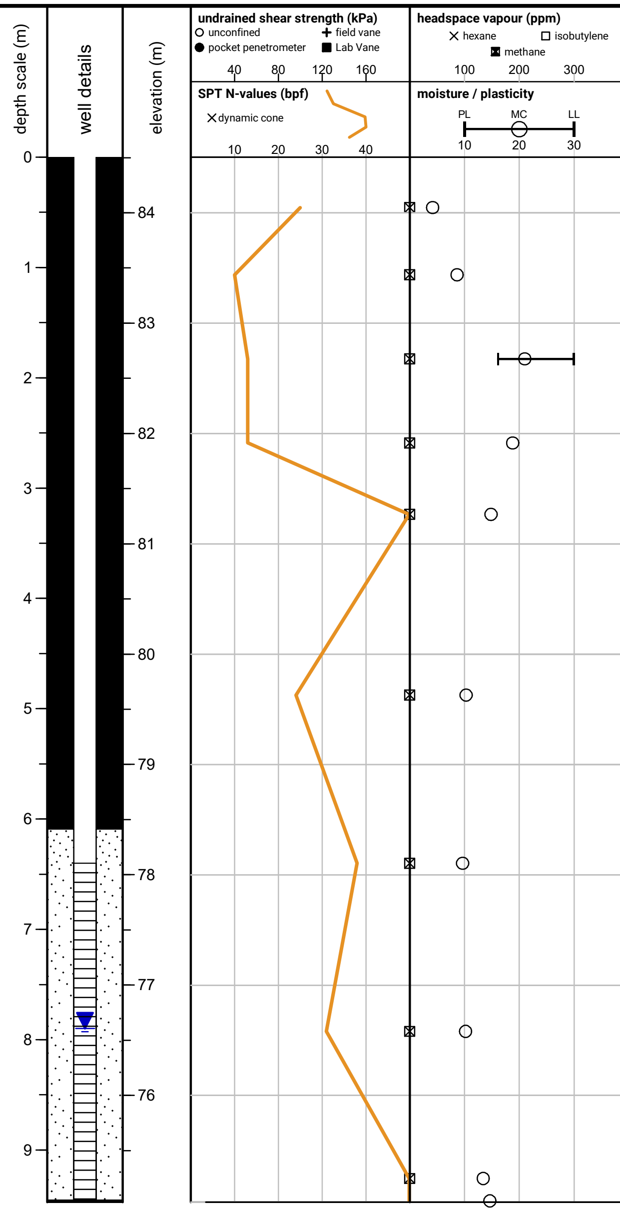
Date Started : Jan 5, 2023
Position : E: 614251, N: 4823641 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 102

File No. : 22-302 **Project** : 88 Park St E, Mississauga **Client** : Edenshaw Queen Developments Limited

Drill method: CME 75 — hollow stem augers OD=215 mm

elev depth (m)	description	number	type	SPT N-value	lab data and comments
84.5	**GROUND SURFACE**				
	75mm **ASPHALT**				
	FILL, sand, trace gravel, trace silt, trace asphalt, compact, brown, moist	1	SS	25	SS1: EC/SAR, H-Ms, Metals, ORPs, PAHs, PCBs
		2	SS	10	
83.0 1.5	**CLAYEY SILT**, trace sand, trace gravel, stiff, brown, moist (GLACIAL TILL)	3	SS	13	1 6 70 23; SS3: BTEX, PHCs, VOCs
		4	SS	13	
	...at 3.0 m, grey, hard	5	SS	56 / 225mm	SS5: EC/SAR, H-Ms, Metals, ORPs, PAHs
	...at 4.6 m, very stiff	6	SS	24	
	...at 6.1 m, hard, sandy, trace shale and limestone fragments	7	SS	38	17 28 38 17
		8	SS	31	SS8: BTEX, PHCs, VOCs
75.4 9.1	**INFERRED BEDROCK**, shale and limestone fragments, grey	9	SS	50 / 75mm	
75.0 9.5		10	SS	50 / 25mm	9.4m: auger grinding



END OF BOREHOLE

Water level and cave not measured upon completion of drilling.

50 mm dia. monitoring well installed.

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jan 9, 2023	7.9	76.6
Jan 12, 2023	7.9	76.6
Jan 27, 2023	7.9	76.6

file: 22-302 - 88 park st e.gpj

Tech : AK | **PM** : AG | **Rev** : MD

GROUNDED ENGINEERING

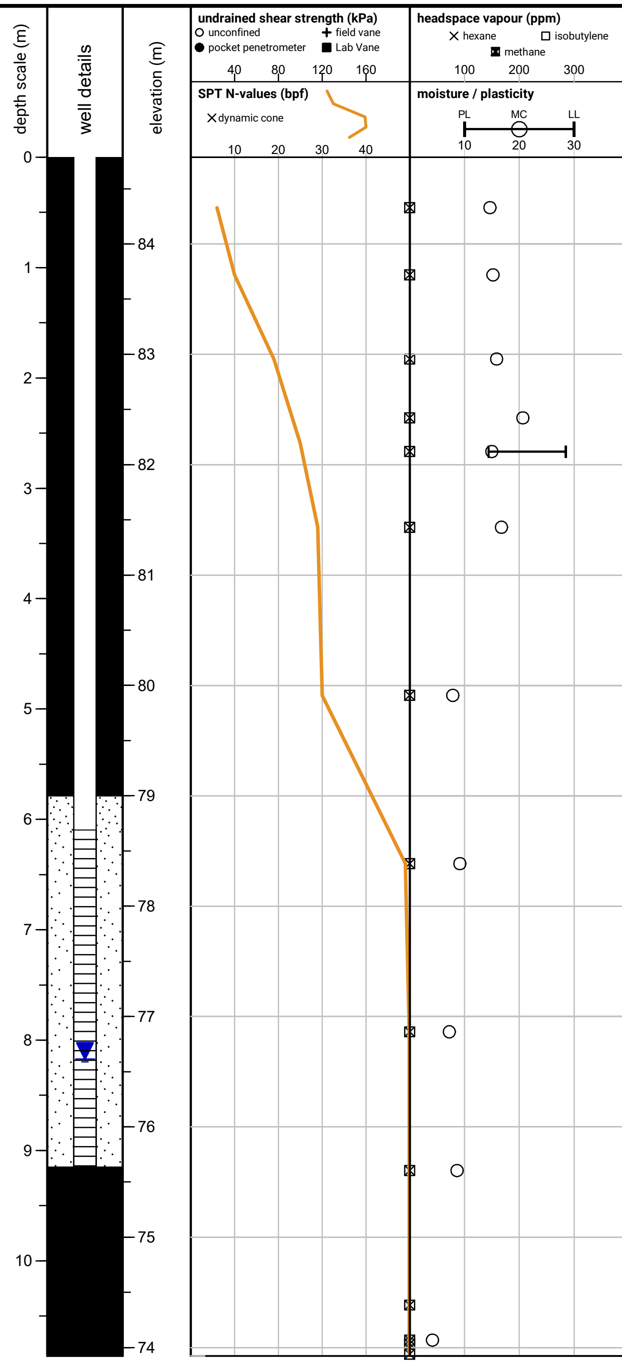
Date Started : Jan 5, 2023
Position : E: 614252, N: 4823665 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 103

File No. : 22-302 **Project** : 88 Park St E, Mississauga **Client** : Edenshaw Queen Developments Limited

drill method : CME 75 — hollow stem augers OD=215 mm

elev / depth (m)	stratigraphy description	sample number	type	SPT N-value
84.8	**GROUND SURFACE**			
	75mm **ASPHALT**			
	FILL, silty sand, trace gravel, trace clay, trace rock fragments, trace cinders, trace asphalt, trace brick fragments, loose to compact, black, moist ...at 0.8 m, orangey brown	1	SS	6
		2	SS	10
83.3 / 1.5	**SILT AND SAND**, trace gravel, trace clay, compact, brown, wet	3	SS	19
		4A	SS	
82.4 / 2.4	**CLAYEY SILT**, trace gravel, trace sand, very stiff to hard, brown, moist (GLACIAL TILL) ...at 3.0 m, grey	4B	SS	25
		5	SS	29
		6	SS	30
	...at 6.1 m, sandy, shale and limestone fragments	7	SS	49
		8	SS	51
75.7 / 9.1	**INFERRED BEDROCK**, shale and limestone fragments, grey	9	SS	50 / 75mm
		10	SS	50 / 75mm
		11	SS	50 / 100mm
		12	SS	50 / 25mm
		13	SS	50 / 50mm
73.9 / 10.9	**END OF BOREHOLE**			



lab data and comments (grain size distribution (%) (MIT): GR SA SI CL)
SS2: EC/SAR, H-Ms, Metals, ORPs
1 43 51 5
SS3: BTEX, PHCs, VOCs
1 9 67 23
SS5: EC/SAR, H-Ms, Metals, ORPs, PAHs
SS6: BTEX, PHCs
SS7: PAHs
9.1m: increased drilling resistance, auger grinding

Borehole was dry upon completion of drilling.

50 mm dia. monitoring well installed.

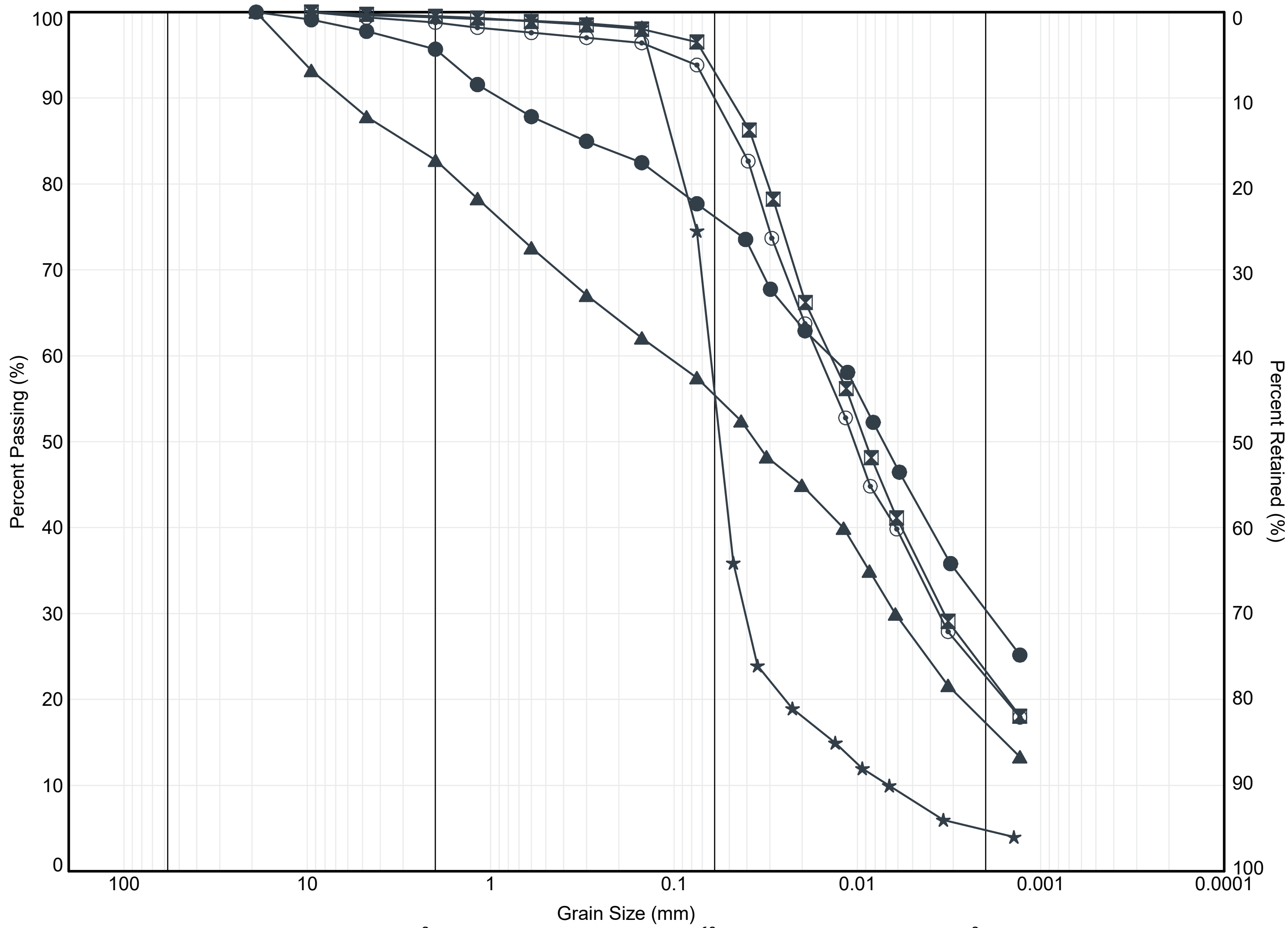
GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jan 9, 2023	8.2	76.6
Jan 12, 2023	8.4	76.4
Jan 13, 2023	8.3	76.5
Jan 27, 2023	8.2	76.6

file: 22-302 - 88 park st e.gpj

Tech : AK | **PM** : AG | **Rev** : MD

APPENDIX B



2mm 60µm 2µm

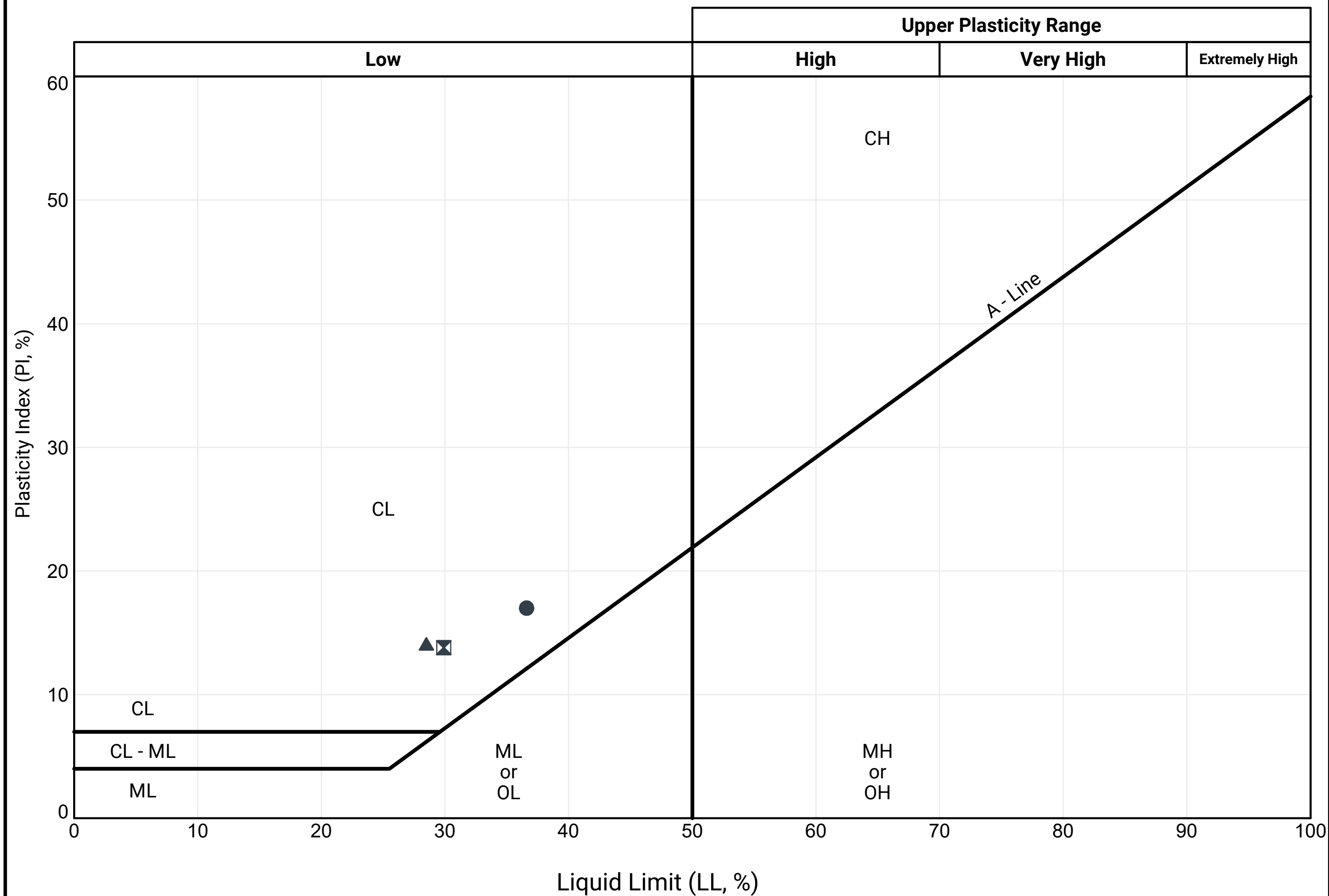
MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

	Borehole	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
					MIT SYSTEM			
●	**101**	SS3B	1.9	82.9	4	19.5	46	31
⊠	**102**	SS3	1.8	82.7	1	6.5	70	23
▲	**102**	SS7	6.4	78.1	17	27.3	38	17
★	**103**	SS3	1.8	83.0	1	43.7	51	5
⊙	**103**	SS4B	2.7	82.1	1	8.8	67	23

GROUNDED ENGINEERING

Title: **GRAIN SIZE DISTRIBUTION GLACIAL TILL WITH SAND SEAMS**

File No.: **22-302**



	Borehole	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)
●	**101**	SS3B	1.9	82.9	37	20	17
⧗	**102**	SS3	1.8	82.7	30	16	14
▲	**103**	SS4B	2.7	82.1	29	14	14

file: 22-302 - 88 park st e-ge-lt-9qmb9s3.gpj

GROUNDED ENGINEERING

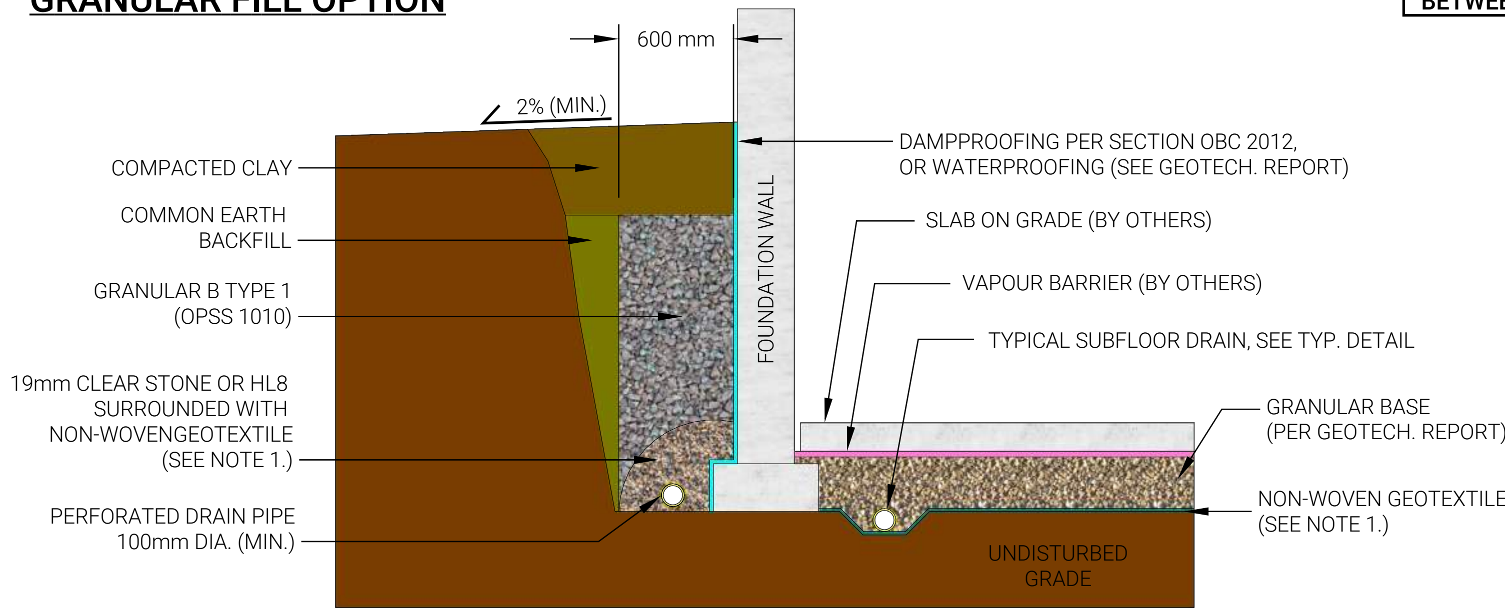
Title: **ATTERBERG LIMITS CHART**

File No.: **22-302**

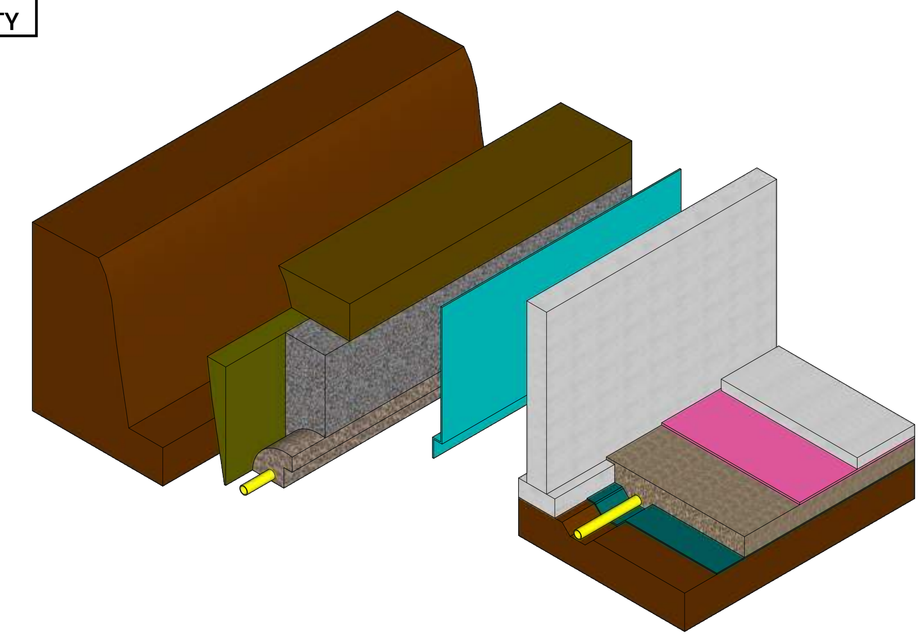
APPENDIX C

GRANULAR FILL OPTION

OBJECTS ARE COLOR-CODED BETWEEN TWO VIEWS FOR CLARITY

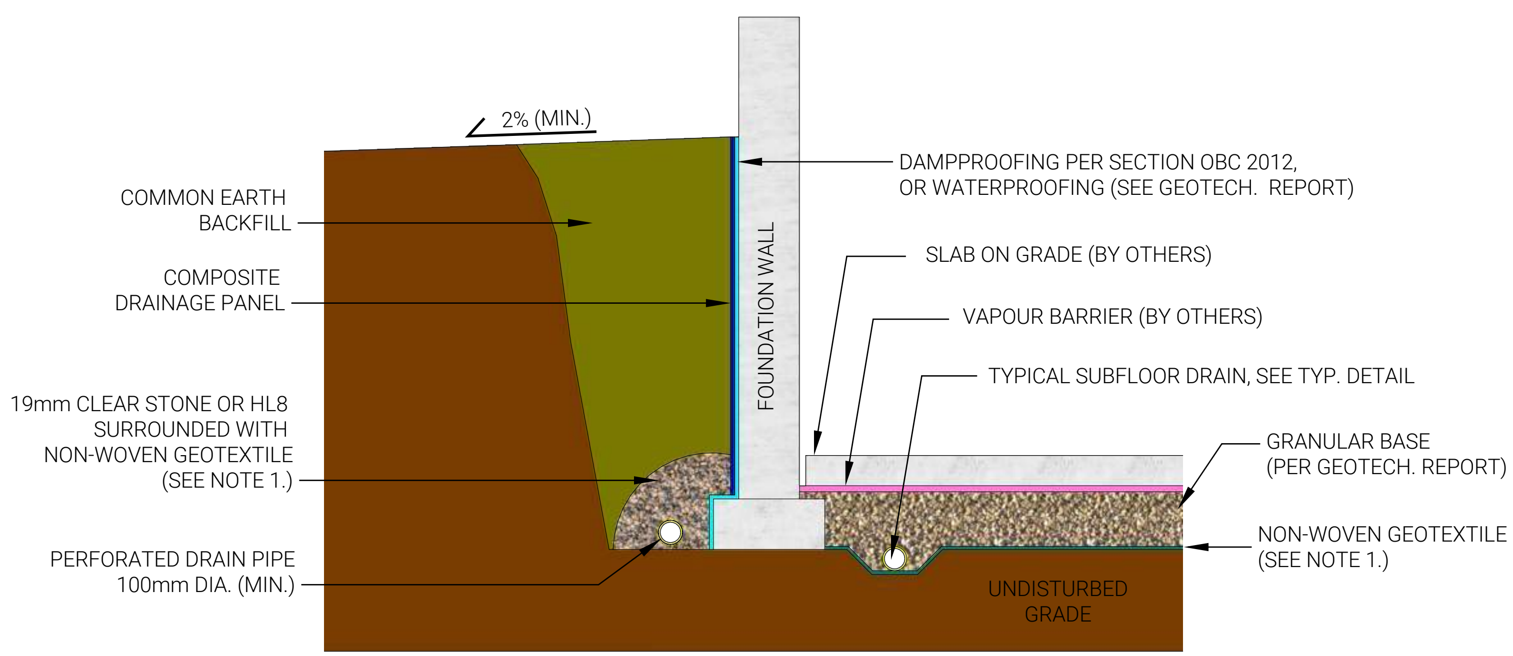


SECTIONAL VIEW

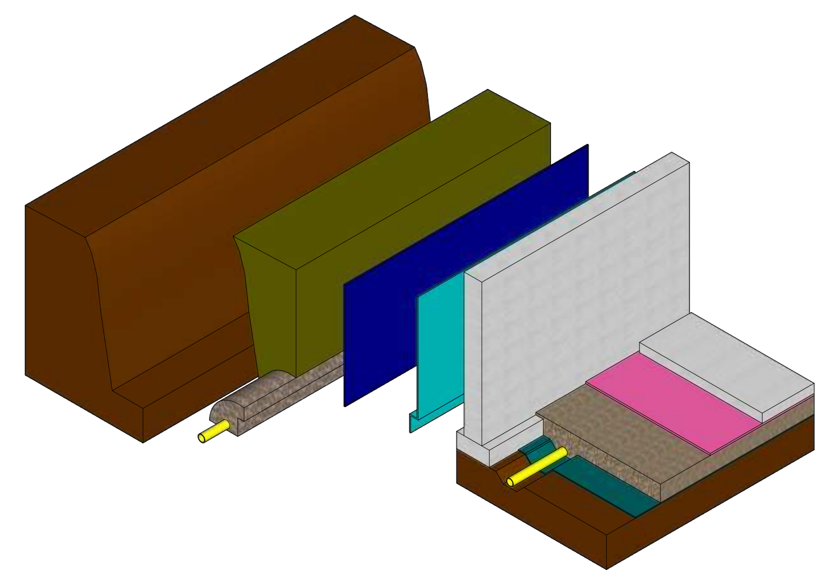


ISOMETRIC VIEW

GEO-COMPOSITE DRAINAGE PANEL OPTION



SECTIONAL VIEW



ISOMETRIC VIEW

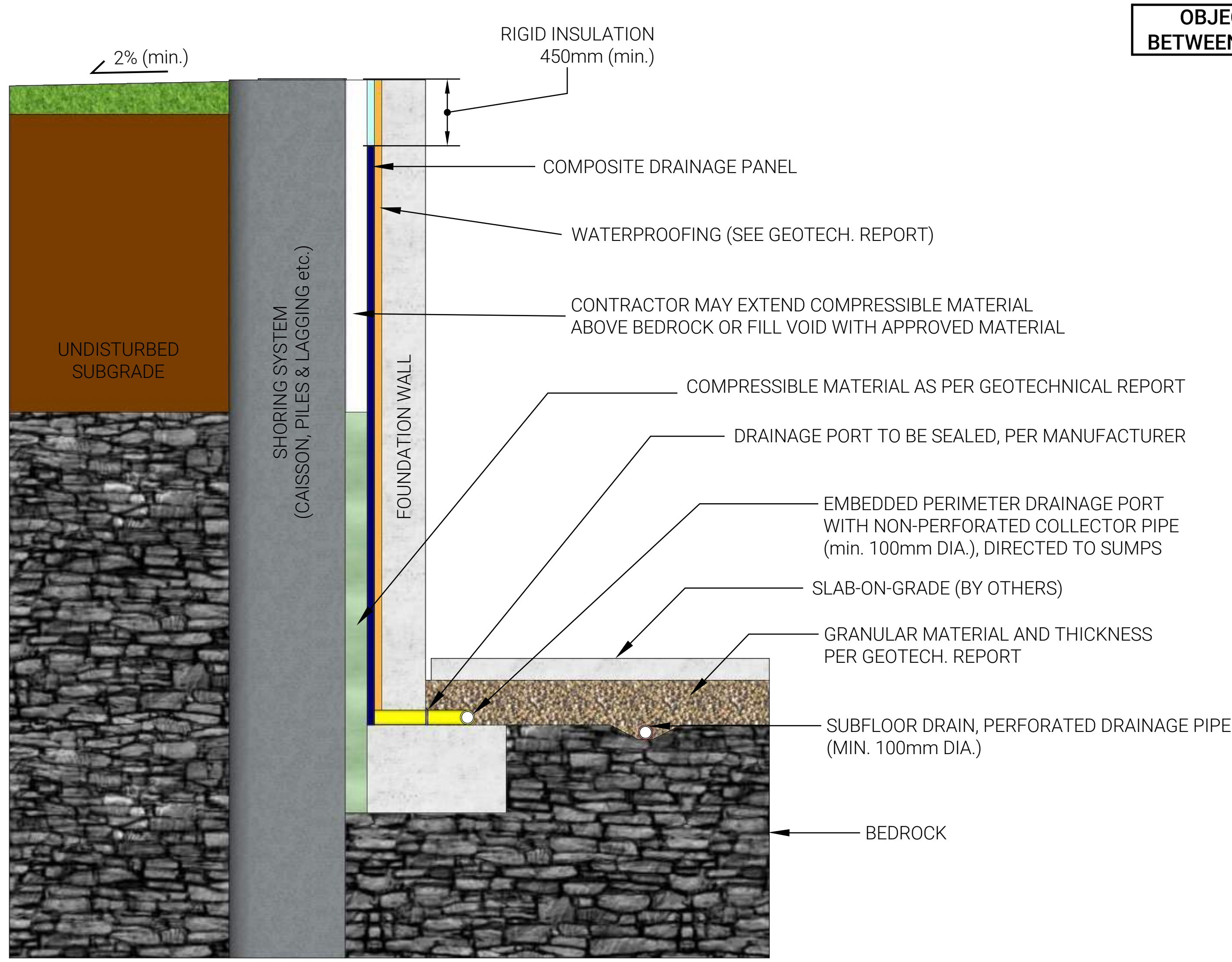
NOTES

1. A NON-WOVEN GEOTEXTILE WITH AN APPARENT OPENING SIZE OF < 0.250mm AND A TEAR RESISTANCE OF > 200 N.

GROUNDED ENGINEERING

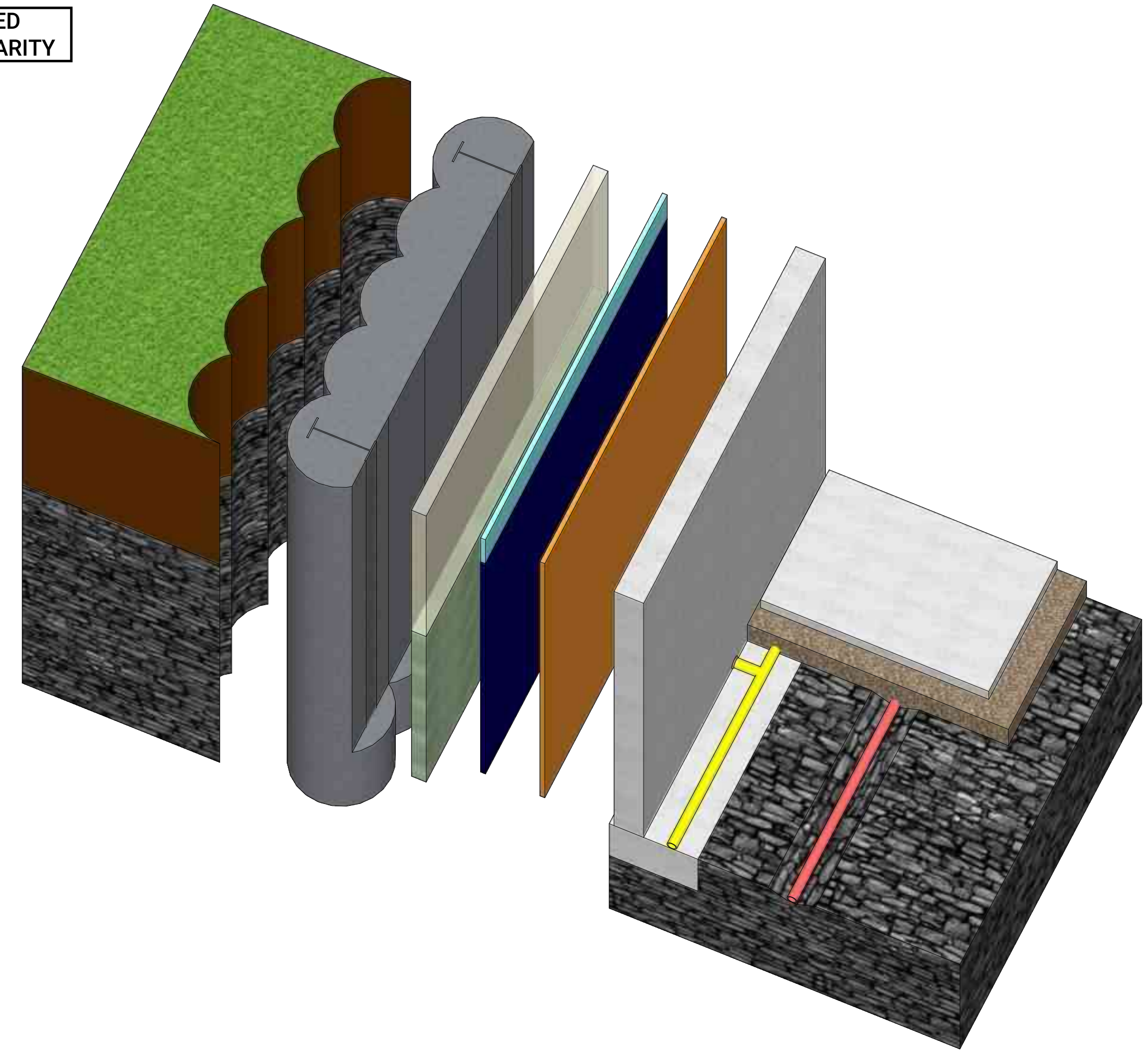
Title

BASEMENT DRAINAGE TYPICAL DETAIL



SECTIONAL VIEW

OBJECTS ARE COLOR-CODED BETWEEN TWO VIEWS FOR CLARITY



ISOMETERIC VIEW

SUBFLOOR DRAINAGE SYSTEM

1. THE SUBFLOOR DRAINS SHOULD BE SET IN PARALLEL ROWS, IN ONE DIRECTION, AND SPACED AS PER THE GEOTECHNICAL REPORT.
2. THE INVERT OF THE PIPES SHOULD BE A MINIMUM OF 300mm BELOW THE UNDERSIDE OF THE SLAB-ON-GRADE.
3. A CAPILLARY MOISTURE BARRIER (I.E. DRAINAGE LAYER) CONSISTING OF A MINIMUM 200 mm LAYER OF CLEAR STONE (OPSS MUNI 1004) COMPACTED TO A DENSE STATE (OR AS PER THE GEOTECHNICAL REPORT). WHERE VEHICULAR TRAFFIC IS REQUIRED, THE UPPER 50 mm OF THE CAPILLARY MOISTURE BARRIER MAY BE REPLACED WITH GRANULAR A (OPSS MUNI 1010) COMPACTED TO A MINIMUM 98% SPMDD.

PERIMETER DRAINAGE SYSTEM

1. FOR A DISTANCE OF 1.2m FROM THE BUILDING, THE GROUND SURFACE SHOULD HAVE A MINIMUM 2% GRADE.
2. PREFABRICATED COMPOSITE DRAINAGE PANEL (CONTINUOUS COVER, AS PER MANUFACTURER'S REQUIREMENTS) IS RECOMMENDED BETWEEN THE BASEMENT WALL AND RIGID SHORING WALL. THE DRAINAGE PANEL MAY CONSIST OF MIRADRAIN 6000 OR AN APPROVED EQUIVALENT.
3. PERIMETER DRAINAGE IS TO BE COLLECTED IN NON-PERFORATED PIPES AND CONVEYED DIRECTLY TO THE BUILDING SUMPS.
4. PERIMETER DRAINAGE PORTS SHOULD BE SPACED A MAXIMUM 3m ON-CENTRE. EACH PORT SHOULD HAVE A MINIMUM CROSS-SECTIONAL AREA OF 1500 mm2.

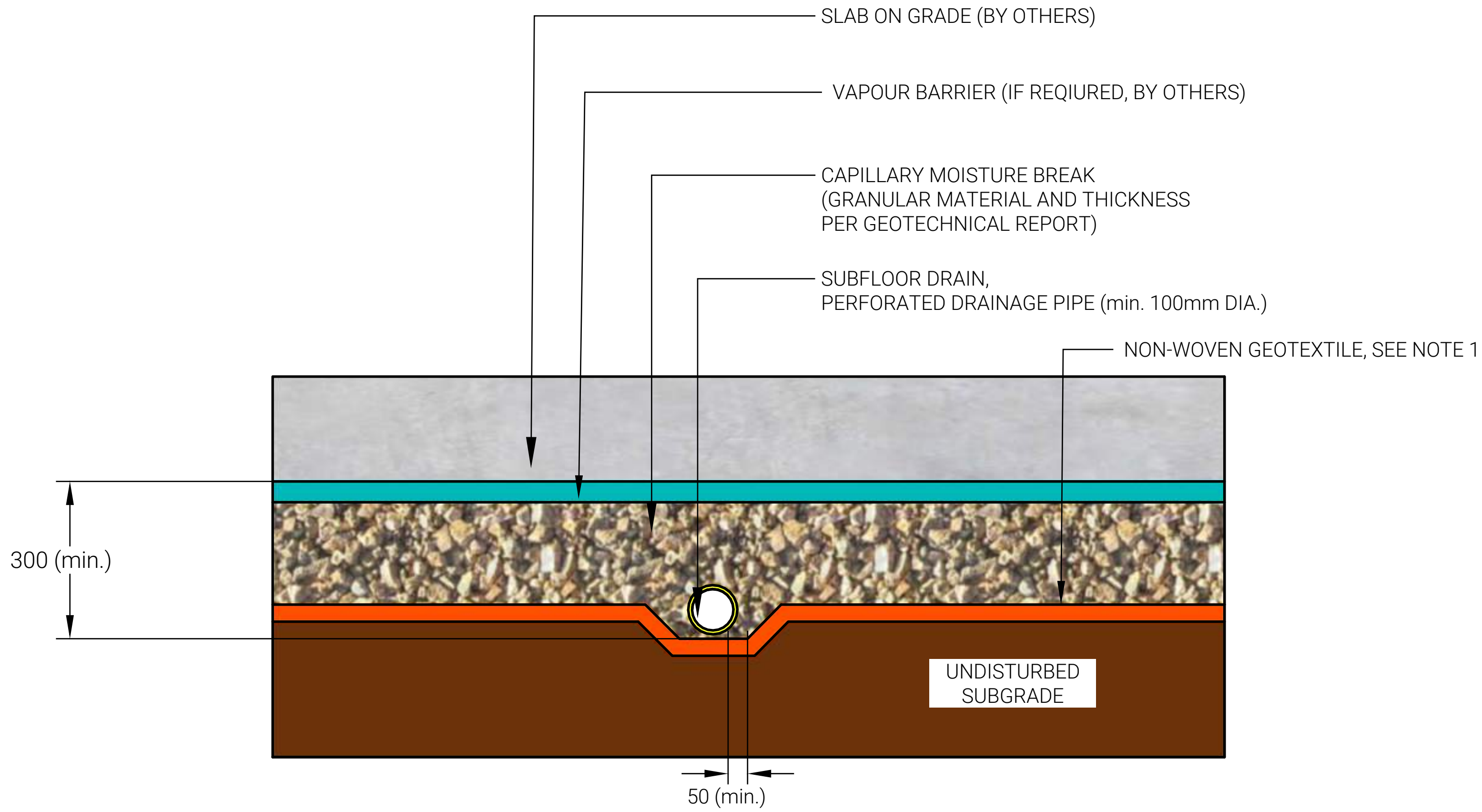
GENERAL NOTES

1. THERE SHOULD BE NO STRUCTURAL CONNECTION BETWEEN THE SLAB-ON-GRADE AND THE FOUNDATION WALL OR FOOTING.
2. THERE SHOULD BE NO CONNECTION BETWEEN THE SUBFLOOR AND PERIMETER DRAINAGE SYSTEMS.
3. THIS IS ONLY A TYPICAL BASEMENT DRAINAGE DETAIL. THE GEOTECHNICAL REPORT SHOULD BE CONSULTED FOR SITE SPECIFIC RECOMMENDATIONS.
4. THE FINAL BASEMENT DRAINAGE DESIGN SHOULD BE REVIEWED BY THE GEOTECHNICAL ENGINEER TO CONFIRM THE DESIGN IS ACCEPTABLE.

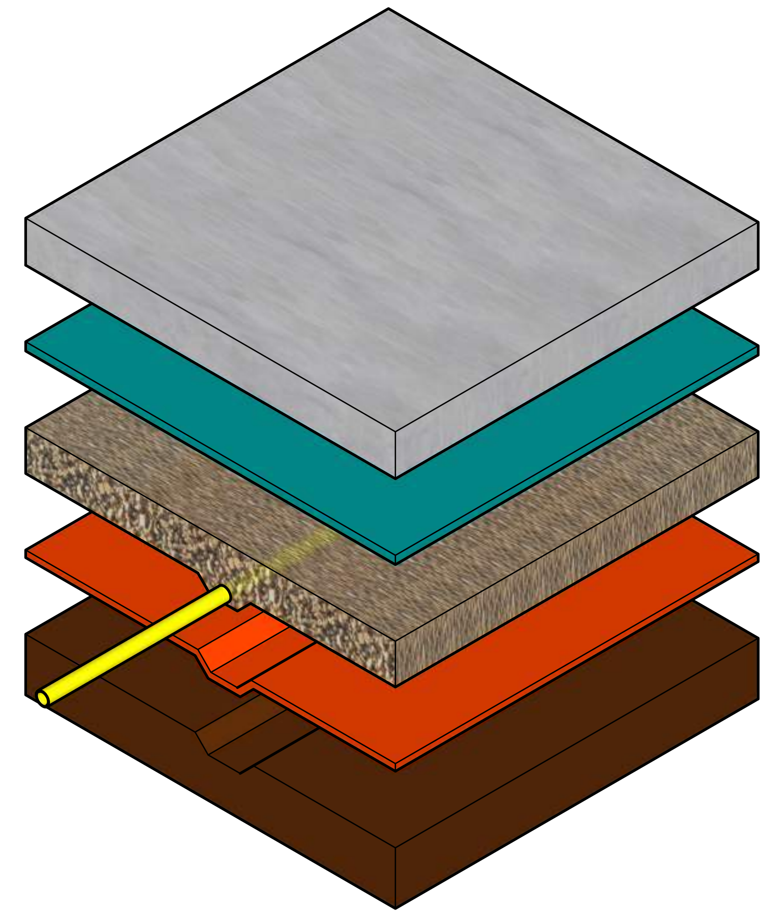
GROUNDED ENGINEERING

Title

FOUNDATION WALL BLINDSIDE DRAINAGE SYSTEM (IN DEEP ROCK) DETAIL



SECTIONAL VIEW



ISOMETRIC VIEW

NOTES

1. WHEN THE SUBGRADE CONSISTS OF COHESIONLESS SOIL, IT MUST BE SEPARATED FROM THE SUBFLOOR DRAINAGE LAYER USING A NON-WOVEN GEOTEXTILE (WITH AN APPARENT OPENING SIZE OF < 0.250mm AND A TEAR RESISTANCE OF > 200 N).

2. TYPICAL SCHEMATIC ONLY. MUST BE READ IN CONJUNCTION WITH GEOTECHNICAL REPORT.

GROUNDED ENGINEERING

Title

BASEMENT SUBDRAIN TYPICAL DETAIL