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PROJECT NO.: SM 241053-G June 6, 2025

MIURA DEVELOPMENT GROUP INC.
931 Southcote Road
Ancaster, Ontario

Attention: David Vrhovnik

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
AND HYDROGEOLOGICAL ASSESSMENT
PROPOSED RESIDENTIAL DEVELOPMENT
84 TO 104 PLAINS ROAD
BURLINGTON, ONTARIO

Dear Mr. Vrhovnik,

Further to your authorisation, SOIL-MAT ENGINEERS & CONSULTANTS LTD. has completed the fieldwork, laboratory testing and report preparation in connection with the above noted project. The scope of work was completed in general accordance with our proposal P241053, dated November 22, 2024, and revised on January 13, 2025. Our comments and recommendations, based on our findings at the three borehole locations, are presented herein.

1. INTRODUCTION

We understand that the project will involve the construction of a 12 storey residential buildings with two to three levels of basement parking levels and associated services at 84 to 104 Plains Road East in Burlington, Ontario. The purpose of this supplemental geotechnical and hydrogeological assessment work is to assess the subsurface soils and groundwater conditions, and to provide our comments and recommendations with respect to the design and construction of the proposed development, from a geotechnical point of view.

The previous geotechnical investigation report prepared by Terraprobe has been provided to this office [Geotechnical Investigation 92 Plains Road East, Burlington, Ontario, Terraprobe, Project No. 71-15-5012, dated September 7, 2016] and the Phase Two Environmental Stite Assessment provided by Rubidium Environmental [Phase Two Environmental Site Assessment 92 Plains Road East, Burlington, Ontario, Rubidium Environmental, Rb File #2671, Dated November 21, 2017]

This report is based on the above summarised project description, and on the assumption that the design and construction will be performed in accordance with applicable codes and standards. Any significant deviations from the proposed project design may void the recommendations given in this report. If significant changes are made to the proposed design, such as additional storeys or basement levels, this office must be consulted to review the new design with respect to the results of this investigation.

2. PROCEDURE

A total of three [3] sampled boreholes were advanced at the locations illustrated in the attached Drawing No. 1, Borehole Location Plan. The boreholes were advanced using continuous flight power auger equipment, as well as mud-rotary in Borehole Nos. 101 and 102, on January 28, 29 and February 6, 2025 under the direction of a representative of SOIL-MAT ENGINEERS to termination and/or practical auger/sampling spoon refusal on Queenston shale bedrock at depths between approximately 9.7 and 21.5 metres below the existing grade Two of the existing monitoring wells [MW2 and MW3] from prior investigation work by others were able to be located on the site, and utilised for this investigation.

Upon completion of drilling, a groundwater monitoring well was installed at one [1] of the borehole locations, noted to be Borehole No. 101. The monitoring well consists of 50-millimetre PVC pipe, screened in the lower 1.5 metres encased with well filter sand up to approximately 0.3 metres above the screened portion, then with bentonite 'hole plug' to the surface and fitted with a protective 'stick up' steel casing. The remaining boreholes were backfilled in general accordance with Ontario Regulation 903, and the ground surface was reinstated flush with the surrounding grade.

Representative samples of the subsoils were recovered from the borings at selected depth intervals using split barrel sampling equipment driven in accordance with the requirements of the ASTM test specification D1586, Standard Penetration Resistance Testing. After undergoing a general field examination, the soil samples were preserved and transported to the SOIL-MAT laboratory for visual, tactile, and olfactory classifications.

The boreholes were located on site by a representative of SOIL-MAT ENGINEERS & CONSULTANTS LTD., based on accessibility over the site and clearance of underground services. The ground surface elevation at the borehole locations was referenced to a site-specific geodetic benchmark, described as the top of cross cut in the sidewalk as

shown in Drawing No. 1. The benchmark has a known elevation of 101.044 metres as described in the topographic survey provided to this office [Topographic Survey of Part of Lots 34, 35, and 36 Registered Plan 725PF in the City of Burlington, A.T. McClaren Limited, Drawing No. 34581, Dated March 26, 2015]. It is noted that the ground surface elevations of the Phase Two ESA borehole logs are not in agreement with the site survey and as such the elevations of the previously existing monitoring wells have been surveyed to the above noted benchmark.

Details of the conditions encountered in the boreholes, together with the results of the field and laboratory tests, are presented in Log of Borehole Nos. 101 to 103, inclusive, following the text of this report. It is noted that the boundaries of soil types indicated on the borehole logs are inferred from non-continuous soil sampling and observations made during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and therefore should not be construed as the exact depths of geological change.

3. SITE DESCRIPTION AND SUBSURFACE CONDITIONS

The subject site is currently occupied by residential properties from 84 to 104 Plains Road East in Burlington, Ontario. It is noted the former single-family dwelling on the 92 Plains Road East Lot has been demolished. Assuming a east-west orientation of Plains Road East, the site is bounded to the east by Glenwood Avenue, to the north by Plains Road East, to the west by Birchwood Avenue, and to the south by residential properties. The site has relatively flat and even topography with the surrounding roadways.

The subsurface conditions encountered at the borehole locations are summarised as follows:

Topsoil

A surficial layer of topsoil approximately 200 millimetres in thickness was encountered at Borehole Nos. 1 and 3. It is noted that the depth of topsoil may vary across the sites and from the borehole locations, and as such a conservative approach should be taken in estimating topsoil quantities across the site. It is also noted that the term ‘topsoil’ has been taken from a geotechnical point of view and does not necessarily reflect its nutrient content or its ability to support plant life.

Silty Sand Fill

A surficial layer of and gravel fill was encountered in Borehole No. 2. The silty sand fill was brown in colour and contained trace to some amounts of grave increasing with depth with moderate construction debris encountered within the upper levels. The fill soils were noted to be in a loose to compact state as was proven to a depth of approximately 3.0 metres below the existing ground surface.

Silty Sand

Native silty sand was encountered beneath the topsoil and surface fill deposit, and below sand and gravel layer at depth, at all borehole locations. The native sand was brown in colour containing trace amounts of gravel and clay and was generally in a loose to dense state. The native silty sand soils were proven to termination at depths of approximately 9.8 to 12.8 metres below the existing ground surface in borehole Nos 1 and 3 and to a depth of approximately 10.8 metres in Borehole No. 2.

Sand and Gravel

A sand and gravel deposit was encountered beneath the native silty sand at all borehole locations at depths of approximately 3 to 4.1 metres. The granular material was brown to grey in colour, with varying gravel content throughout and was generally found to be in a very dense state. This presents as a partially cemented gravel or beach deposit associated with former Lake Iroquois. The sand and gravel deposit was proven to depths of approximately 7.2 to 8.7 metres below the existing ground surface.

Silty Clay/Clayey Silt

Native silty clay/clayey silt was encountered beneath the native silty sand at Borehole No 2 at a depth of approximately 11.0 metres. The silty clay/clayey silt was grey in colour, occasionally contained trace to some gravel, and was generally in a very stiff to hard consistency. The fine-grained cohesive soil was proven to a depth of approximately 20 metres below the existing ground surface in Borehole Nos. 2 upon Queenston shale bedrock.

Queenston Shale

Queenston Shale bedrock was encountered beneath the silty clay/clayey silt in Borehole No. 2 at a depth of approximately 20 metres. The shale was reddish brown in colour, severely weathered in the upper levels, becoming sounder with depth, and containing occasional more resistant grey layers. The degree of weathering in the upper levels of the Queenston Shale is such that it tends to exhibit the properties of a hard cohesive soil, or weak rock, making the transition from the overlying hard silty clay somewhat indistinct.

A review of available published information [Quaternary Geology of Ontario, Southern Sheet Map 2556] indicates the surficial geology of the subject site to consist of coarse textured glaciolacustrine deposits of sand and gravel with minor silty and clay, consistent with our experience in the area and observations in the upper levels during fieldwork.

Groundwater Observations

As mentioned above, in addition to two [2] existing groundwater monitoring wells, one [1] additional borehole was fitted with a monitoring well to allow for measurement of the static groundwater level. The installation details for each of the monitoring wells is summarised as follows:

Table A – Monitoring Well Details

Well No.	Surface Elev. [m]	Depth [m]	Screen Length [m]	Screen Interval [m]
101	100.78	12.2	1.5	10.7 – 12.2
2	101.16	9.2	3.1	6.1 – 9.2
3	100.96	9.2	3.1	6.1 – 9.2

Note that MW2 and MW3 were installed by others, details are per the supplied report.

The groundwater level was measured at the wells by SOIL-MAT ENGINEERS with the following results:

Table B - Ground Water Level Measurements

Well No.	Ground Surface Elev. [m]	1/8/2025		3/10/2025		3/27/2025	
		Ground Water Depth (m)	Ground Water Elev. (m)	Ground Water Depth (m)	Ground Water Elev. (m)	Ground Water Depth (m)	Ground Water Elev. (m)
1	100.78	-	-	8.44	92.34	8.38	92.40
2	101.16	8.45	92.71	8.74	92.42	8.50	92.66
3	100.96	5.42	95.54	8.65	92.31	8.50	92.46

The prior Phase Two ESA report by others noted groundwater measured at depths of 8.30 to 8.43 metres on October 27, 2017. This is consistent with the current readings, and provides a reasonable indication to the seasonal high. Based on our observations during drilling, the available data, and our experience in the area, the seasonal high groundwater level is conservatively estimated at an elevation of roughly 92.7 metres. It is recommended that, in order to avoid long-term interaction with groundwater, the lowest basement floor level be a minimum of 0.5 metres above the estimated high groundwater level, or 93.2 metres.

It is noted that the groundwater level would be subject to seasonal variation due to prevailing weather conditions and long-term groundwater measuring data collection loggers have been installed to record the spring high ground water level.

4. GEOTECHNICAL LABORATORY TESTING

As noted above, three [3] selected samples of the recovered soil were subjected to grain size analyses including sieve and hydrometer tests. The results of these grain size analyses have been summarised in Table C as follows:

Table C- Grain Size Analyses Summary

Sample	Sample Depth [m]	Clay [%]	Silt [%]	Sand [%]	Gravel [%]	Estimated Infiltration Rate [mm/hr]	Estimated Permeability [cm/sec]
BH1 SS5	3.0	9	10	80	1	50 to 60	10^{-4}
BH1 SS7	6.1	3	5	34	58	150 to 200	10^{-2}
BH3 SS8	7.6	5	12	70	13	75 to 100	10^{-4}

The grain size analyses demonstrate the sand and gravel deposit present between about 3 to 9 metres depth to be classified as G.P. – Gravel-sand mixtures, to G.M. – Gravel-sand-silt mixtures. This is a relatively permeable granular deposit, with permeability estimated on the order of 10^{-2} cm/sec. However, it is noted that the cemented nature of the deposit may tend to reduce the overall permeability.

The overburden and underlying silty sand is classified as S.P. – poorly graded sand to S.M – Sand-silt mixtures and G.M. – Gravel-sand-silt mixtures. The sand soils are moderately permeable with an estimated permeability in the range of 10^{-3} to 10^{-4} cm/sec.

5. FOUNDATION CONSIDERATIONS

With two to three levels of underground parking, it is anticipated that the proposed founding depth will be on the order of 6 to 10 metres below the finished grade. Preliminary plans noted a proposed basement floor slab elevation of 92.82 metres. As noted above, it is recommended that the basement slab be raised to a minimum of 93.2 metres, in order to avoid long-term interaction with groundwater.

This will result in a founding level extending through the cemented sand and gravel deposit and into the compact to dense silty sand soils. Conventional spread footings founded within the native silty sand soils may be designed considering a bearing capacity of 250 kPa [~5,000 psf] SLS and 400 kPa [~8,000 psf] ULS. Where these bearing capacities require greater than 50 per cent of the building footprint to support the proposed building loads, the use of a raft foundation would be recommended. A raft foundation bearing on the compact to dense silty sand, and stress influence into the underlying very stiff silty clay, may be designed considering a bearing capacity of 200 kPa [~4,000 psf] SLS and 300 kPa [~6,000 psf] ULS. Alternatively, a flexible design method may be used, considering a modulus of subgrade reaction of k = 30 MN/m3.

Where higher foundation capacities are required, the use of intermediate or deep foundation options could be considered. These could include ground improvement techniques to increase the available bearing for spread or raft footings, or deep foundations, such as drive piles, micropiles or caissons, extending to the Queenston Shale. For the scale of structure, it is not anticipated that such higher loadings will be required. Should this become necessary, our office should be contacted to provide more detailed recommendations.

It is noted that the SLS value represents the Serviceability Limit State, which is governed by the tolerable deflection [settlement] based on the proposed building type, using unfactored load combinations. The ULS value represents the Ultimate Limit State and is intended to reflect an upper limit of the available bearing capacity of the founding soils in terms of geotechnical design, using factored load combinations. There is no direct relationship between ULS and SLS; rather they are a function of the soil type and the tolerable deflections for serviceability, respectively. Evidently, the baring capacity would be lower for very settlement sensitive structures and larger for more flexible buildings.

In areas where it will be necessary to provide adjacent footing at different founding elevations, the lower footings should be constructed before the higher footing is constructed, if possible, and the higher footing should be set below and imaginary line drawn up from the lower edge of the footing at 10 horizontal to 7 vertical. This practice will aid in limiting stress transfer from the higher footings to the lower footings.
All footings exposed to the environment must be provided with a minimum of 1.2 metres of earth cover of equivalent insulation to protect against frost damage. This frost protection would also be required if construction were undertaken during the winter months. It is noted that while the basement levels are anticipated to be unheated parking garage spaces, such spaced do not typically experience freezing air temperatures, especially considering 2 to 3 basement levels are proposed. As such interior footings within the parking garage would not require a full 1.2 metres of cover below the basement floor level. It is recommended that a minimum cover of 0.6 metres would be adequate. The noted exception would be at the location of penetrations, such as air vent shafts, that allow for more direct entry of outside air temperatures. At these locations it is recommended that a layer of 50-millimetre-thick rigid insulation be provided as supplemental protection of footings within perhaps 4 metres of any openings.

With foundations designed as outlined above and as required by the Building Code, in accordance with the structural design drawings and details, and with careful attention paid to construction detail, total and differential settlements should be well within normally tolerated limits of 25 and 30 millimetres respectively. However as is typical with most new construction, ‘cosmetic’ cracking of the foundation walls, etc., may occur within the first year of construction as a result of shrinkage, minor settlement, etc. Subsequent to repair, additional cracking should be minimal.

It is imperative that a soils engineer be retained from this office to provide geotechnical engineering services during excavation and foundation construction phases of the project. This is to observe compliance with the design concepts and recommendations of this report and to allow changes to be made in the event that subsurface conditions differ from the conditions identified at the borehole locations.

6. LATERAL EARTH PRESSURE

The lateral earth pressures on basement walls and excavation support systems can be estimated on the basis of the following soil parameters. Any additional pressures due to surcharge loading, such as adjacent structures, roadways, parked vehicles, floor slab loading, etc. must be included in the design. Where foundations are constructed as a 'waterproofed' section, they must also be designed to support the vertical and horizontal hydrostatic pressures of a conservative groundwater level at ground surface.

Table D – Lateral Earth Pressure Parameters

Soil Strata	Soil Unit Weight, γ [kN/m^3]	Angle of Internal Friction, θ [^{o}]	Coefficient of Active Earth Pressure, K_a	Coefficient of At Rest Earth Pressure, K_o	Coefficient of Passive Earth Pressure, K_p
Sand	20	30	0.33	0.5	3.0
Sand and Gravel	21	35	0.27	0.43	3.69

7. SEISMIC DESIGN CONSIDERATIONS

The structure shall be designed according to Section 4.1.8 of the Ontario Building Code, Ontario Regulation 203/24. Based on the subsurface soil conditions encountered in this investigation the applicable Site Classification for the seismic design is Site Class D – Stiff Soil, based on the average soil characteristics for the site. It is noted that a seismic site class of C may be available, however would need to be confirmed via site specific shear wave velocity testing.

The seismic data, from the Ontario Building Code, for 84-14 Plains Road East in Burlington Ontario are as follows.

S_a[0.2]	S_a[0.5]	S_a[1.0]	S_a[2.0]	S_a[5.0]	S_a[10.0]	PGA	PGV
0.393	0.319	0.178	0.082	0.0211	0.0065	0.248	0.207

8. EXCAVATIONS AND EXCAVATION SUPPORT CONSIDERATIONS

Excavations for the proposed development will result in excavations depths on the order of 7 to 10 metres below the existing ground surface, extending through the fill materials, silty sand, sand and gravel soils to the native silty sand soils. Excavations through the fill and silty sand soils would be expected to remain stable at inclinations of up to 45 degrees to the horizontal to for the short construction period. Excavations through the cemented sand and gravel deposit will present greater difficulty, requiring heavy hydraulic excavator equipment with rock teeth and possibly mechanical rock splitter in some levels. Excavations in the cemented sand will remain stable at inclinations of 45 degrees, and possible steeper, subject to detailed review and evaluation by a senior representative of this office. Where wet seams are encountered, during periods of extended precipitation, and where the excavations extend near the static groundwater level, the excavations through any fill materials and native soils may have a tendency to 'slough in' to as flat as 3 horizontal to 1 vertical, or flatter.

Notwithstanding the forgoing, all excavations must comply with the current Occupational Health and Safety Act and Regulations of Construction Projects. Excavation slopes steeper than those required in the safety act must be supported or a trench box must be provided, and a senior geotechnical engineer from this office should monitor the work. With respect to the Act, the majority of the soils would be considered as Type 3 soil with the dense to very dense sand and gravel considered as Type 2 soil.

Given the proximity of the excavations for the proposed development to adjacent structures, roadways, property limits, etc., it is anticipated that an excavation shoring system will need to be provided to ensure the support conditions for the adjacent structures are not compromised. The use of driven steel sheeting, soldier pile and timber lagging, or a caisson wall into the native sand soils could be considered. Both steel sheeting and the caisson wall will provide the added benefit of restricting the infiltration of groundwater, in the event that the basement level is proposed to extend below the groundwater level.

A specialty contractor or shoring consultant should be consulted with respect to the design of such a shoring system, where required. For preliminary design purposes the shoring system should be designed on the basis of a retained soil with the properties outlined in Tabe D – Lateral Earth Pressure Parameters.

Where required, the shoring can be laterally supported either by anchors extending through the overburden soil into the native soils (where allowed by adjacent property owners/possible with adjacent structures) or by rakers extending into the excavations. Racker footings on the native silty sand soils may be designed on the basis of the recommended bearing values noted above. From a contractor's point of view, tie-back

anchors would be preferred, provided they can be installed to avoid adjacent foundations and underground utilities and permission can be obtained from adjacent property owners. It is noted that significant movements of the shoring system may take place if conventional rakers are used to support the shoring systems since compression of the members and the supporting footings must occur before the rakers can begin to carry load. In this regard, anchors are preferred since they allow pre-stressing of the shoring system to the design load even before the excavations reach their final grade. Alternatively, the rakers can be designed to allow jacking in the design load, and thus eliminating/minimizing movements.

The shoring system should be monitored during construction, and the contractor should have a contingency plan in place to be implemented should deflections of the shoring system exceed the tolerable limits. In addition, it is imperative that a pre-construction condition survey be conducted of the adjacent structures, roadways, etc. in order to document the existing conditions prior to the commencement of construction. This will allow for comparison and assessment in the event that disturbance due to construction activities is claimed.

As noted above, the seasonal high groundwater level is estimated at an elevation of 92.7 metres approximately 8.5 metres below the existing ground surface. With the anticipated excavations depths for foundation construction extending to or slightly below the groundwater, some groundwater infiltration is anticipated. Additionally, some infiltration from surface run off or through perched water in more permeable seams should be expected. The rate of infiltration should be such that conventional construction dewatering methods such as pumping from sumps, may be sufficient to control such infiltration, though where excavations extend below the ground water level additional dewatering efforts should be expected. Excavations completed in the dryer summer months may reduce the need for extensive groundwater control schemes. Regardless, given the size of the proposed excavation, groundwater infiltration of greater then 50,000 L/day may be experienced, and an Environmental Activity Sector Registry [EASR] or even a Permit to Take Water [PTTW] may be required. Pending more detailed design of the strucutre, a more detailed groundwater studies including hydrogeological dewatering assessments, etc., would be warranted.

The base of the excavation into the native silty sand should remain firm and stable for the proposed works, however may be prone to localised disturbance based on the proximity of the static groundwater level, as well from construction traffic. As such it is recommended that the founding soils be provided with a lean-mix [~5 MPa] concrete ‘mud slab’ be placed upon the completion and evaluation of the founding soils. This will act to protect the founding soils from disturbance, as well as provide a clean and stable

working surface for construction of formwork and placement reinforcing steel. Alternatively, the excavation base may be provided with a layer of granular material such, as Ontario Provincial Standard Specification [OPSS] Granular 'B', Type II (crushed limestone bedrock), perhaps 200 to 300 millimetres thick and compacted to 100 per cent of its standard Proctor maximum dry density [SPMDD] to provide a stable 'clean' working surface.

9. FLOOR SLAB AND PERIMETER DRAINAGE/WATERPROOFING CONSIDERATIONS

As noted above, it is recommended that the lower basement slab level be at an elevation of 93.2 metres, or above. This will provide sufficient separation from the seasonal high groundwater level such that long-term interaction and dewatering of groundwater will not be required. In the event that the basement level is proposed to extended greater depths, below the groundwater level, it would be recommended for the foundations and foundation walls to be designed and constructed as water-tight, to eliminate the need for ongoing dewatering of groundwater.

Depending on the final design, it is anticipated that the basement floor slab will be constructed either using conventional slab-on-grade techniques on a prepared subgrade, or on top of a raft slab foundation. As with all concrete floor slabs, there is a tendency for the slab to develop cracks. The slab thickness, concrete mix design, the amount of steel and/or fibre reinforcement and/or wire mesh placed into the concrete slab, if any, will therefore be a function of the owner's tolerance for cracks in, and movements of, the slabs-on-grade, etc. The 'saw-cuts' in the concrete floors, for crack control, should extend to a minimum depth of 1/3 of the thickness of the slab.

A moisture barrier will be required under the floor slabs such as the placement of at least 200 millimetres of well-compacted 20-millimetre clear crushed stone. At a minimum the moisture barrier material should contain no more than 10 per cent passing the No. 4 sieve. Where a 'non-damp' floor slab is required, as for instance under sheet vinyl floor coverings, etc., extra efforts will be required to damp proof the floor slab, as with the additional provision of a heavy 'poly' sheet, damp proofing sprays/membranes, drainage board products, etc. Where 'poly' sheets are used care should be taken to prevent puncturing and tearing and/or sufficiently heavy gauge sheeting specified. Alternatively, a proprietary product such as Delta-MS Underslab or WR Meadows membrane may be considered in lieu of the 'poly' sheets.

Curing of the slab must be carefully specified to ensure that slab curl is minimised. This is especially critical during the hot summer months of the year when the surface of the slab tends to dry out quickly while high moisture conditions in the moisture barrier or water trapped on top of any 'poly' sheet at the saw cut joints and cracks, and at the edges of the slabs, maintains the underside of the slab in a moist condition.

It is important that the concrete mix design provide a limiting water/cement ratio and total cement content, which will mitigate moisture related problems with low permeance floor coverings, such as debonding of vinyl and ceramic tile. It is equally important that excess free water not be added to the concrete during its placement as this could increase the potential for shrinkage cracking and curling of the slab.

To address the potential for build up of groundwater beneath the basement floor slabs, in addition to the under-slab damp proofing measures recommended above, underfloor drains should be provided. Underfloor drains may consist of 150-milimetre diameter perforated pipe, with a geofabric sock, placed in the clear stone beneath the floor slabs on nominal 4 to 6 metre centers. If is noted that the underfloor and perimeter drainage systems have separate piping, i.e., piping from the perimeter system does not connect to the under-floor system, in order to prevent surcharging of the underfloor system. They may outlet into a common sump-pit, though separate systems would be preferred. The enclosed drawing No. 2 shows schematics of requirements for foundation construction with an underfloor drain system.

This office should examine the installation of the perimeter and underfloor drains. Even a small break in the filtering materials could result in loss of fines into the drains with attendant performance difficulties, including settlements of the ground surface. The exterior grade around the structure should be sloped away from the structure to prevent the ponding of water against the foundation walls. Elevator pits should also be designed to be water-tight. It is recommended that this water-tight design for the elevator pit be implemented regardless of the water proofing or drainage design adopted for the general basement level.

10. BACKFILL CONSIDERATIONS

It is expected that the majority of the material excavated to accommodate the basement levels will be transported off-site. Where required, select portions of the fill material, sand and gravel, and silty sand are considered suitable for use as backfill in service trenches, etc. provided it is free of organics or any other deleterious material. The native on-site soils are generally moderately permeable and would be considered to be reasonably free draining, and may be effective for use as backfill in areas of restricted access. This should be reviewed on site at the time of construction by a representative of our office.

The use of free draining, well-graded granular material, such as an Ontario Provincial Standard Specification [OPSS] Granular B, Type II (crushed limestone bedrock) is often preferred for backfill against foundation walls. This material is more readily compacted in restricted access areas, and generally presents a more positive support condition for exterior concrete sidewalks, pavement, etc. The use of alternative materials, including recycled concrete aggregates, clear stone materials, etc., may be considered, however should be reviewed and approved by our office prior to use. Any imported fill required in service trenches or to raise the subgrade elevation should have its moisture content within 3 per cent of its optimum moisture content and meet the necessary environmental guidelines.

Fine-grained granular practically impossible to compact using conventional compaction equipment if they become wet during the 'wet' periods of the year. After a period of heavy precipitation, any near-surface wet, or softened material, should be allowed to air-dry or be removed and discarded. These soils are may considered to be free draining however should not be used where this characteristic is necessary without further testing. They should not be used in confined spaces where access for compaction is difficult.

Backfill materials used in service trenches or to raise the subgrade elevation under exterior pavement areas should have its moisture content within 3 per cent of its optimum moisture content. In service trenches the backfill material should be compacted to a minimum of 98 per cent of its standard Proctor maximum dry density [SPMDD]. Any fill required against the basement walls should consist of free draining granular material compacted to 98 per cent of its standard Proctor maximum dry density. Where asphalt or concrete paving extends up to the building the upper 1 metre of foundation wall backfill should be compacted to 100 per cent of its standard Proctor maximum dry density. Lift thickness would depend on the material type and moisture content of the backfill material and size of the compaction equipment used by the contractor. A representative of SOIL-MAT should be retained to monitor the backfilling and compaction operations to confirm uniform compaction of the backfill material to project specification requirements.

11. PAVEMENT STRUCTURE DESIGN CONSIDERATIONS

All areas to be paved must be cleared of all organic and otherwise unsuitable materials, and the exposed subgrade proof rolled with 3 to 4 passes of a fully-loaded tandem-axle truck in the presence of a representative of SOIL-MAT ENGINEERS & CONSULTANTS LTD., immediately prior to the placement of the sub-base material. Any areas of distress revealed by this or other means should be sub-excavated and replaced with suitable backfill material. Where the subgrade condition is poorer it may be necessary to implement more aggressive stabilisation methods, such as the use of coarse aggregate

[50mm clear stone, 'rip rap' stone, etc.] 'punched' into the soft areas. It may also be prudent to consider the provision of a heavy geofabric over the subgrade to act as a separator between the subgrade and granular base materials.

The need for sub-excavations of softened subgrade materials will be reduced if construction is undertaken during dry periods of the year and careful attention is paid to the compaction operations. As noted above the on-site soils are sensitive to disturbance and moisture and may present difficulty for roadway construction during 'wet' periods of the year. Should pavement construction be undertaken during 'wet' periods of the year it should be anticipated that greater stabilisation efforts will be required and/or additional depth of OPSS Granular 'B', Type II sub-base course material may be required.

Good drainage provisions will optimise the long-term performance of the pavement structure. The subgrade must be properly crowned and shaped to promote drainage to the subdrain system. Subdrains should be installed to intercept excess subsurface water and to prevent softening of the subgrade material. Surface water should not be allowed to pond adjacent to the outer limits of the paved areas.

The most severe loading conditions on the subgrade typically occur during the course of construction, therefore precautionary measures may have to be taken to ensure that the subgrade is not unduly disturbed by construction traffic. SOIL-MAT should be given the opportunity to review the final pavement structure design and subdrain scheme prior to construction to ensure that they are consistent with the recommendations of this report.

The suggested pavement structures outlined in Table E are based on subgrade parameters estimated on the basis of visual and tactile examinations of the on-site soils and past experience. The outlined pavement structure may be expected to have an approximate fifteen to twenty-year life, assuming that regular maintenance is performed. Should a more detailed pavement structure design be required, site specific traffic information would be needed, together with detailed laboratory testing of the subgrade soils.

Table E - Recommended Pavement Structures

LAYER DESCRIPTION	COMPACTION REQUIREMENTS	LIGHT DUTY SECTIONS	HEAVY DUTY [TRUCK ROUTE]
Asphaltic Concrete Wearing course OPSS HL 3 or HL 3A	92 per cent Marshall MRD	40 millimetres	40 millimetres
Binder Course OPSS HL 8	92 per cent Marshall MRD	50 millimetres	80 millimetres
Base Course OPSS Granular 'A'	100% SPMDD	150 millimetres	150 millimetres
Sub-base Course OPSS Granular 'B' Type II	100% SPMDD	300 millimetres	450 millimetres

* Marshall MRD denotes Maximum Relative Density.

* SPMDD denotes Standard Proctor Maximum Dry Density, ASTM-D698.

Depending on the anticipated traffic, a reduced light duty asphalt structure consisting of 65 millimetres of HL3 surface course may also perform sufficiently. This would be reasonable in areas subjected only to light vehicles such as cars for parking. Such a structure may have a reduced lifespan if subjected to heavier vehicles, and would also not allow for 'mill and pave' type operations for future rehabilitation.

Where asphalt pavement is to be constructed above the roof deck of the below grade parking level, the granular base layers recommended for the light duty pavement structure recommended above may be considered for both light duty and heavy-duty areas. It is noted that in such cases the roof deck slab should be sufficiently sloped and/or provided with suitable subdrains, in order to promote rapid drainage of water from beneath the pavement. As well the roof slab should be provided with a suitable water proofing system.

To minimise segregation of the finished asphalt mat, the asphalt temperature must be maintained uniform throughout the mat during placement and compaction. All too often, significant temperature gradients exist in the delivered and placed asphalt with the cooler portions of the mat resisting compaction and presenting a honeycomb surface. As the spreader moves forward, a responsible member of the paving crew should monitor the pavement surface, to ensure a smooth uniform surface. The contractor can mitigate the surface segregation by 'back-casting' or scattering shovels of the full mix material over the segregated areas and raking out the coarse particles during compaction operations. Of course, the above assumes that the asphalt mix is sufficiently hot to allow the 'back-casting' to be performed.

12. GENERAL COMMENTS

The comments provided in this document are intended only for the guidance of the design team. The material in it reflects SOIL-MAT ENGINEERS' best judgement in light of the information available at the time of preparation. The subsurface descriptions and borehole information are intended to describe conditions at the borehole locations only. It is the contractors' responsibility to determine how these conditions will affect the scheduling and methods of construction for the project. 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. SOIL-MAT ENGINEERS 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 trust that this geotechnical report is sufficient for your present requirements. Should you require any additional information or clarification as to the contents of this document, please do not hesitate to contact the undersigned.

Yours very truly,
SOIL-MAT ENGINEERS & CONSULTANTS LTD.



Kevin Reid, B. Eng., EIT
Project Manager



Ian Shaw, P. Eng., QP_{ESA}
Senior Engineer



Enclosures: Drawing No. 1, Borehole Location Plan
Log of Borehole Nos. 1 to 3, inclusive
Grain Size Analyses
Drawing No. 2, Basement with Perimeter Drainage and Underfloor Drains

Distribution: Miura Development Group Inc. [PDF]



LEGEND

Borehole and Monitoring Well Location
BH/MW #

Existing Monitoring Well Location

NOTES

1. This drawing should be read in conjunction with Soil-Mat Engineers & Consultants Ltd. Report No. SM 241053-G.

2. Borehole locations are approximate.

SOIL-MAT

ENGINEERS & CONSULTANTS LTD.

Geotechnical Investigation
Proposed Residential Development 84 to 104 Plains Road East
Burlington, Ontario

Proposed Borehole and Monitoring Well Location Plan

Project No. SM 241053-G

Date: January, 2025

Drawn: KJR

Checked: IS

Drawing No. 1

Log of Borehole No. 101

Project No: SM 241053-G
Project: Proposed Residential Development
Location: 84-104 Planins Road, Burlington
Client: Miura Development Group Inc.

Project Manager: Ian Shaw, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4795755
E: 593348

Depth (ft)	Depth (m)	Elevation (m)	Description	Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)
0	0	100.78	Ground Surface						
0	0	100.58	**Topsoil** Approximately 200 millimetres of topsoil.						
1			**Silty Sand** Brown,trace gravel, loose.	SS	1	16,12,5,4	17		
2									
3	1			SS	2	3,3,4,4	7		
4									
5				SS	3	2,2,1,2	3		
6	2								
7				SS	4	1,1,2,1	3		
8									
9				SS	5	1,2,1,1	3		
10	3								
11									
12									
13	4	96.60	**Sand and Gravel** Brown, Hard.						
14									
15				SS	6	50/5"	100		
16	5								
17									
18									
19									
20	6			SS	7	35,50/4"	100		
21									
22									
23	7								
24									
25				SS	8	5,40,50/2"	100		
26	8								
27									
28		92.00							
29									

Moisture Content w% (10 20 30 40)
Standard Penetration Test blows/300mm (20 40 60 80)

Drill Method: Solid Stem Auger
Drill Date: January 29,2025
Hole Size: 150 Millimetres
Drilling Contractor: Terra Firma

Soil-Mat Engineers & Consultants Ltd.
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www.soil-mat.ca E: info@soil-mat.ca

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Sheet: 1 of 2

Log of Borehole No. 101

Project No: SM 241053-G
Project: Proposed Residential Development
Location: 84-104 Planins Road, Burlington
Client: Miura Development Group Inc.

Project Manager: Ian Shaw, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4795755
E: 593348

Depth	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	SAMPLE Number	SAMPLE Blow Counts	SAMPLE Blows/300mm	SAMPLE Recovery	PP (kgf/cm2)
0			**Silty Sand** Brown, trace gravel, dense		SS	9	7,14,15,15	29		
					SS	10	15,18,13,13	31		
					SS	11	12,14,14,15	28		
	88.00		End of Borehole							

Moisture Content w% (10 20 30 40)
Standard Penetration Test blows/300mm (20 40 60 80)

NOTES:

1. Borehole was advanced using solid stem auger equipment on January 29, 2025 to termination at a depth of 12.8 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Auger
Drill Date: January 29,2025
Hole Size: 150 Millimetres
Drilling Contractor: Terra Firma

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Log of Borehole No. 102

Project No: SM 241053-G
Project: Proposed Residential Development
Location: 84-104 Planins Road, Burlington
Client: Miura Development Group Inc.

Project Manager: Ian Shaw, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4795749
E: 593315

Depth (ft / m)	Elevation (m)	Description	Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)
0 / 0	101.35	Ground Surface						
0–9 / 0–3	101.35–98.40	**Silty Sand Fill** Brown, trace to some gravel increasing with depth, construction debris in the upper levels. loose to very hard.	SS	1	20,50/4"	100		
			SS	2	2,4,5,4	9		
			SS	3	5,7,8,10	15		
			SS	4	7,13,13,16	26		
10–23 / 3–7	98.40–94.10	**Sand and Gravel** Brown, hard.	SS	5	33,50/2"	100		
			SS	6	40,50/2"	100		
			SS	7	23,22,48,50	70		
24–29 / 7–8	94.10	**Silty Sand** Brown, trace gravel, compact.	SS	8	20,13,16,20	29		

Moisture Content w% (10 20 30 40)
Standard Penetration Test blows/300mm (20 40 60 80)

Drill Method: Hollow Stem Auger
Drill Date: February 6,2025
Hole Size: 200 Millimetres
Drilling Contractor: Terra Firma

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Datum: Temporary Benchmark
Field Logged by: YP
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Sheet: 1 of 3

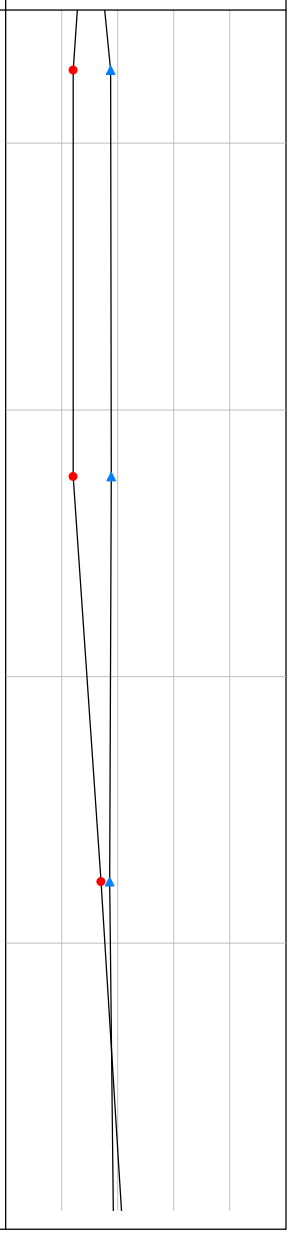
Log of Borehole No. 102

Project No: SM 241053-G
Project: Proposed Residential Development
Location: 84-104 Planins Road, Burlington
Client: Miura Development Group Inc.

Project Manager: Ian Shaw, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4795749
E: 593315

Depth	Elevation (m)	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)
10	90.30	**Silty Clay/Clayey Silt** Grey, trace gravel, very stiff to hard		SS	9	7,9,15,16	24		
12				SS	10	9,13,11,15	24		
15				SS	11	11,14,20,27	34		

Moisture Content w% (10 20 30 40)
Standard Penetration Test blows/300mm (20 40 60 80)



Drill Method: Hollow Stem Auger
Drill Date: February 6,2025
Hole Size: 200 Millimetres
Drilling Contractor: Terra Firma

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Checked by: IS
Sheet: 2 of 3

Log of Borehole No. 102

Project No: SM 241053-G
Project: Proposed Residential Development
Location: 84-104 Planins Road, Burlington
Client: Miura Development Group Inc.

Project Manager: Ian Shaw, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4795749
E: 593315

Depth	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	SAMPLE Number	SAMPLE Blow Counts	SAMPLE Blows/300mm	SAMPLE Recovery	SAMPLE PP (kgf/cm2)
19										
20	81.10		**Queenston Shale** Red, with more resistant grey layers, highly weathered in upper levels, becoming more sound with depth, hard. Spoon Refusal on assumed bedrock		SS	12	15,19,24,32	43		
21										
22	79.40				SS	13	50/2"	100		
23			End of Borehole							
24										
25										
26										

Moisture Content w%: 10 20 30 40

Standard Penetration Test blows/300mm: 20 40 60 80

NOTES:

1. Borehole was advanced using solid stem auger equipment on February 6, 2025 to Spoon Refusal on assumed bedrock at a depth of 21.3 metres.

2. Borehole was recorded as open and 'dry' upon completion and backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Hollow Stem Auger
Drill Date: February 6,2025
Hole Size: 200 Millimetres
Drilling Contractor: Terra Firma

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Datum: Temporary Benchmark
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Sheet: 3 of 3

Log of Borehole No. 3

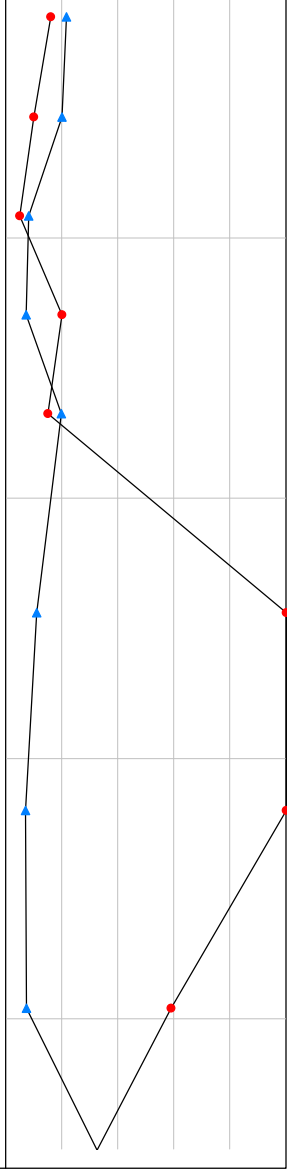
Project No: SM 241053-G
Project: Proposed Residential Development
Location: 84-104 Planins Road, Burlington
Client: Miura Development Group Inc.

Project Manager: Ian Shaw, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4795734
E: 593281

Depth (ft / m)	Elevation (m)	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)
0 / 0	101.19	Ground Surface							
0 / 0	100.99	**TopSoil** Approximately 200 millimetres of topsoil.							
1 / 0		**Silty Sand** Brown, trace gravel, loose to compact.		SS	1	9,7,3,3	16		
2 / 0									
3 / 1				SS	2	3,6,4,4	10		
4 / 1									
5 / 1				SS	3	1,2,3,3	5		
6 / 2									
7 / 2				SS	4	4,10,10,12	20		
8 / 2									
9 / 3				SS	5	4,7,8,9	15		
10 / 3									
11 / 3									
12 / 4									
13 / 4	97.10	**Sand and Gravel** Brown, hard.		SS	6	50/5"	100		
14 / 4									
15 / 5									
16 / 5									
17 / 5									
18 / 5									
19 / 6									
20 / 6				SS	7	14,50/4"	100		
21 / 6									
22 / 6									
23 / 7									
24 / 7									
25 / 7	93.60	**Silty Sand** Brown, trace gravel, compact.		SS	8	40,30,29,50/1"	59		
26 / 8									
27 / 8									
28 / 8									
29 / 8									

Moisture Content w% 10 20 30 40

Standard Penetration Test blows/300mm 20 40 60 80



Drill Method: Solid Stem Auger
Drill Date: January 28,2025
Hole Size: 150 Millimetres
Drilling Contractor: Terra Firma

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Datum: Temporary Benchmark
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Sheet: 1 of 2

Log of Borehole No. 3

Project No: SM 241053-G
Project: Proposed Residential Development
Location: 84-104 Planins Road, Burlington
Client: Miura Development Group Inc.

Project Manager: Ian Shaw, P.Eng
Borehole Location: See Drawing No.1
UTM Coordinates - N: 4795734
E: 593281

Depth	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	SAMPLE Number	SAMPLE Blow Counts	SAMPLE Blows/300mm	SAMPLE Recovery	SAMPLE PP (kgf/cm2)
30										
31					SS	9	8,10,11,12	22		
32	91.40									
33	10		End of Borehole							
34										
35										
36	11									
37										
38										
39	12									
40										
41										
42										
43	13									
44										
45										
46	14									
47										
48										
49	15									
50										
51										
52	16									
53										
54										
55										
56	17									
57										
58										
59										

Moisture Content w%
10 20 30 40

Standard Penetration Test blows/300mm
20 40 60 80

NOTES:

1. Borehole was advanced using solid stem auger equipment on January 28, 2025 to termination at a depth of 9.8 metres.

2. Borehole was recorded as caved to 8.2 metres and 'Wet' at a depth of 8.5 metres below the existing ground surface upon completion and backfilled as per Ontario Regulation 903.

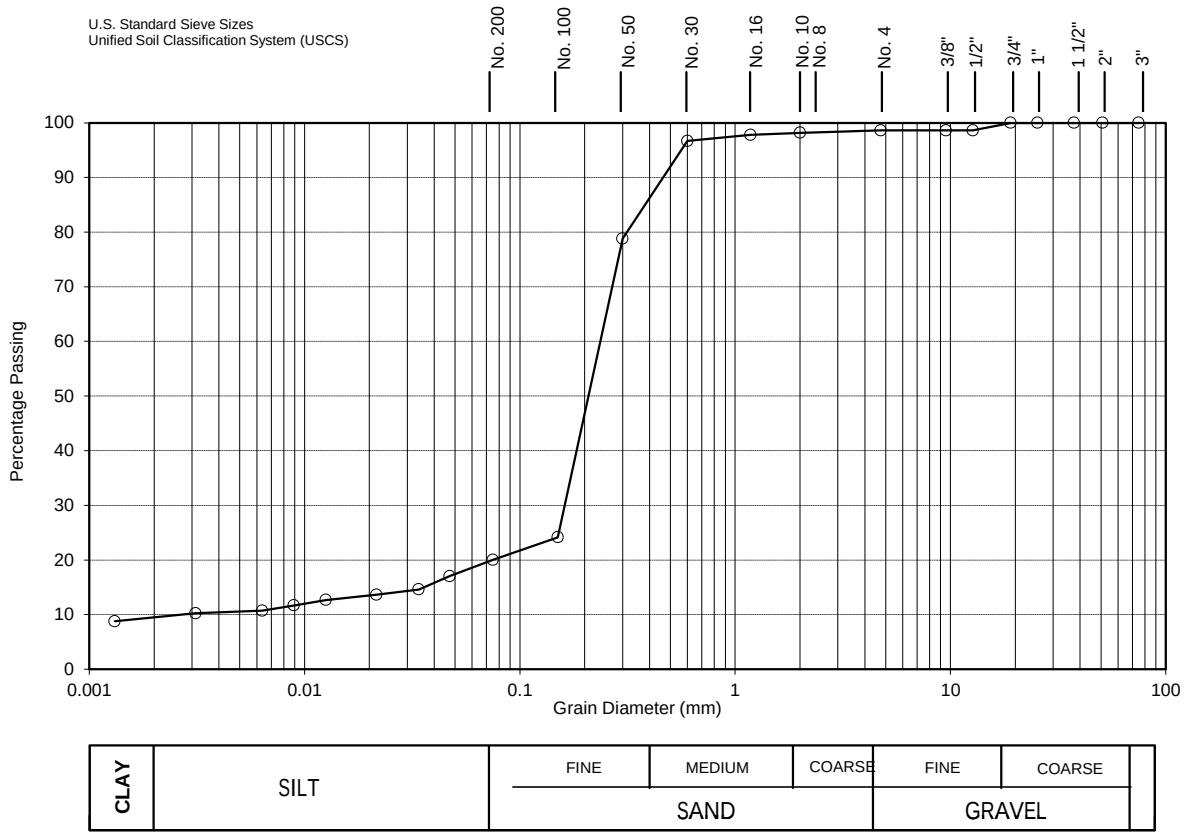
3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Solid Stem Auger
Drill Date: January 28,2025
Hole Size: 150 Millimetres
Drilling Contractor: Terra Firma

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Mechanical & Hydrometer Analyses



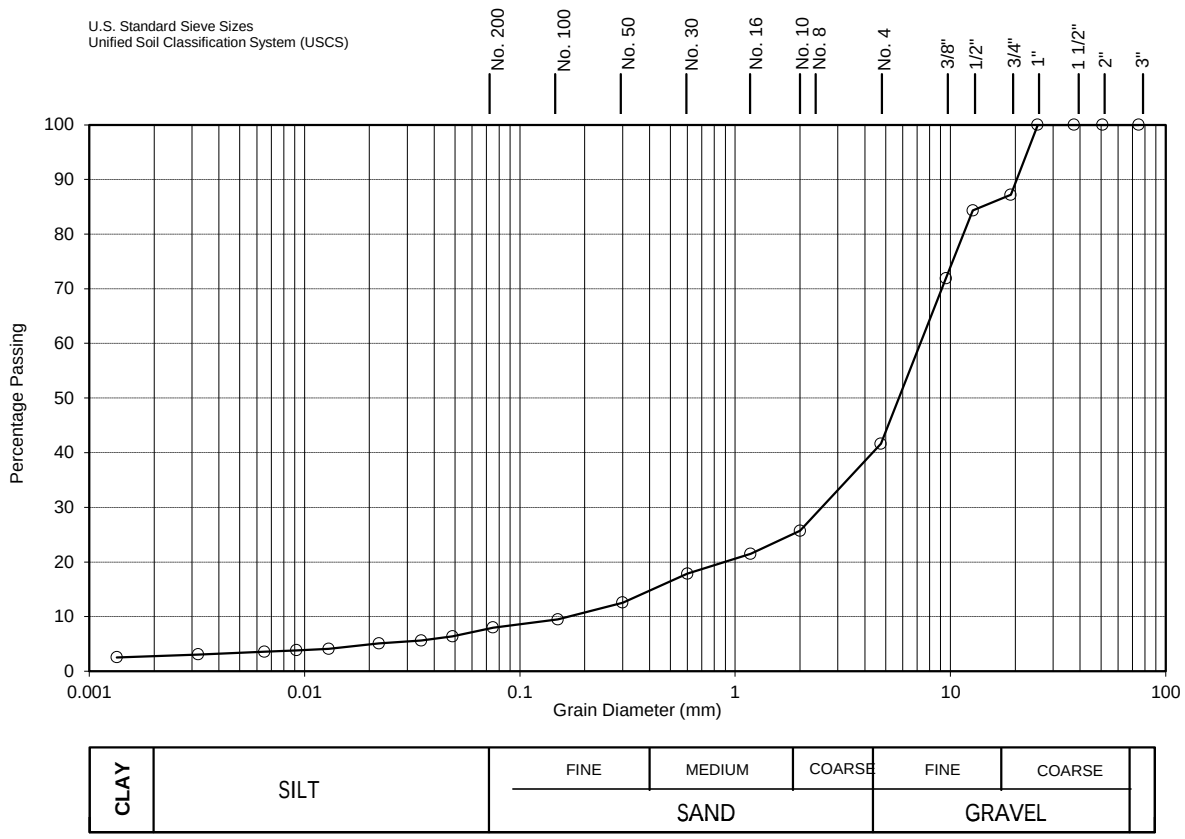
Lab No.:	**25-067**	Notes: Depth: 10'	
Borehole No.:	**1**		
Sample No.:	**5**		
CLAY [%]:	**9**	Soil Description:	**Reddish Brown Sand w/ some Silt and Clay and a trace of Gravel**
SILT [%]:	**10**		**S.P. - poorly graded sand, to S.M. - Sand-silt mixtures**
SAND [%]:	**80**		
GRAVEL [%]:	**1**	Estimated Infiltration Rate [mm/hr] : **50 to 60**	Estimated Permeability, k [cm/s]: **10^{-4}**
D_{10} (Effective Diam. in mm):	**0.0100**	Coefficient of Uniformity C_U: **24.0**	Coefficient of Curvature C_C: **12.0**

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

84 - 104 Plains Road East, Burlington ON

Date Tested: March 17, 2025	**Grain Size Analysis No. 1**	**Project No.: SM 241053-T**

Mechanical & Hydrometer Analyses



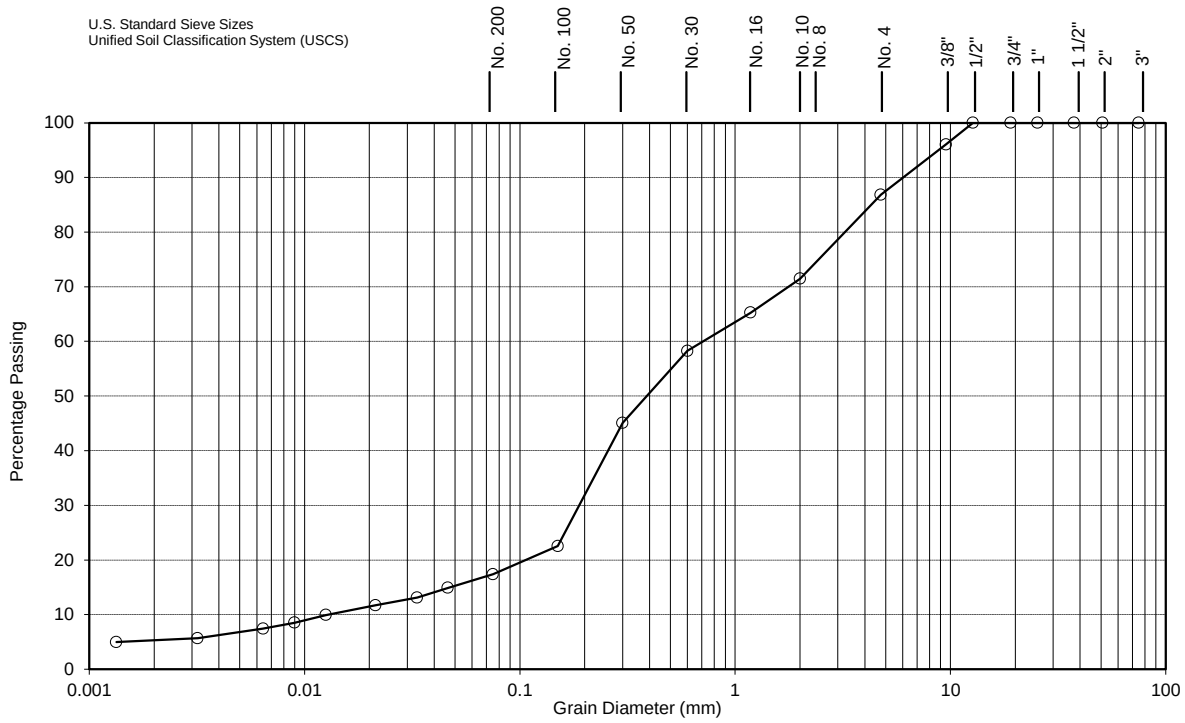
Lab No.:	**25-068**	Notes: Depth: 20'			
Borehole No.:	**1**				
Sample No.:	**7**				
CLAY [%]:	**3**	Soil Description: **Brown Sandy Gravel w/ traces of Silt and Clay**			
SILT [%]:	**5**	**G.P. - Gravel-sand mixtures, more than 5% fines to G.M. - Gravel-sand-silt mixtures**			
SAND [%]:	**34**				
GRAVEL [%]:	**58**	Estimated Infiltration Rate [mm/hr] :	**150 to 200**	Estimated Permeability, k [cm/s]:	**10^{-2}**
D_{10} (Effective Diam. in mm):	**0.16**	Coefficient of Uniformity C_U:	**44.4**	Coefficient of Curvature C_C:	**6.0**

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

84 - 104 Plains Road East, Burlington ON

Date Tested: March 17, 2025	**Grain Size Analysis No. 2**	**Project No.: SM 241053-T**

Mechanical & Hydrometer Analyses

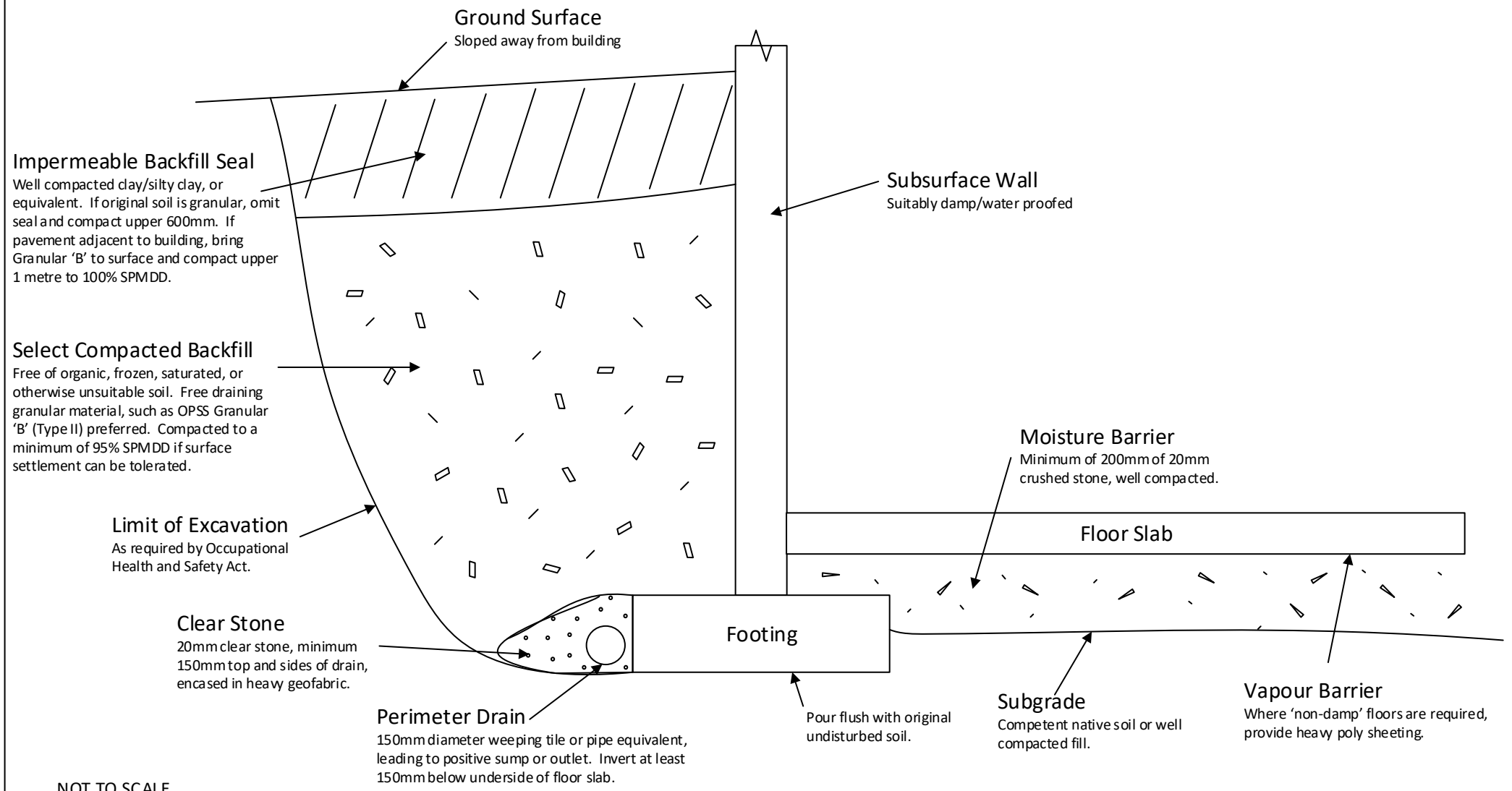


Lab No.:	**25-069**	Notes: Depth: 25'			
Borehole No.:	**3**				
Sample No.:	**8**				
CLAY [%]:	**5**	Soil Description:	**Brown Sand w/ some Gravel and Silt and a trace of Clay**		
SILT [%]:	**12**		**G.M. - Gravel-sand-silt mixtures**		
SAND [%]:	**70**				
GRAVEL [%]:	**13**	Estimated Infiltration Rate [mm/hr] :	**75 to 100**	Estimated Permeability, k [cm/s]:	**10^{-3}**
D_{10} (Effective Diam. in mm):	**0.013**	Coefficient of Uniformity C_U:	**53.8**	Coefficient of Curvature C_C:	**4.0**

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84 - 104 Plains Road East, Burlington ON

Date Tested: March 18, 2025	**Grain Size Analysis No. 3**	**Project No.: SM 241053-T**



NOT TO SCALE

Soil-Mat

Soil-Mat Engineers & Consultants Ltd.

Project No.:	SM 241053-G
Date:	June, 2025

Typical Design Requirements
Drainage and Backfill for Basement Walls

Drawing No. 2