GEI Consultants

Preliminary Geotechnical Investigation & Report

Proposed Residential Building

60 Dean Ave, Barrie, Ontario

Submitted to:
Nestwise Inc.
35 Worsley Street,
Barrie, Ontario

Submitted by:
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Certification

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Acronyms and Abbreviations

%	Percent (per 100 units)
<	Less than …
>	Greater than …
Δ	Change in …
µm	micrometer
ANSI	Area of Natural and Scientific Interest
APEC	Areas of Potential Environmental Concern
ASTM	American Society for Testing and Materials
bgs	Below Ground Surface
BH	Borehole
BH/MW	Borehole / Monitoring Well
cm	centimeters
CPT	Cone Penetration Tests
CVC	Credit Valley Conservation
DCPT	Dynamic Cone Penetration Test
EASR	Environmental Activity and Sector Registry
EBA	Event Based Area
ERIS	EcoLog Environmental Risk Information Services Ltd.
Elev.	Elevation
ET	Evapotranspiration/Evaporation
FOS	Factor of Safety
FSR	Functional Servicing Report
GEI	GEI Consultants Canada Ltd.
ha	hectares
hr	hours
HVA	Highly Vulnerable Aquifer
I	Infiltration
ICA	Issue Contributing Area
ID	Identification
IPZ	Intake Protection Zone
K	Hydraulic Conductivity
kg	kilogram
km	Kilometres
kPa	Kilopascal
L	Litres
LID	Low Impact Development
m	Metres
m^3	Cubic Meters
MECP	Ministry of Environment, Conservation and Parks
min	minute

mm	Millimetres
MMAH	Ministry of Municipal Affairs and Housing
MW	Monitoring Well
N values	“N” Values
OBC	Ontario Building Code
OHSA	Occupational Health and Safety Act
OPSS	Ontario Provincial Standard Specifications
OPSD	Ontario Provincial Standard Drawing
O.Reg.	Ontario Regulation
O.D.	Outside Diameter
P	Precipitation
PHC	Petroleum Hydrocarbon
PTTW	Permit to Take Water
PWQO	Provincial Water Quality Objective
PVC	Polyvinyl Chloride
R	Runoff
ROI/ROIs	Radius/Radii of Influence
ROW	Right of Way
s	Seconds
SLS	Serviceability Limit State
S	Storage
SCS	Site Condition Standards
SGRA	Significant Groundwater Recharge Area
SPT	Standard Penetration Test
SPmdd	Standard Proctor maximum dry density
SS	Split Spoon
SWM	Storm Water Management
ULS	Ultimate Limit State
USCS	Unified Soil Classification System
VOC	Volatile Organic Compound
WHPA	Wellhead Protection Area

It is noted that all elevations in this report are metric/geodetic and expressed in m. All measurements are also in metric and expressed in mm, m or km.

1. Introduction

GEI was retained by Nestwise Inc. (the Client) to complete a preliminary geotechnical report in support of the proposed residential building at the property at 60 Dean Ave in Barrie, Ontario. A site location plan is enclosed as Figure 1.

The project site is located on the north side of Dean Ave and is approximately 60 m east/west and 120 m north/south. Commercial properties border the site to the North and West and residential developments border to the East and South. Some stockpiles of soil are noted at the site. A public library with a parking lot borders west side of the property. The site currently consists of an undeveloped field with stockpiles scattered in the central and northern portion of the property. An aerial photograph of the site with concept plan overlaid is shown in Figure 2.

GEI was provided with following drawings for review:

- *“Conceptual Site Plan – 60 Dean Avenue, City of Barrie, County of Simcoe 2024”* By The Jones Consulting Group, dated July 2024.
- *“General Servicing Plan – 60 Dean Avenue”,* By The Jones Consulting Group, dated February 2025, Project No.: PRA-24067.
- *“Site Grading Plan – 60 Dean Avenue”,* By The Jones Consulting Group, dated February 2025, Project No.: PRA-24067.

The current concept consists of a six (6) storey building with one level of underground parking (Elev. 249.7). At grade parking/access will also be provided. The development would be connected to municipal servicing and the proposed concept plan is shown in Figure 2.

The purpose of the preliminary investigation was to assess the subsurface soil conditions at the site, and based on this information, provide preliminary geotechnical engineering recommendations in support of the proposed development. This report summarizes the borehole findings, provides preliminary design geotechnical engineering recommendations regarding site earthworks and engineered fill, available bearing capacities for foundations, slabs-on-grade, earth pressures and drainage for underground parking, site servicing installation, pavement design, and preliminary infiltration parameters. Considerations for constructability such as soil excavation, compaction, on-site backfill suitability and temporary groundwater control are also provided.

As the design progresses further geotechnical review and input may be required which might necessitate the need for additional investigation and/or analysis.

A Phase 1 Assessment Report and a Hydrogeological Report were also requested and are presented under separate covers.

In addition, one (1) year of groundwater level monitoring has been requested with the summary report to be provided when monitoring is completed.

Lastly, MASW testing was requested for the site and the report is appended.

2. Procedures and Methodology

Prior to the commencement of drilling activities, the borehole locations were staked in the field by GEI. Ground surface elevations of the boreholes and horizontal coordinates (referencing NAD 83 geodetic datum) were surveyed by GEI with a Topcon FC – 5000 GPS Survey unit. Underground utilities including natural gas, electrical, telephone, water, etc. were marked out by public and private utility locating companies prior to drilling.

The fieldwork for the drilling program was carried out on December 18, 2024. Boreholes 1 to 5 were advanced to 6.6 m below existing grade (Elev. 242.6 to 244.3) across the site. The elevations are provided on the borehole logs in Appendix A. Borehole locations are shown on Figure 2.

The boreholes were advanced by a drilling subcontractor retained and supervised by GEI using a track-mounted drill rig, solid stem augers, and standard soil sampling equipment. Sampling was conducted using a 51 mm OD SS sampler. SPT N values were recorded for the sampled intervals as the number of blows required to drive an SS sampler 305 mm into the soil using a 63.5 kg drop hammer falling 750 mm, in accordance with ASTM D1586. In each borehole, soil sampling was conducted at 0.75 m intervals for the upper 3.0 m, and at 1.5 m intervals thereafter.

Monitoring wells were installed in three (3) boreholes by GEI to facilitate long-term groundwater monitoring, each consisting of 50 mm diameter PVC pipe with a 1.5 m or 3.0 m long screen and protective casing. Monitoring well construction is shown on the borehole logs in Appendix A. Boreholes without wells were backfilled in accordance with O.Reg. 903.

The GEI field staff examined and classified characteristics of the soils encountered in the boreholes, including the presence of fill materials, groundwater observations during and upon completion of the drilling, recorded observations of borehole construction, and processed the recovered samples. All recovered soil samples were logged in the field, carefully packaged, and transported to GEI's laboratory for more detailed examination and classification.

In GEI's laboratory, the soil samples were classified as to their visual and textural characteristics. All samples were submitted for moisture content determination in accordance with ASTM D2216. Four (4) representative soil samples were selected and submitted to our laboratory for grain size analysis. Grain size analysis results are provided in Appendix B.

3. Subsurface Conditions

3.1. General Overview

The detailed soil profiles encountered in the boreholes are indicated in the attached borehole logs in Appendix A. The geotechnical laboratory results are included in Appendix B. The borehole locations are shown on Figure 2.

It should be noted that the conditions indicated on the borehole logs are for specific locations only and can vary between and beyond the locations. It should be noted that the soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. The boundaries are intended to reflect approximate transition zones and should not be interpreted as exact planes of geological change.

In addition, the descriptions provided in the borehole logs are inferred from a variety of factors, including visual observations of the soil samples retrieved, laboratory testing, measurements prior to and after drilling, and the drilling process itself (speed of drilling, shaking/grinding of the augers, etc.). The passage of time may result in changes in conditions to exist at locations where sampling was conducted.

3.2. Stratigraphy

3.2.1. Topsoil

A surficial topsoil layer was at the ground surface in Boreholes 1, 2, 4, and 5. The topsoil ranged in thickness from 75 to 760 mm. Topsoil thickness may vary between boreholes and in other areas of the site.

3.2.2. Fill

A fill layer was encountered beneath the topsoil in Boreholes 1, 2, 4, and 5 and was penetrated at 0.8 and 1.5 m depth (Elev. 248.4 to 249.0). A surficial fill layer was encountered in Borehole 3 and extended to 1.5 m below ground surface (Elev. 249.3). The fill material consisted of silty sand with trace to some gravel. Trace organics were also observed in some samples. The fill was moist with moisture contents of 10 to 19%. The fill had N values ranging from 6 to 30 blows, showing loose to compact conditions.

3.2.3. Glacial Till

Underlying the topsoil and fill layer, a glacial till deposit was encountered in all boreholes and extended to the 6.6 m depth of exploration (Elev. 242.6 to 244.3), locally penetrated at 4.6 m depth (Elev. 245.2) in Borehole 5. The till matrix consisted of silty sand with trace to some clay and trace gravel. Cobbles and boulders should be expected based on augers grinding during advancement of the boreholes. Three (3) samples of the material were submitted for grain size analysis and the results are provided in Figure B1 in Appendix B. The soil was moist, with moisture contents of 5 to 12%, local wet seams were noted. N values ranged from 13 to greater than 100 blows, indicating compact to very dense conditions, but typically dense to very dense, being compact near the top of the unit.

3.2.4. Gravelly Sand

Underlying the silty sand glacial till in Boreholes 5, a gravelly sand unit with some silt and trace clay was encountered from 4.6 to 6.6 m depth of exploration (Elev.245.2 to 243.2). One (1) sample of the material was submitted for grain size analysis and the results are provided in Figure B2 in Appendix B. The gravelly sand was wet, with moisture contents of 7 to 11%. N values were greater than 100 blows, indicating very dense conditions.

3.3. Groundwater

Unstabilized groundwater level measurements and cave measurements were taken upon the completion of drilling of each borehole as shown on the borehole logs in Appendix A. These measurements were taken to provide a rough estimate of the possible excavation and temporary groundwater control constructability considerations that may arise. Three (3) boreholes were outfitted with a monitoring well with 50 mm diameter pipe and 1.5 to 3.0 m long screen. Monitoring well configuration and groundwater observations are noted on the borehole logs in Appendix A and summarized in the table below.

Table 3-1. Groundwater Levels

Borehole	Depth of Cave (m) / Elev.	Unstabilized Groundwater Level Depth (m) / Elev.	Depth (m) / Elev. of Groundwater Table, January 08, 2025
BH1	Open	3.4 / 246.7	N/A
BH2	Open	4.0 / 246.8	2.3 / 248.5
BH3	Open	No Water	2.2 / 248.6
BH4	Open	No Water	N/A
BH5	Open	5.2 / 244.6	1.7 / 248.1

The stabilized groundwater level in the monitoring wells was measured to be 1.7 to 2.3 m below the existing ground surface, at Elev. 248.1 to 248.6, in Boreholes 2,3 and 5. Some wet seams were noted in the till and the groundwater in the gravelly sand is under some artesian pressure.

GEI is currently conducting a one (1) year groundwater level monitoring program, and the results will be provided separately upon completion.

The existing fill and gravelly sand are permeable and will allow for the free flow of water. The silty sand glacial till layer is only semi-permeable will allow for some free flow of ground water when wet.

Groundwater levels are expected to show seasonal fluctuations and vary in response to prevailing climate conditions.

4. Engineering Design Parameters & Analysis

The project site is located on the north side of Dean Ave and is approximately 60 m east/west and 120 m north/south. Commercial properties border the site to the North and West and residential developments border to the East and South. Some stockpiles of soil are noted at the site. A public library with a parking lot borders west side of the property. The site currently consists of an undeveloped field with stockpiles scattered in the central and northern portion of the property. An aerial photograph of the site with concept plan overlaid is shown in Figure 2.

GEI was provided with following drawings for review:

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The current concept consists of a six (6) storey building with one level of underground parking (Elev. 249.7). At grade parking/access will also be provided. The development would be connected to municipal servicing and the proposed concept plan is shown in Figure 2.

4.1. Site Grading

Based on the grading plan provided, the finished ground floor of the building is set at Elev. 252.9, and the finished floor of the underground parking level is set at Elev. 249.7. It is anticipated that footings would be set just below low the underground parking slab at about Elev. 249.2.

It is noted that the upper 0.8 to 1.5 m of soil is comprised of topsoil and fill and is considered unsuitable for building foundations and floor slabs. The stockpiled soil is also not suitable to support the building. In the building area, Boreholes 2, 3 and 5 show that the footings would be founded typically on the native till. It is noted that the anticipated footing elevation described above is at the top of the till unit and locally there may be areas where fill is present at the footing elevation. In this scenario it is recommended that footings extend down to competent native soil for uniform bearing. The underground parking floor slab can be supported by engineered fill or native soil as required.

In the building area, it is recommended to strip the topsoil and stockpile separately, then sub-excavate the existing fill down to competent native soil. The exposed subgrade surface should be thoroughly compacted and then engineered fill placement can commence to the desired grade.

4.1.1. Engineered Fill

GEI defines "engineered fill" as material that will support foundations, and which is placed and compacted in a specified and controlled manner under full-time supervision of geotechnical engineering staff.

In any location where engineered fill will be placed to raise grades or replace poor/weak soil, the topsoil, vegetation, weathered/disturbed, or existing earth fill must be fully removed down to competent soil. The exposed subgrade soil must be proof-rolled and inspected by the geotechnical engineer to ensure all unsuitable material (e.g. organics, weak or soft soil, weathered/disturbed soil, deleterious materials, existing fill) was removed from the engineered fill footprint. Any unsuitable areas must be further sub-excavated and replaced with approved fill compacted to 100% SPmdd, and 95% SPmdd in road and servicing areas.

Once the subgrade is approved, engineered fill can be placed. Engineered fill must be placed under the full-time supervision of a geotechnical engineer as required in the Ontario Building Code. The engineered fill may consist of excavated on-site cohesionless soil provided the material has been moisture conditioned to a moisture content within 2% of optimum moisture content and do not contain organics, topsoil or deleterious material. It is recommended that any imported soil used for site grading consist of Granular B (OPSS.MUNI 1010) and be first used in building areas, with suitable on-site soil used in landscaped or road areas. Engineered fill must be placed in loose lifts of 200 mm or less and compacted as noted above.

The exposed subgrade may be wet from locally perched water. In wet subgrade areas, the first lift of engineered fill shall consist of 400 mm of Granular B Type II (OPSS.MUNI 1010). This will help to bridge the weaker subgrade and improve the ability to achieve the compaction specifications for subsequent engineered fill lifts.

The engineered fill must extend a minimum of 1 m out from all sides of the foundations and extend at a 1 horizontal to 1 vertical slope (1H:1V) down to the exposed subgrade. A typical detail for engineered fill pad dimensioning is included in Appendix C.

4.2. Foundations

4.2.1. Foundations on Native Soil

As noted earlier, the finished ground floor of the building is set at Elev. 252.9, and the finished floor of the underground parking level is set at Elev. 249.7. It is anticipated that footings would be set just below low the underground parking slab at about Elev. 249.2.

Based on Boreholes 2, 3 and 5 in the building area, foundations on the native till soil at this site may be constructed as conventional shallow spread and strip footing foundations that bear on the native, undisturbed soil. For design purposes, a geotechnical reaction at SLS of 250 kPa is available Elev. 249.2 with corresponding factored geotechnical resistance at ULS of 375 kPa on the native till. The geotechnical reaction at SLS is for 25 mm or less of total settlement.

As noted above, the anticipated footing elevation described above is at the top of the till unit and locally there may be areas where fill is present at the footing elevation (till is slightly below the footing founding elevation). In this scenario it is recommended that footings extend down to competent native soil for uniform bearing.

Final footing elevations must be reviewed by geotechnical personnel from GEI to confirm bearing capacity values. The final site configuration must also be reviewed by GEI to assess the potential for footings to be founded on different soil subgrades, and to assess the potential for differential settlement. It is recommended that all foundations for each individual building / structure be set on the same soil subgrade wherever possible, to reduce the potential for differential settlement.

4.2.2. Foundations on Engineered Fill

If any building foundations are supported on an engineered fill pad, constructed as discussed in Section 4.1.1, the spread or strip footings can be designed using the underlying native soil bearing capacity shown above, up to a maximum of 150 kPa at SLS and 225 kPa at ULS. It is noted that this represents a lower bearing capacity than the native soil and would require additional design requirements for different bearing capacities.

It is recommended that nominal reinforcing steel for stiffening of the foundation walls made on engineered fill be provided to help mitigate minor cracking due to minor differential settlement. The reinforcing steel in the poured concrete foundation walls may consist of 2-15M bars continuous at the top of the foundation wall, and 2-15M bars continuous at the bottom of the foundation walls. Typically, these bars are placed 100 to 200 mm from the top or bottom of the foundation wall, respectively. The reinforcing steel should extend a minimum of 3 m past any transition zones between engineered fill and native soil. A typical reinforcing steel detail for foundation walls placed on engineered fill is provided within Appendix C. The recommended nominal reinforcing steel should not be considered a structural design. The need for different or additional reinforcement should be reviewed by a structural engineer to ensure the original structural design intent of the structure is maintained.

4.2.3. General Foundation Considerations

All footings exposed to ambient air temperature throughout the year must be provided with a minimum of 1.2 m of earth cover or equivalent insulation for frost protection (25 mm of polystyrene insulation is equivalent to 300 mm of soil cover). The minimum strip and spread footing widths to be used shall be dictated as per the Ontario Building Code, regardless of loading considerations. Footings stepped from one level to another must be at a slope not exceeding 7V:10H.

The foundation design parameters provided above are predicated on the assumption that the foundation subgrade surface is undisturbed, and that all earth fill, deleterious, softened, disturbed, organic, and caved material is removed. The foundation excavation must be done in such a way that groundwater is controlled to prevent any disturbance to the foundation base. The groundwater table must be lowered at least 1 m below the founding elevation prior to excavation to prevent disturbance to the foundation subgrade from groundwater seepage.

The foundation subgrade must be reviewed prior to concrete placement to ensure the foundation design parameters provided are applicable, and to provide remedial recommendations if necessary. If the foundation excavation will be open for a prolonged period of time, the foundation subgrade should be protected with a skim coat of lean mix concrete (applied immediately after inspection by the geotechnical engineer), to ensure that no deterioration will occur due to weather effects.

4.3. Seismic Site Classification

The 2024 Ontario Building Code came into effect on January 1, 2025, and notable amendments to the 2012 Building Code pertaining to the seismic site classification are listed below:

- As per section 4.1.8.4, Site Properties, OBC 2024, the site designation shall be determined from Table 4.1.8.4.-A using the average shear wave velocity, Vs30, calculated from in situ measurements of shear wave velocity.
- Where Vs30 calculated from in situ measurements is not available, the site designation shall be Xs, where S is the Site Class determined using the energy-corrected average standard penetration resistance, N60, or the average undrained shear strength, Su, in accordance with Table 4.1.8.4,-B, N60 and Su being calculated based on rational analysis.

MASW testing has been requested for the site and the report is in Appendix D. Based on the data in the report the site designation is represented by the average shear wave velocity of 750 m/s or V_{750} corresponding to Site Class C.

4.4. Earth Pressure Design Parameters

Underground parking levels or other retaining type walls must be designed to resist unbalanced lateral earth pressures imparted from the weight of adjacent soils. Lateral earth pressures are calculated using the following equation:

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

where,
P = the horizontal pressure at depth, **h** (m)
K = the earth pressure coefficient (dimensionless)
h = depth below ground surface (m)
$\boldsymbol{\gamma}$ = the bulk unit weight of soil, (kN/m^3)
q = surcharge loading (kPa)

The above equation assumes that a drainage system is present which prevents the build-up of any hydrostatic pressure behind the structure subjected to the unbalanced lateral earth pressures. If this is not the case, the equation must be revised to also incorporate the submerged unit weight of the soil multiplied by the earth pressure coefficient, in addition to the water pressure itself.

The values for use in the design of the walls subjected to unbalanced lateral earth pressures in the upper 3 to 4 m of the site are as follows:

Table 4-1. Earth Pressure Coefficients

Soil Type	γ – Bulk Unit Weight (kN/m3)	Φ – Friction Angle (degrees)	Earth Pressure Coefficient		
			Ka - Active	Ko – At-Rest	Kp - Passive
Granular ‘B’ (OPSS 1010)	21.0	32	0.31	0.47	3.25
Compact to Very Dense Native Soil/Engineered Fill	20.0	30	0.33	0.50	3.00
Fill Soil	20	28	0.36	0.57	2.77

The calculation of the earth pressure coefficients is based on Rankine theory, which provides a conservative estimate as no friction between the soil and the structure is accounted for. The earth pressure coefficients provided above are applicable for flat ground surfaces beyond the structure and must be revised for sloping ground surfaces.

The earth pressure coefficients referenced within the above table are a function of the friction angle of the adjacent soil, and both the degree and direction of movement of the structure subjected to unbalanced lateral earth pressures. For structures that are restrained at the top (such as basement walls), the at-rest earth pressure coefficient will apply. For structures that allow for 0.1 to 1% of movement away from the soil (such as unrestrained retaining walls), the full active earth pressure coefficient will apply. For structures that allow for 1 to 10% of movement into the soil, the full passive earth pressure coefficient will apply. The percentage movement is based on the height of the structure.

Other types of structures such as shoring walls with multiple rows of tiebacks and soil nail walls are subject to different loading conditions and must be analyzed separately.

4.5. Floor Slabs

The native soils or engineered fill are suitable to support the parking garage floor slabs.

The exposed subgrade/top of the engineered fill must be proof-rolled and inspected by the geotechnical engineer. If any soft or weak subgrade areas are identified, or if there are areas containing excessive amounts of deleterious/organic material, they must be locally sub-excavated and backfilled with approved site earth fill or imported granular material and compacted to a minimum of 98% SPmdd, targeted 100% SPmdd, within 2% optimum moisture content.

All floor slabs must be provided with a capillary moisture barrier and drainage layer. This is made by placing the concrete slab on a minimum 200 mm layer of 19 mm clear stone (OPSS.MUNI 1004) compacted by vibration to a dense state. The upper 50 mm of clear stone can be replaced with 19 mm crusher run limestone for a working surface. The clear stone and a cohesionless subgrade must be separated by a geotextile such as Terrafix 270R (or approved equivalent) to prevent the migration of fines into the clear stone layer which could result in loss of support for the slab. Alternatively, Granular A (OPSS.MUNI 1010) compacted to 100% SPmdd can be utilized without filter cloth.

4.6. Drainage

For underground parking, all foundation walls must be provided with damp-proofing provisions in conformance to the Ontario Building Code. Backfill along the foundation wall must consist of Granular B Type I (OPSS.MUNI 1010) for a minimum lateral distance of 600 mm out from the foundation wall. Alternatively, if a filtered cellular drainage media is provided adjacent to the foundation wall, the backfill may consist of common earth fill.

For underground structures, a perimeter drainage system must be installed that will remove any water that infiltrates into the building backfill, to ensure that any water does not infiltrate into the underground parking. The perimeter drains must consist of minimum 100 mm diameter perforated pipes wrapped in filter socks, sufficiently covered on all sides by 19 mm clear stone. Perimeter drains should be directed to the sump underneath the basement floor in solid pipes so as not to surcharge the underfloor drainage layer with water. As the underground parking will be established more than 1 m above the groundwater levels, an underfloor drainage system is not required. A typical basement drainage detail is included in Appendix C.

4.7. Site Servicing

Based on the servicing plan, service inverts on the site range about 3.0 to 4.0 m below grade. Some slightly deeper excavation may be required for connection points on Dean Avenue.

4.7.1. Bedding

The type of material and depth of granular bedding below the pipe will, to some extent, depend on the method of construction used by the contractor. Pipe bedding for flexible pipes should follow the requirements in OPSD 802.010 or applicable municipal standards. Pipe bedding for rigid pipes should follow the requirements in OPSD 802.030 to 802.032 or applicable municipal standards.

A subgrade consisting of the native soil will provide adequate support for pipes with the bedding requirements as laid out in the above referenced OPS drawings. Where disturbance of the trench base has occurred from groundwater seepage, construction traffic, etc., the material should be sub-excavated and replaced with suitably compacted granular fill. If weak zones are encountered, additional bedding materials and differing construction practices may be required and should be determined during construction. Any zones of peat or organic soil should be sub-excavated and replaced with approved earth fill or imported granular material compacted to 95% SPmdd. Details on temporary groundwater control are provided in Section 5.2.

Regardless of whether flexible or rigid pipes are implemented, granular bedding and cover material should consist of a well graded, free draining material, such as Granular “A” (OPSS.MUNI 1010). All granular bedding must be compacted to a minimum of 95% SPmdd.

4.7.2. Backfill

Excavated native cohesionless soil may be re-used as backfill in trenches, provided they are moisture conditioned so that the moisture content is within 2% of optimum. Additional soil compaction details are provided in Section 5.3. The backfill should be compacted to a minimum of 95% SPmdd. In confined areas the layer thickness will have to be reduced to utilize smaller compaction equipment efficiently or by using granular material instead of locally sourced fill. Any backfill that is frozen, contains a high percentage of organic material (topsoil, peat, etc.) or moisture, or has otherwise unsuitable deleterious inclusion should not be used as backfill. The maximum cobble or boulder size should not exceed half of the loose lift thickness (i.e. all particles with a diameter greater than 100 mm should be removed).

Where trenches are within the traveled portions of a roadway/parking area, backfill within the frost penetration depth of 1.2 m should consist of native, non-organic, excavated material consistent with the soils surrounding the trench. If this technique is not undertaken, then frequently problems arise with yearly differential frost heave movements between the trench backfill and the adjacent native soil. This would occur, for example, if imported granular material is used to backfill trenches which is less susceptible to frost effects compared to the native soils on site. Alternatively, if different soil is used as the backfill due to issues with achieving compaction, a frost taper of 10H:1V can be implemented to help mitigate the potential for differential settlement and frost heave.

4.8. Infiltration

LID facilities are being planned at the site and will generally be located underneath the proposed parking lot and driveway.

Typical design of infiltration facilities has the base of the feature a minimum of 1 m above the groundwater table. The preliminary infiltration rate provided below is not applicable below the groundwater table. The latest groundwater level measurement shows the groundwater onsite to be 1.7 to 2.3 m below existing grade.

For preliminary design purposes, the hydraulic conductivity of the silty sand glacial till encountered on site is generally estimated to be about 1 x 10^{-5} to 1 x 10^{-6} cm/s based on the grain size curves, not considering the density of the soil. The resulting unfactored infiltration rate is assumed to be about 30 to 50 mm/hr.

Appendix C of “Low Impact Development Stormwater Management and Planning Design Guide” (Version 1.0, 2010, by CVC and TRCA) suggests safety factors to be applied to infiltration rates. The safety factor applicable to the site is expected to be 2.5 but this must be confirmed during detailed design. Based on the above, factored values of 12 to 20 mm/hr. are anticipated.

Once the final location and elevation of LID measures are known, in-situ infiltration testing using the Guelph Permeameter can be completed to refine the infiltration rates. However, it is noted that unfactored values of less than 15 mm/hr. are generally considered impractical for infiltration.

4.9. Pavement Design

Based on the grading plan provided, other than removal of the stockpiles, grade will be close to existing grade requiring cut and fill of about 1 m+/-. As such the pavement subgrade will consist of the near surface soils comprising silty sand fill which is generally considered to have a low frost susceptibility.

The pavement subgrade must be inspected and approved by the geotechnical engineer at the time of construction. The exposed pavement subgrade should be compacted to a minimum of 95% SPmdd. If any soft or weak subgrade areas are identified, or if there are areas containing excessive amounts of moisture or deleterious/organic material, they must be locally sub-excavated and backfilled with approved clean earth fill or imported granular material and compacted to a minimum of 95% SPmdd.

The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures must be maintained to ensure that uniform subgrade moisture and density conditions are achieved as much as possible when fill is placed, and the subgrade is not disturbed or weakened after it is exposed.

Control of surface water is an important factor in achieving a good pavement life. The need for adequate subgrade drainage cannot be over-emphasized. The subgrade must be free of depressions and sloped (at a minimum grade of 2 percent) to provide effective drainage toward subgrade drains. Grading adjacent to pavement areas should be designed to ensure that water is not allowed to pond adjacent to the outside edges of the pavement.

Continuous pavement subdrains should be provided along the edges of the pavement and drained into respective catch basins to facilitate drainage of the subgrade and the granular materials. The subdrain invert should be maintained at least 0.3 m below subgrade level. To minimize the problems of differential movement between the pavement and catch basins/manhole due to frost action, the backfill around the structures should consist of free-draining OPSS Granular B. Typical pavement drainage details are provided in Appendix C.

4.9.1. Pavement Structure

The industry pavement design methods are based on a design life of 15 to 20 years for typical weather conditions depending on actual traffic volumes. The following pavement thickness designs are provided on the above noted considerations and anticipating the subgrade will comprise of a low frost susceptible soil.

Table 4-2. Pavement Design

Pavement Layer	**Compaction Requirement**	**Min. Component Thickness (mm)**	
		Light Duty	**Heavy Duty**
Surface Course Asphaltic Concrete: HL3 (OPSS 1150) with PG 58-28 Asphalt Cement (OPSS.MUNI 1101)	92% MRD (OPSS.MUNI 310)	40 mm	40 mm
Binder Course Asphaltic Concrete: HL8 (OPSS 1150) with PG 58-28 Asphalt Cement (OPSS.MUNI 1101)		50 mm	80 mm

Pavement Layer	**Compaction Requirement**	**Min. Component Thickness (mm)**	
		Light Duty	**Heavy Duty**
Base Course: Granular A (OPSS.MUNI 1010)	100% SPmdd (OPSS.MUNI 501)	150 mm	150 mm
Subbase Course: Granular B Type I (OPSS.MUNI 1010)		300 mm	450 mm

The granular materials should be placed in lifts 200 mm thick or less and be compacted to a minimum of 100% SPmdd for both granular base and subbase. The granular and asphalt pavement materials and their placement should conform to OPSS 310, 501, 1010 and 1150.

If the pavement construction occurs in wet, winter or inclement weather, it may be necessary to provide additional subgrade support for heavy construction traffic by increasing the thickness of the granular subbase, base or both. Further, traffic areas for construction equipment may experience unstable subgrade conditions. These areas may be stabilized utilizing additional thickness of granular materials or the use of geogrid.

It should be noted that in addition to adherence of the above pavement design recommendations, a close control on the pavement construction process will also be required in order to obtain the desired pavement life. Therefore, it is recommended that regular inspection and testing should be conducted during the pavement construction to confirm material quality, thickness, and to ensure adequate compaction.

Smooth transitions are required in all areas where the new pavement meets the existing asphalt surface. Asphalt joints shall follow OPSS.MUNI 310. Frost tapers of 10H:1V should be implemented between areas of differing pavement thickness and tie-in areas to existing pavement. Longitudinal asphalt joints should be milled into the existing asphalt a minimum 0.5 m for each lift. Transvers joint shall be milled into the existing asphalt a minimum 0.5 m for each lift. Successive joints should be staggered.

5. Constructability Considerations

5.1. Excavation

Excavations for the project are anticipated to be 2.0 to 3.0 m for the building and as deep as 4.0 m for servicing, locally deeper at the tie in location on Dean Avenue. Below the topsoil and stockpiled materials, excavations are anticipated to encounter the local fill and the underlying native glacial till. Harder digging should be expected in the dense to very dense soils. The presence of cobbles and boulders should be expected in the till deposit.

Excavations must be carried out in accordance with the Occupational Health and Safety Act, Ontario Regulation 213/91 (as amended), Construction Projects, Part III - Excavations, Section 222 through 242. Where workers must enter a trench or excavation the soil must be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act (OHSA). These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. If more than one soil type is encountered in an excavation, the most conservative soil type must be followed for sloping the sidewalls of the excavation. It is expected that most excavations should be completed considering a Type 3 soil geometry, 1H:1V from the base of the excavation, assuming adequate groundwater control is implemented as discussed in the next section, prior to excavation.

Excavation sidewalls will need to be continuously reviewed for evidence of instability and ground water seepage, particularly following periods of heavy rain or thawing. When required, remedial action must be taken to ensure the continued stability of excavation slopes and the safety of the workers.

Minimum support system requirements for steeper excavations are stipulated in Sections 235 through 238 and 241 of the OHSA and include provisions for timbering, shoring and moveable trench boxes. To reduce the potential for instability of the trench excavations, materials excavated from the service trenches and/or other fill materials or heavy equipment should not be placed near the crest of the trench excavations.

It is important to note that soils encountered in the construction excavations may vary significantly across the site. Our soil classifications are based solely on the materials encountered in the boreholes advanced on site. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions are encountered at the time of construction, we recommend that GEI be contacted immediately to evaluate the conditions encountered.

5.2. Temporary Construction Groundwater Control

As noted above, excavation is envisioned to extend to about 2 to 4 m depth below existing grades for the project.

Groundwater was present in all three (3) wells installed in Boreholes 2, 3, and 5. The stabilized groundwater levels in the monitoring wells were measured at 1.7 to 2.3 m depth below the existing ground surface, corresponding to Elev. 248.1 to 248.6. The groundwater in the gravelly sand appears to be under some artesian pressure, and there are wet seams in the till soil.

The exact scenario where certain groundwater control techniques will work are directly correlated to how coarse/fine the native soils are in an excavation, and both the lateral and vertical extent of the wet cohesionless deposits. If the groundwater table is not controlled during construction, the base of the excavations will be unstable, leading to difficulties in excavating and placement of pipes, footings or engineered fill, and providing safety for the workers.

The excavation for the building is generally above the groundwater table. Servicing excavation is anticipated to be as much as about 2 to 3 m below the groundwater level measurements taken to date. Conventional sump pumping should suffice to control the anticipated groundwater seepage for excavation slightly below the groundwater levels noted, due to the low seepage volumes expected from the till. Areas with greater seepage from wet seams may require multiple pumps and/or sumps created with a corrugated steel pipe filled with gravel. For deeper excavations into the groundwater table where gravelly sand is present, dewatering through well points will be required.

It is recommended to carry out the work during the dry time of the year when the ground water table is lowest, to mitigate groundwater control measures. Also reducing the size of the excavation that is open at any one time will aid in reducing groundwater control requirements.

A PTTW is likely not required if excavation is kept above the gravelly sand. Registry on the EASR system is also likely not required if excavation is carried out in small manageable sections and excavation is generally above the groundwater table.

GEI's hydrogeological study under a separate cover provides further details regarding water taking analysis, regulatory and permitting requirements, impact assessments, monitoring plans, etc. for the site and must be referenced for groundwater control considerations.

5.3. Compaction Specifications

SPmdd is the specification to indicate the degree to which soil or aggregate is compacted. To achieve the specified SPmdd as indicated in this report, all soils or aggregates must be placed in lift thicknesses no greater than 200 mm. If this is not the case, only the upper portion of the lift will be adequately compacted, and the lower portion of the lift has a high probability of not meeting compaction specifications. In addition, industry standard equipment used to determine the degree of compaction consists of nuclear densometers. These devices have an inherent limitation in that they cannot test beyond 300 mm in depth, and so the degree of compaction beyond this depth cannot be quantitatively determined.

Along with lift thickness, ensuring that the soil or aggregate is within 2% of its optimum moisture content ensures that the specified compaction can be reached. If the soil or aggregate is too dry/wet, it is either very difficult or impossible to reach the specified compaction. This is especially true for when higher compaction specifications such as 98% and 100% SPmdd are required.

Moisture can be increased by adding water and mixing the soil prior to re-use, blending the soil with wetter material, or by importing soil to the site that is at optimum and can be readily compacted.

Moisture can be reduced by tilling or spreading out the soil to dry or blending it with drier material. In-situ moisture contents can change based on the season and local groundwater levels and can also change for stockpiled material due to precipitation. Zones of the fine-grained soil with very high moisture contents may fine moisture conditioning to be difficult to accomplish.

In addition to the above compaction specifications, in any areas where compacted fill will be placed over the exposed native soil subgrade, any loose, soft, wet, organic or unstable areas should be sub-excavated, and backfilled with clean earth fill or Granular ‘B’ (OPSS.MUNI 1010) compacted to a minimum of 95% SPmdd. This recommendation applies to site servicing and pavement subgrades. Where structures/buildings require upfilling beneath the structure the fill should be compacted to 100% SPmdd.

5.4. Quality Verification Services

On-site quality verification services are an integral part of the geotechnical design function, and for foundations, engineered fill and retaining walls, are required under the Ontario Building Code. Quality verification services are used to confirm that construction is being conducted in general conformance with the requirements as outlined in the drawings, reports and specifications prepared for the proposed development.

GEI can provide all the on-site quality verification services outlined below:

- The subgrade for shallow foundations for the proposed buildings will need to be field reviewed by the geotechnical engineer.
- Installation of retaining structures over 1.0 m high and related backfilling operations must be field reviewed on a continuous basis by the geotechnical engineer as required in the OBC.
- Full-time monitoring, testing and inspection of engineered fill placement is required by the geotechnical engineer per the OBC.
- Part-time monitoring of the subgrade support capabilities, material quality, lift thickness, moisture content, degree of compaction, etc. is recommended for the following areas to ensure the recommendations within this report are followed and they perform adequately in the long-term;
  - Slab-on-grades;
  - Pavement structure (granular and asphalt);
  - Pipe bedding and cover;
- Testing of the concrete (compressive strength, slump, air content, etc.) and testing of the asphalt (asphalt content and gradation) are recommended to ensure that the quality of the materials being brought to site meet the requirements of the project.

6. Limitations and Conclusions

6.1. Limitations

The recommendations and comments provided are necessarily on-going as new information of underground conditions becomes available. More specific information with respect to the conditions between samples, or the lateral and vertical extent of materials may become apparent during excavation operations. The interpretation of the borehole information must, therefore, be validated during excavation operations. Consequently, conditions not observed during this investigation may become apparent. Should this occur, GEI should be contacted to assess the situation and additional testing and reporting may be required.

GEI should be retained for a general review of the final design drawings and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege of making this review, GEI will assume no responsibility for interpretation of the recommendations in the report.

The comments given in this report are intended only for the guidance of the design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. could be greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

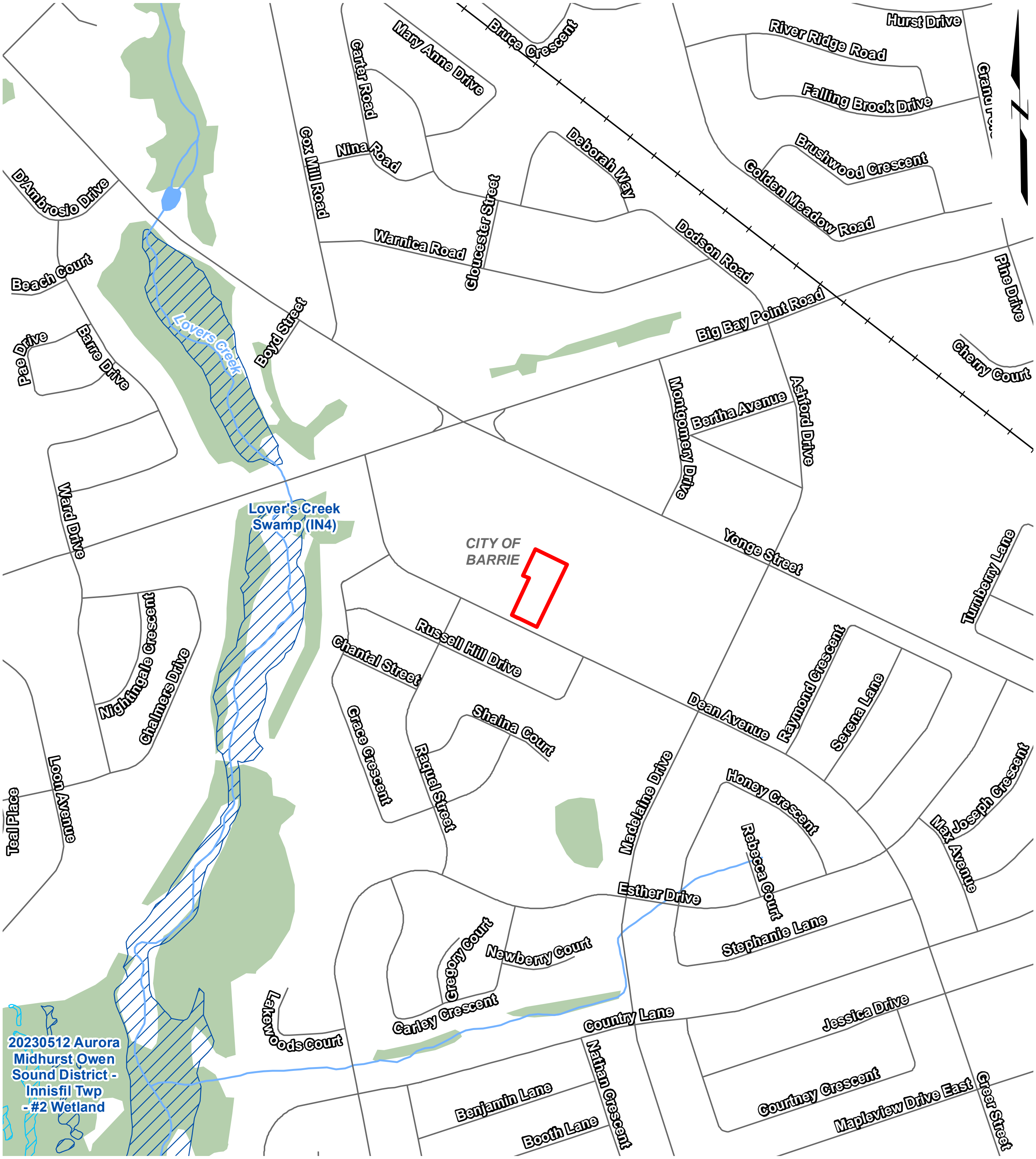
This report was authorized by, and prepared by GEI for, the account of the Nestwise Inc. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. GEI accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this project.

6.2. Conclusions

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

We trust this report is complete within our terms of reference, and the information presented is sufficient for your present purposes. If you have any questions, or when we may be of further assistance, please do not hesitate to contact our office.

Figures



🟥 Subject Lands

— Road

┼ Railway

— Watercourse

🟦 Waterbody

▨ Provincially Significant Wetland

▨ Locally/Other Significant Wetland

🟩 Wooded Area

Reference(s):
1. Coordinate System: NAD 1983 CSRS UTM Zone 17N.
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © King's Printer for Ontario, 2025.

0 120 240
Metres

Geotechnical Investigation
Proposed Residential Building, 60 Dean Ave
Barrie, Ontario

Nestwise Inc.

GEI Consultants

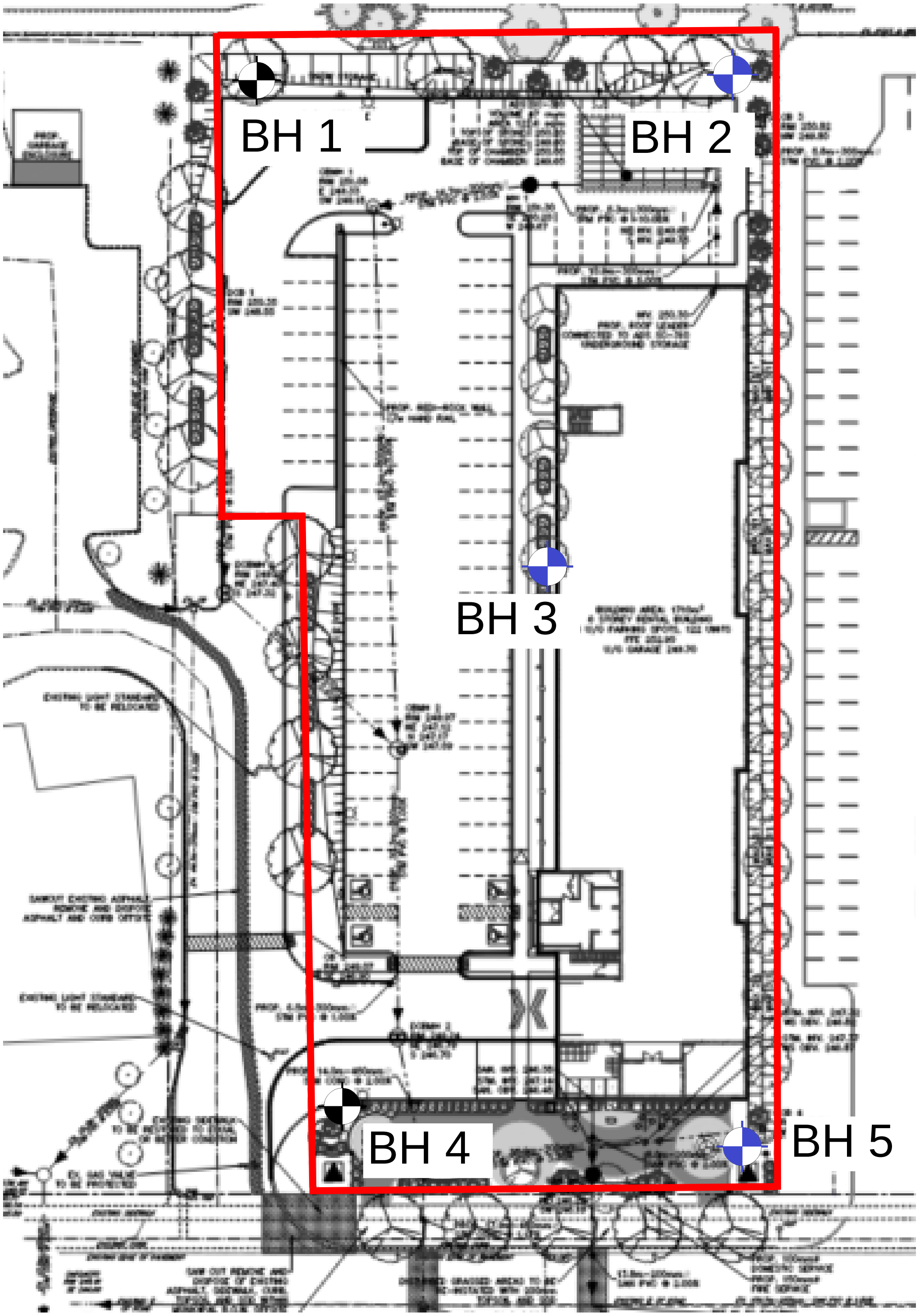
SITE LOCATION PLAN

Project 2408904

Feb 2025

Fig. 1

\\bos-pzcc-1\Data_Storage\Working\HANSEN GROUP\2408904 Proposed Residential Building, 48 Dean Ave, Barrie ON\05_GIS\MXD\2025-01_GeoTech\2408904_rpt_fig01_Site_Location.mxd



🟥 Subject Lands
— Road
⊕ Approximate Borehole Location
⊕ Approximate Borehole/Monitoring Well Location

Reference(s):
1. Coordinate System: NAD 1983 CSRS UTM Zone 17N.
2. Base features produced under license with the Ontario Ministry of Natural Resources and Forestry © King's Printer for Ontario, 2025.
3. Site Plan © Jones Consulting Group Ltd., July 12, 2024.

0 8.5 17
Metres

Geotechnical Investigation
Proposed Residential Building, 60 Dean Ave
Barrie, Ontario

Nestwise Inc.

GEI Consultants

Project 2408904

BOREHOLE LOCATION PLAN

Feb 2025

Fig. 2

\\bos-pzcc-1\Data_Storage\Working\HANSEN GROUP\2408904 Proposed Residential Building, 48 Dean Ave, Barrie ON\05_GIS\MXD\2025-01_GeoTech\2408904_rpt_fig02_BH_Location_Plan.mxd

Appendix A Borehole Logs

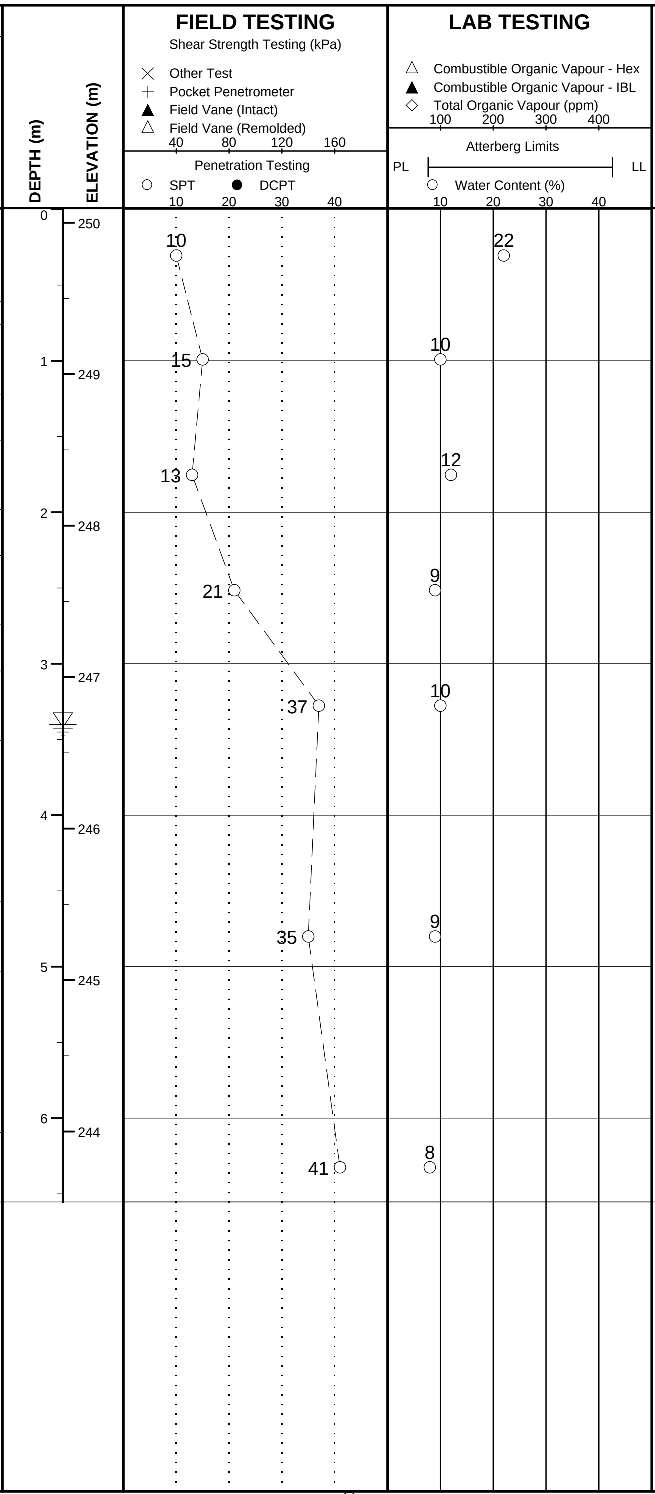
RECORD OF BOREHOLE No. 01

GEI Consultants

Project Number: **2408904**
Project Client: **Hansen Group Inc.**
Project Name: **60 Dean Ave.**
Project Location: **Barrie, ON**
Drilling Location: **See Borehole Location Plan**
Local Benchmark:

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Rubber Tire**
Logged By: **BH**
Reviewed By: **MH**
Northing: **4912185**
Easting: **607859**
Date Started: **Dec 18/24**
Date Completed: **Dec 18/24**

LITHOLOGY PROFILE		SOIL SAMPLING				LAB TESTING	COMMENTS & GRAIN SIZE DISTRIBUTION (%)			
Depth / Elevation	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value	Water Content (%)	GR	SA	SI	CL
0.0 / 250.1	TOPSOIL: 760 mm	SS	1	100	10	22				
0.8 / 249.3	FILL: Silty sand, trace gravel, compact, brown, moist	SS	2	100	15	10				
1.5 / 248.6	SILTY SAND GLACIAL TILL: Some clay trace gravel, inferred cobbles and boulders, compact to dense, brown, moist	SS	3	100	13	12				
		SS	4	100	21	9	6	53	31	10
	- - - Wet seams - - -	SS	5	100	37	10				
		SS	6	100	35	9				
6.6 / 243.5		SS	7	100	41	8				
	Borehole Terminated at 6.6 m									



Instrumentation Installation

Groundwater depth encountered on completion of drilling: 3.4 m.
Cave depth after auger removal: Open
Groundwater depth observed on:
Groundwater Elevation:

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Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

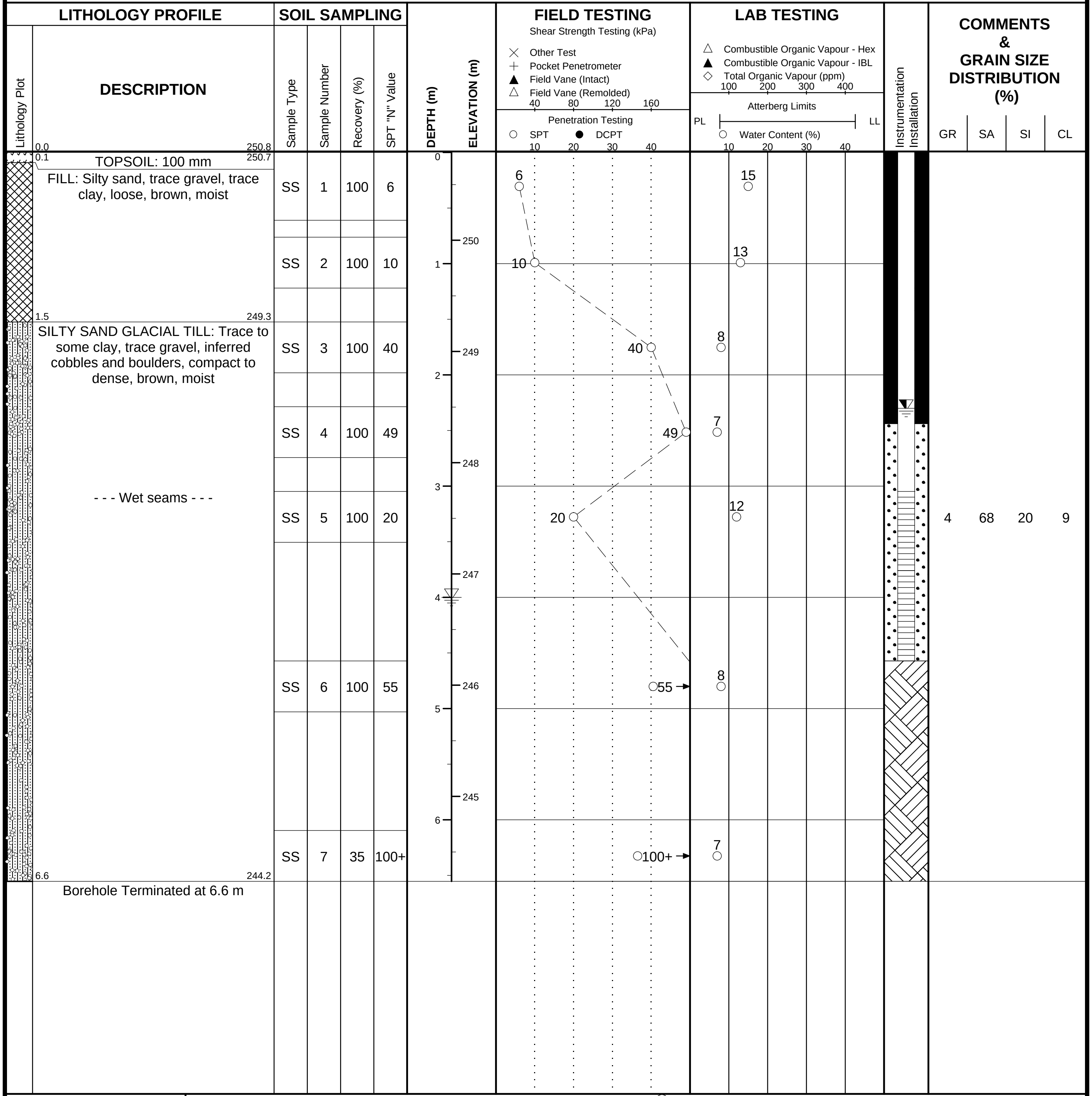
Scale: **1 :50**
Page: **1 of 1**

RECORD OF BOREHOLE No. 02

GEI Consultants

Project Number: **2408904**
Project Client: **Hansen Group Inc.**
Project Name: **60 Dean Ave.**
Project Location: **Barrie, ON**
Drilling Location: **See Borehole Location Plan**
Local Benchmark:

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Rubber Tire**
Logged By: **BH**
Reviewed By: **MH**
Northing: **4912185**
Easting: **607907**
Date Started: **Dec 18/24**
Date Completed: **Dec 18/24**



Depth (m)	Elevation (m)	Description	Sample Type	Sample Number	Recovery (%)	SPT "N" Value	Water Content (%)	GR	SA	SI	CL
0.0	250.8										
0.1	250.7	TOPSOIL: 100 mm									
		FILL: Silty sand, trace gravel, trace clay, loose, brown, moist	SS	1	100	6	15				
			SS	2	100	10	13				
1.5	249.3	SILTY SAND GLACIAL TILL: Trace to some clay, trace gravel, inferred cobbles and boulders, compact to dense, brown, moist	SS	3	100	40	8				
			SS	4	100	49	7				
		- - - Wet seams - - -	SS	5	100	20	12	4	68	20	9
			SS	6	100	55	8				
			SS	7	35	100+	7				
6.6	244.2	Borehole Terminated at 6.6 m									

Groundwater depth encountered on completion of drilling: 4.0 m.
Cave depth after auger removal: Open
Groundwater depth observed on: Jan 08/25 at depth of: 2.3 m.
Groundwater Elevation: 248.5 m

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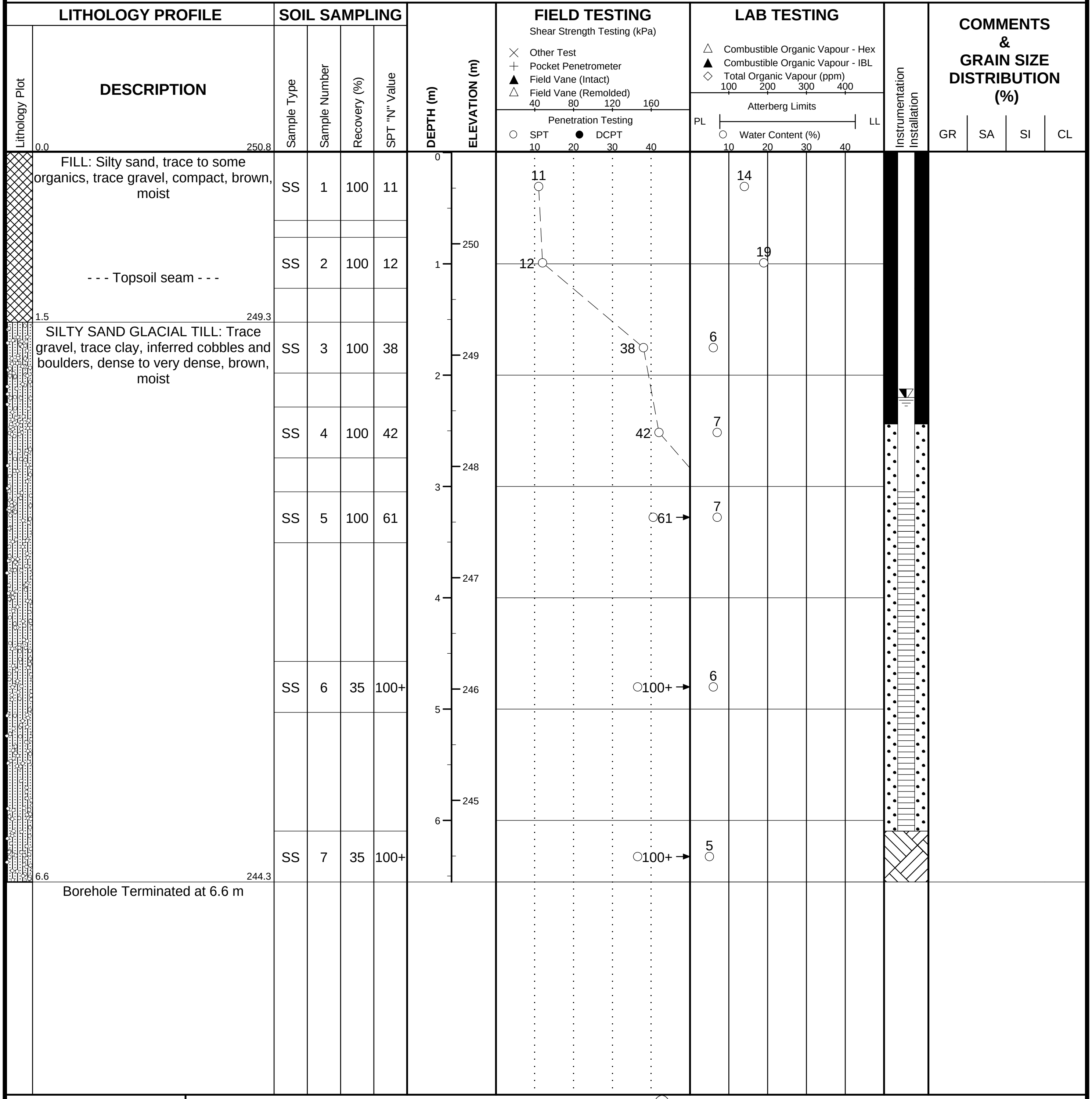
Scale: **1 :50**

RECORD OF BOREHOLE No. 03

Project Number: **2408904**
Project Client: **Hansen Group Inc.**
Project Name: **60 Dean Ave.**
Project Location: **Barrie, ON**
Drilling Location: **See Borehole Location Plan**
Local Benchmark:

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Rubber Tire**
Logged By: **BH**
Reviewed By: **MH**
Northing: **4912123**
Easting: **607866**
Date Started: **Dec 18/24**
Date Completed: **Dec 18/24**



Depth / Elevation (m)	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value	Water Content (%)
0.0 / 250.8	FILL: Silty sand, trace to some organics, trace gravel, compact, brown, moist	SS	1	100	11	14
	- - - Topsoil seam - - -	SS	2	100	12	19
1.5 / 249.3	SILTY SAND GLACIAL TILL: Trace gravel, trace clay, inferred cobbles and boulders, dense to very dense, brown, moist	SS	3	100	38	6
		SS	4	100	42	7
		SS	5	100	61	7
		SS	6	35	100+	6
6.6 / 244.3		SS	7	35	100+	5

Borehole Terminated at 6.6 m

Groundwater depth encountered on completion of drilling: Dry
Cave depth after auger removal: Open
Groundwater depth observed on: Jan 08/25 at depth of: 2.2 m.
Groundwater Elevation: 248.6 m

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Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: **1 :50**

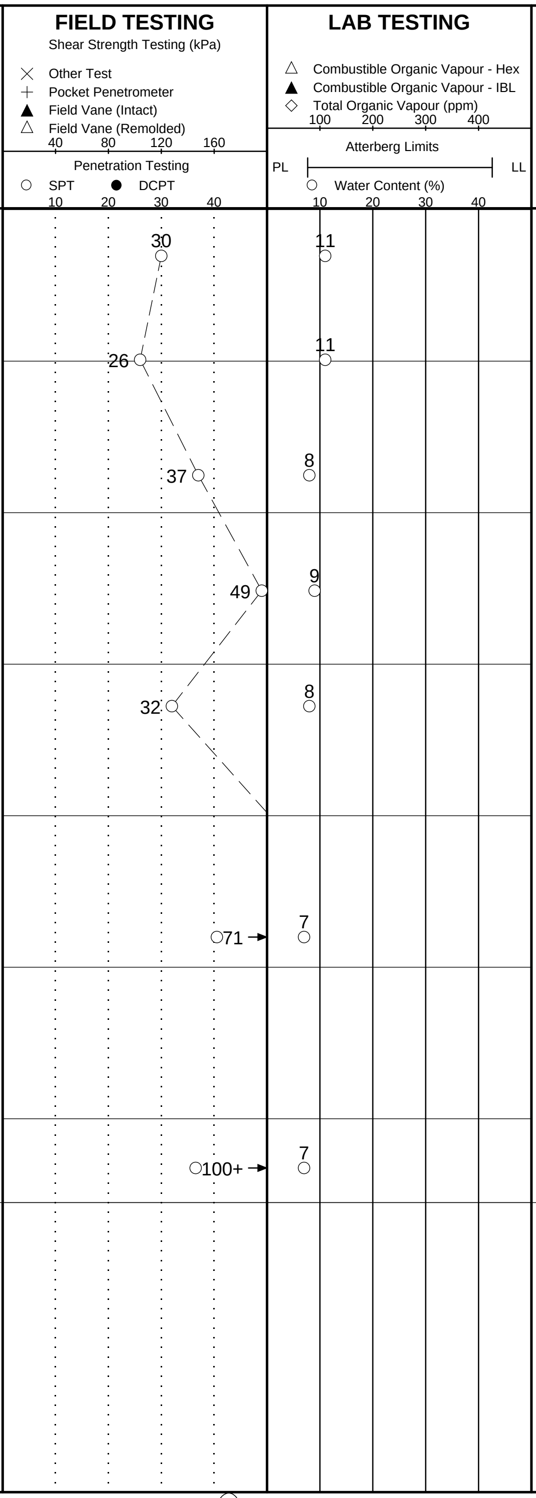
RECORD OF BOREHOLE No. 04

GEI Consultants

Project Number: **2408904**
Project Client: **Hansen Group Inc.**
Project Name: **60 Dean Ave.**
Project Location: **Barrie, ON**
Drilling Location: **See Borehole Location Plan**
Local Benchmark:

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Rubber Tire**
Logged By: **BH**
Reviewed By: **MH**
Northing: **4912080**
Easting: **607826**
Date Started: **Dec 18/24**
Date Completed: **Dec 18/24**

Depth (m)	Elevation (m)	LITHOLOGY PROFILE – DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value	Water Content (%)	GR	SA	SI	CL
0.0	249.1										
0.2	249.0	TOPSOIL: 150 mm									
		FILL: Silty sand, trace gravel, compact, brown, moist	SS	1	100	30	11				
0.8	248.4	SILTY SAND GLACIAL TILL: Trace clay, trace gravel, inferred cobbles and boulders, compact to very dense, brown, moist	SS	2	100	26	11				
			SS	3	100	37	8	8	55	29	8
			SS	4	100	49	9				
		- - - Wet seams - - -	SS	5	100	32	8				
			SS	6	100	71	7				
6.6	242.6		SS	7	35	100+	7				
		Borehole Terminated at 6.6 m									



Instrumentation Installation

Groundwater depth encountered on completion of drilling: Dry
Groundwater depth observed on:
Cave depth after auger removal: Open
Groundwater Elevation:

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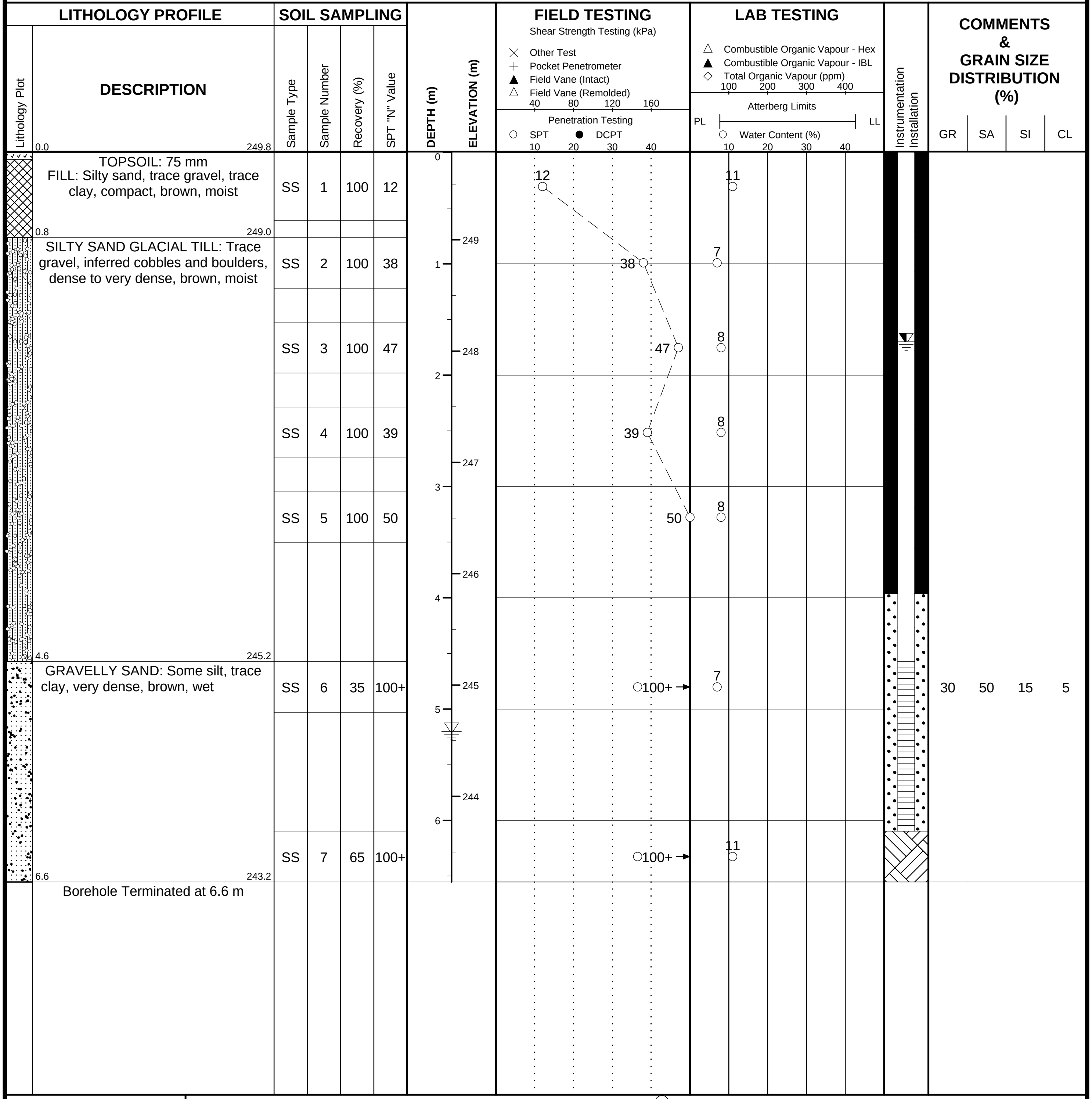
Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: **1 :50**

RECORD OF BOREHOLE No. 05

Project Number: **2408904**
Project Client: **Hansen Group Inc.**
Project Name: **60 Dean Ave.**
Project Location: **Barrie, ON**
Drilling Location: **See Borehole Location Plan**
Local Benchmark:

Drilling Method: **Solid Stem Augers**
Drilling Machine: **Rubber Tire**
Logged By: **BH**
Reviewed By: **MH**
Northing: **4912059**
Easting: **607861**
Date Started: **Dec 18/24**
Date Completed: **Dec 18/24**



Depth (m) / Elevation (m)	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value	Water Content (%)	GR	SA	SI	CL
0.0 / 249.8	TOPSOIL: 75 mm FILL: Silty sand, trace gravel, trace clay, compact, brown, moist	SS	1	100	12	11				
0.8 / 249.0	SILTY SAND GLACIAL TILL: Trace gravel, inferred cobbles and boulders, dense to very dense, brown, moist	SS	2	100	38	7				
		SS	3	100	47	8				
		SS	4	100	39	8				
		SS	5	100	50	8				
4.6 / 245.2	GRAVELLY SAND: Some silt, trace clay, very dense, brown, wet	SS	6	35	100+	7	30	50	15	5
		SS	7	65	100+	11				
6.6 / 243.2	Borehole Terminated at 6.6 m									

Groundwater depth encountered on completion of drilling: 5.2 m.

Cave depth after auger removal: Open

Groundwater depth observed on: Jan 08/25 at depth of: 1.7 m.

Groundwater Elevation: 248.1 m

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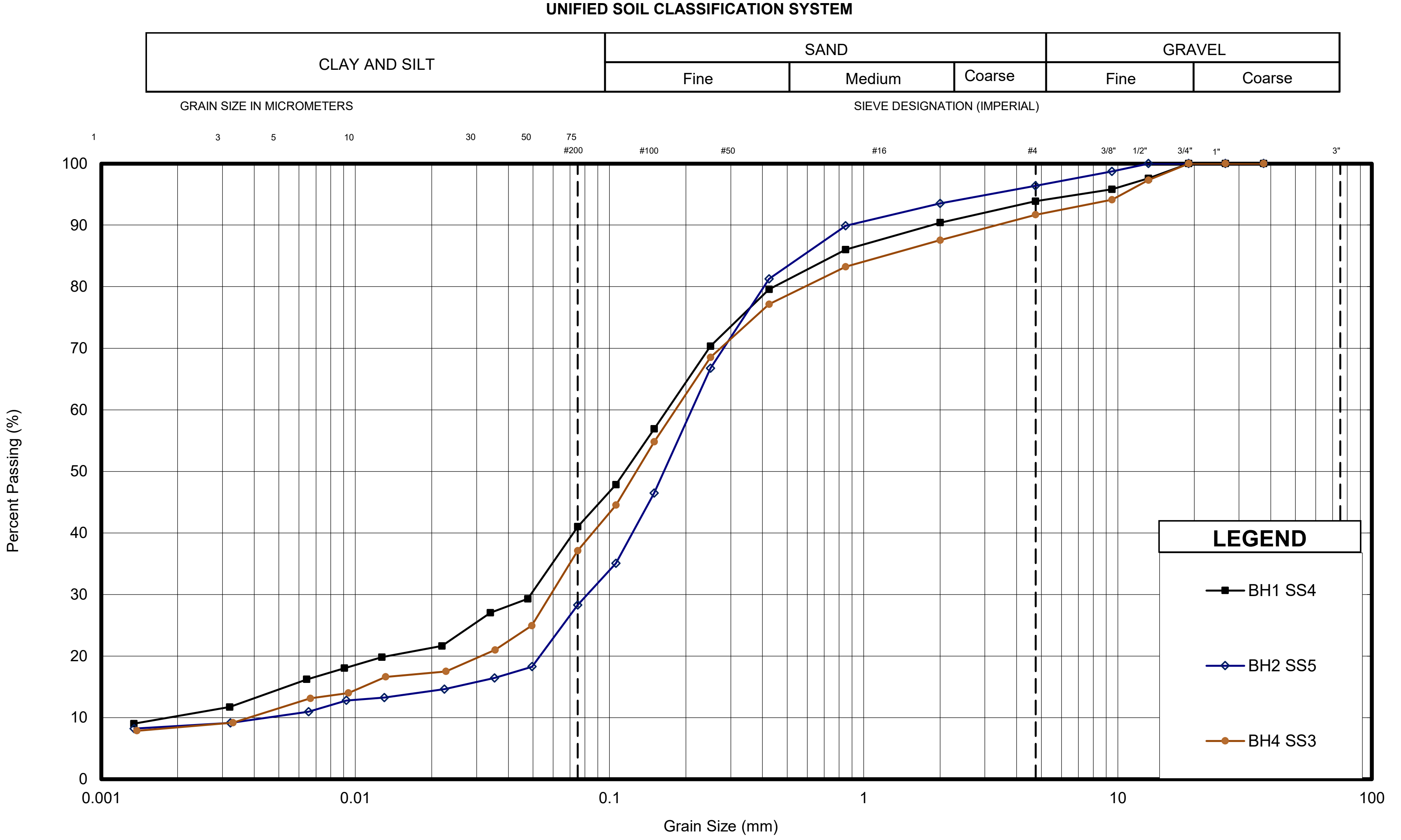
Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: **1 :50**

Appendix B Geotechnical Laboratory Data



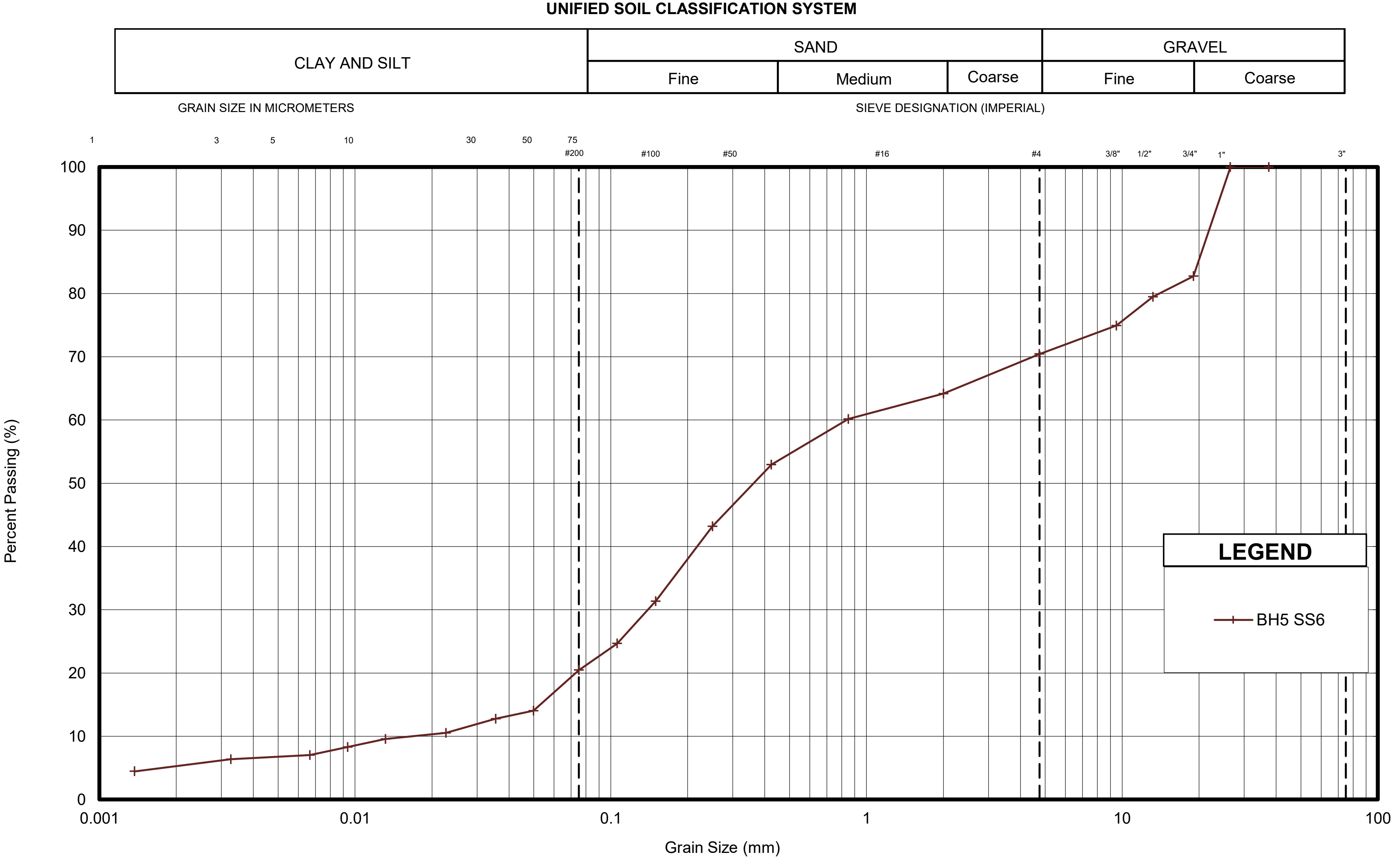
Sample	Description	Gr.	Sa.	Si.	Cl.	D_{10}	D_{30}	D_{60}	C_u	C_c
BH1 SS4	SILTY SAND, Some Clay, Trace Gravel	6	53	31	10	0.002	0.049	0.169	91.4	7.7
BH2 SS5	SILTY SAND, Trace Clay, Trace Gravel	4	68	20	9	0.005	0.082	0.211	46.7	7.0
BH4 SS3	SILTY SAND, Trace Clay, Trace Gravel	8	55	29	8	0.004	0.059	0.182	47.7	5.0

GEI Consultants

GRAIN SIZE DISTRIBUTION - 60 Dean Ave - Pratt Hansen

GLACIAL TILL

FIGURE No. B1
REF. No. 2408904
DATE Feb 2025



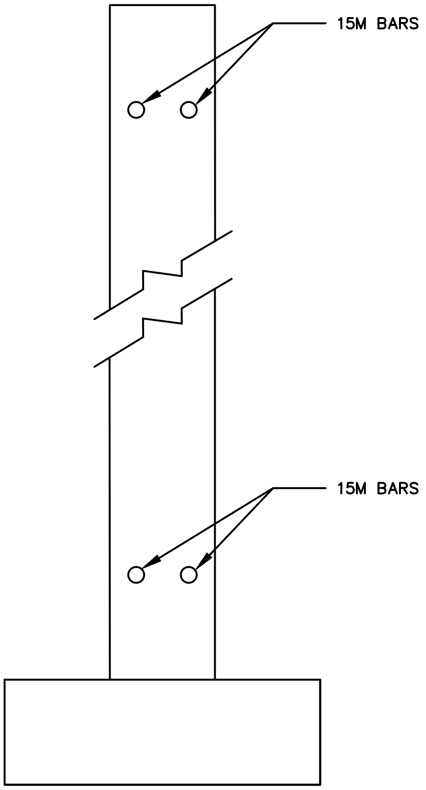
Sample	Description	Gr.	Sa.	Si.	Cl.	D_{10}	D_{30}	D_{60}	C_u	C_c
BH5 SS6	GRAVELLY SAND, Some Silt, Trace Clay	30	50	15	5	0.017	0.140	0.838	49.8	1.4

GRAIN SIZE DISTRIBUTION - 60 Dean Ave - Pratt Hansen

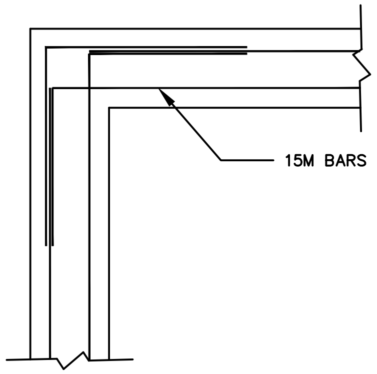
GRAVELLY SAND

FIGURE No.	B2
REF. No.	2408904
DATE	Feb 2025

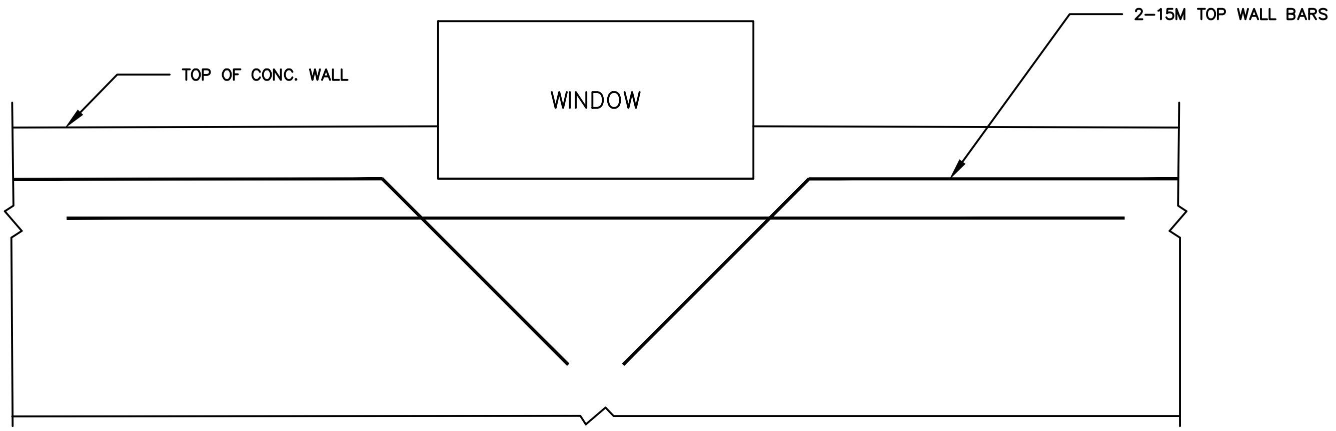
Appendix C Typical Details



TYPICAL REINFORCED WALL



TYPICAL SPLICING AT CORNERS



TYPICAL WINDOW REINFORCING

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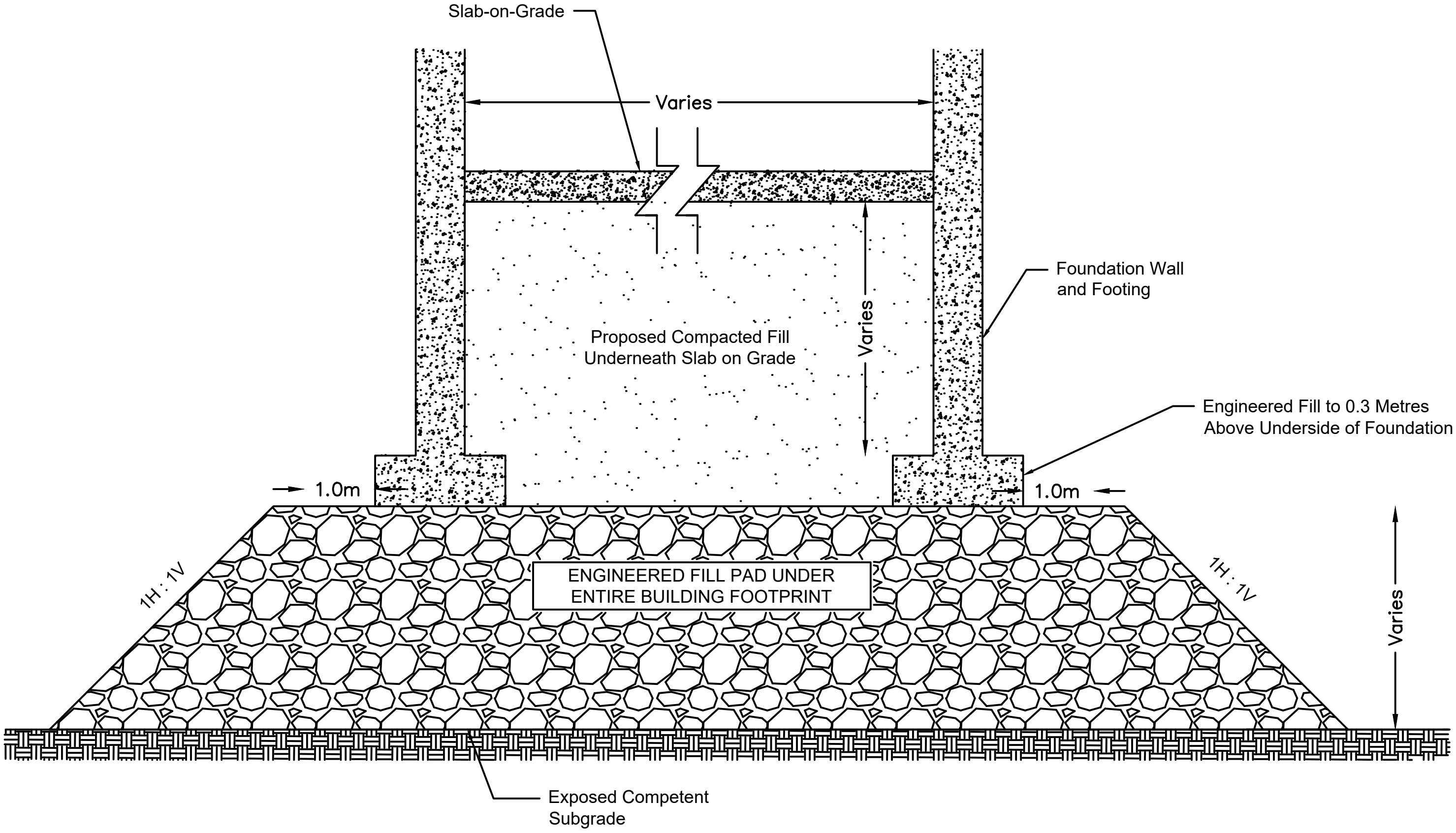
FOUNDATION WALL REINFORCING ON ENGINEERED FILL TYPICAL DETAIL

Notes:

1. Engineered Fill compacted to 100% Standard Proctor Maximum Dry Density (SPMDD) and inspected under the full time supervision of GEI.
2. Engineered fill must be placed in loose lifts of 200 mm or less and then compacted as noted above.
3. Interior non-structural compacted fill compacted to 98% SPMDD with recommended part-time inspection.



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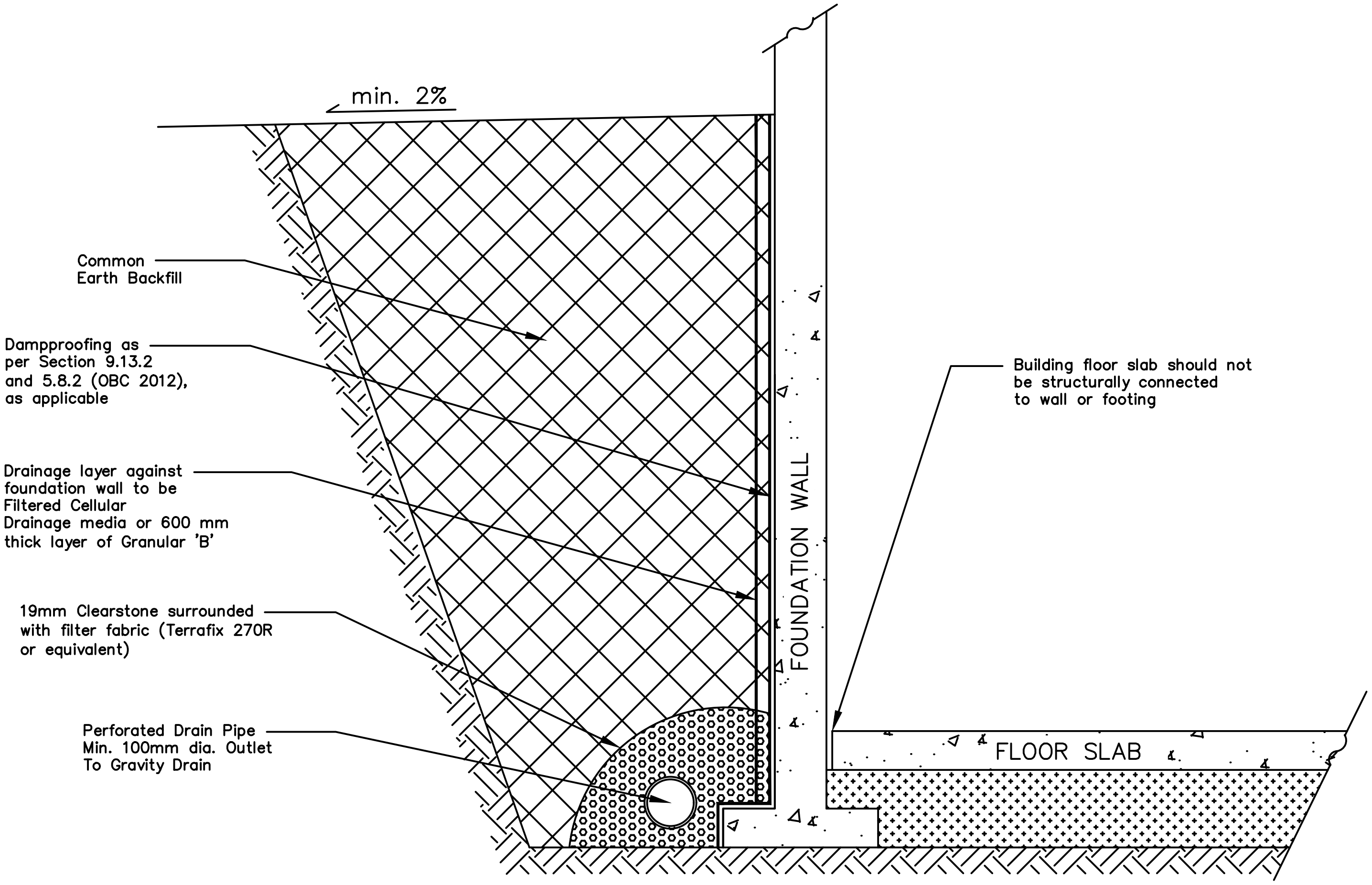
ENGINEERED FILL PAD DIMENSIONING TYPICAL DETAIL



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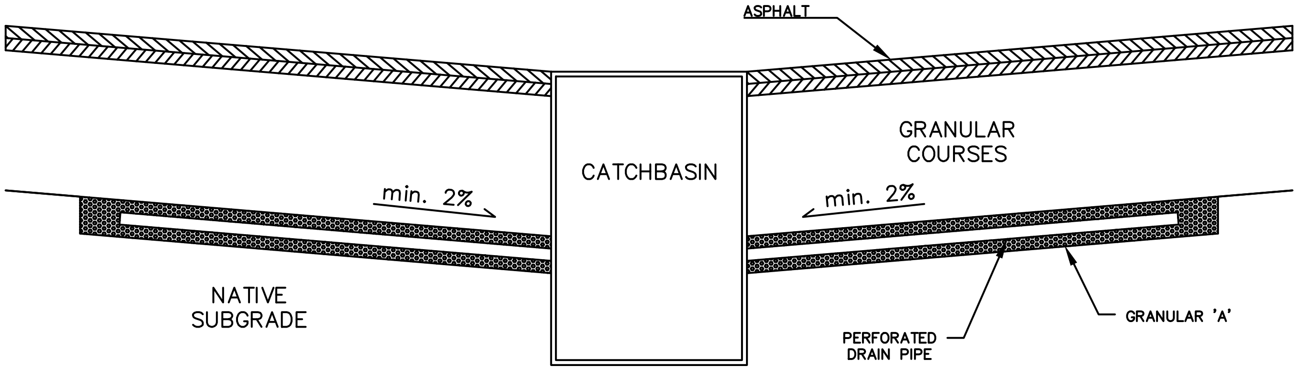
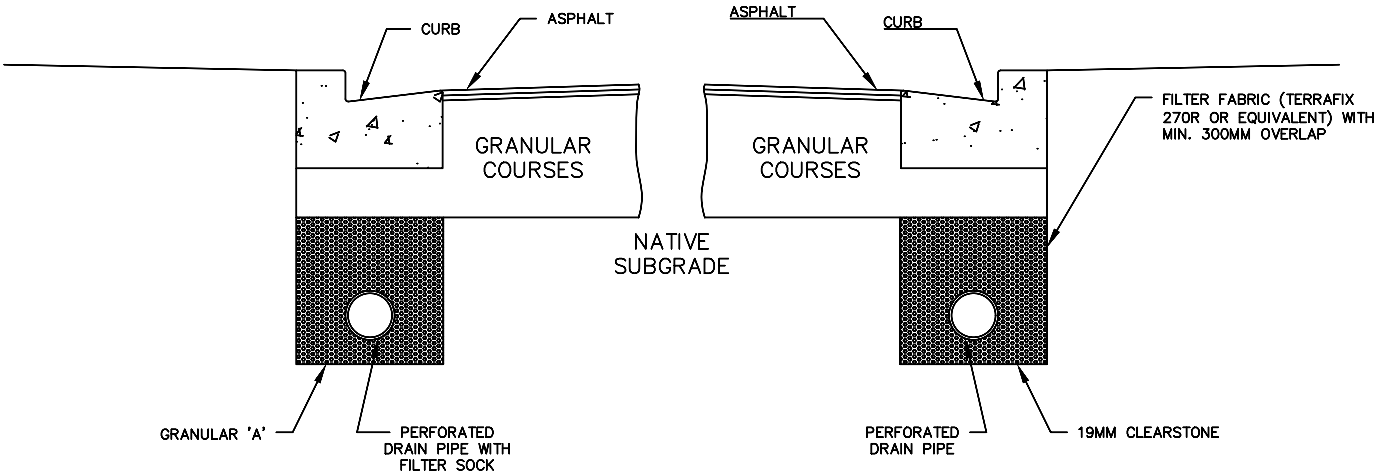
BASEMENT FOUNDATION WALL TYPICAL DETAIL

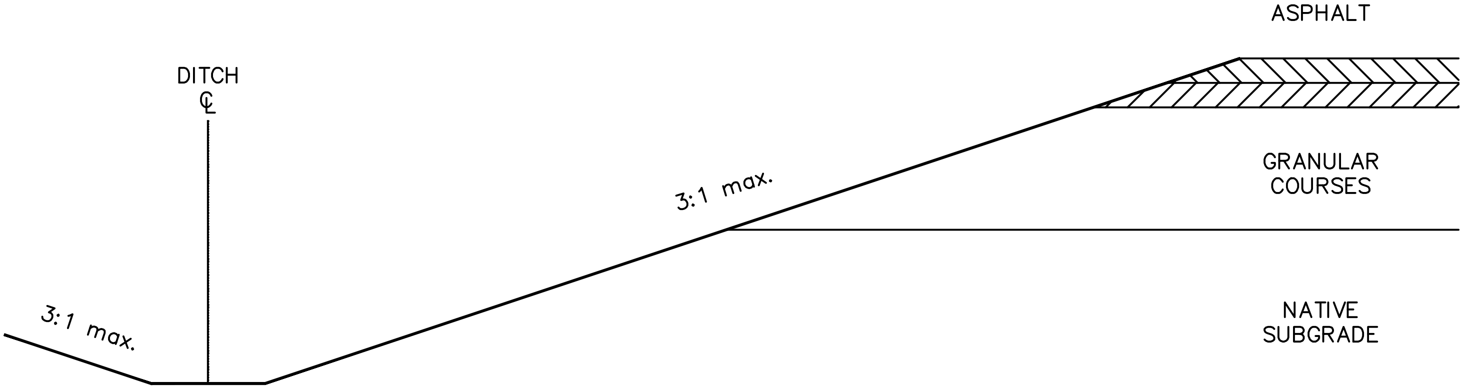




Unwrapped Trench

Wrapped Trench

Urban Cross Sections



Rural Cross Section

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PAVEMENT DRAINAGE ALTERNATIVES TYPICAL DETAILS

Appendix D MASW Testing