Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL (416) 754-8515 · FAX (905) 881-8335

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	GRAVENHURST	PETERBOROUGH	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 440-2040	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 725-1315	FAX: (905) 542-2769

A REPORT TO
DREAMWOOD DEVELOPMENTS INC.

A GEOTECHNICAL INVESTIGATION FOR PROPOSED 4-STOREY APARTMENT BUILDING WITH 1-LEVEL UNDERGROUND PARKING GARAGE

60-90 RIVER ROAD EAST

TOWN OF WASAGA BEACH

REFERENCE NO. 1707-C037

AUGUST 2017

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1.0 INTRODUCTION

In accordance with instructions from Mr. Matthew Marsili, Project Manager of Dreamwood Developments Inc., and Purchase Order No. PO000008 dated June 28, 2017, an update to the recommendations presented in the original Geotechnical Investigation Report, Reference No. 1306-S162 dated August 2013, is carried out to take into account the proposed new development.

The soil investigation was carried out on July 15, 2013 at a parcel of land located northeast of Beck Street and River Road East having a municipality address of 60-90 River Street East in the Town of Wasaga Beach, for a proposed Residential Development.

The purpose of the investigation was to reveal the subsurface conditions and to determine the bank stability along the north boundary limit of the development abutting Nottawasaga River, as well as to reveal the engineering properties of the disclosed soils for the design and construction of the proposed development.

The resulting geotechnical findings and recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The site is situated on a bluff on the Nottawasaga basin where glacial Lake Nipissing previously extended. The stratigraphy consists of sand derived from outwash of the Edenvale Moraine and fluvial deposits of Lake Nipissing and the present Nottawasaga River.

The property is rectangular in shape and currently consists of a vacant lot with weed cover and scattered trees. It is bounded by River Road East at the south, Nottawasaga River to the north, and residential lots to the east and west. The ground surface is slightly undulated and generally descends to the north, extending to a valley bank at the north limit of the property. The bank is approximately 3.5± m in height with a gradient of 1 vertical:5± horizontal. The bank face is generally vegetated with weed cover, scattered trees and bushes. No signs of seepage or deep-seated failure were observed on the bank face. A concrete retaining wall extends at the toe of the slope along the river shoreline.

It is understood that the proposed development will consist of a 4-storey apartment building with one level of underground parking. It will also be provided with on-grade parking, landscaping, and municipal water and sewer services meeting urban standards.

3.0 FIELD WORK

The field work, consisting of 6 boreholes to a depth of 6.6 m, was performed on July 15, 2013, at the locations shown on the Borehole and Cross-Section Location Plan, Drawing No. 1.

The holes were advanced at intervals to the sampling depths by a track-mounted, continuous-flight power-auger machine equipped for soil sampling. Standard Penetration Tests, using the procedures described on the enclosed “List of Abbreviations and Terms”, were performed at the sampling depths. The test results are recorded as the Standard Penetration Resistance (or ‘N’ values) of the subsoil. The relative density of the granular strata and the consistency of the cohesive strata are inferred from the ‘N’ values. Split-spoon samples were recovered for soil classification and laboratory testing.

The field work was supervised and the findings recorded by a Geotechnical Technician.

The elevation at each of the borehole locations was determined with reference to a site bench mark shown on Drawing No. 1, which is the top of the catch basin at the north side of River Road East. It has a geodetic elevation of 180.85 m.

4.0 SUBSURFACE CONDITIONS

Detailed descriptions of the encountered subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 6, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2, and the engineering properties of the disclosed soils are discussed herein.

Beneath a veneer of topsoil in one location, or a layer of sand fill, the site is generally underlain by a stratum of fine sand extending to at least the maximum investigated depth in all boreholes. A layer of silty fine sand was found embedded within the fine sand stratum in one location.

4.1 **Topsoil** (Borehole 1)

The topsoil veneer was 23 cm in thickness. The topsoil is dark brown in colour, indicating that it contains appreciable amounts of roots and humus. These materials are unstable under loads and highly compressible, rendering the topsoil unsuitable for engineering applications. The topsoil can only be used for normal landscaping and landscape contouring purposes. Prior to using the topsoil fill as a planting material, it should be assessed by a fertility analysis.

Due to its humus content, the topsoil may produce volatile gases and will generate an offensive odour under anaerobic conditions. Therefore, it must not be buried within the building envelope, or deeper than 1.2 m below the exterior finished grade. This is to avoid an adverse impact on the environmental well-being of the project.

4.2 **Sand Fill** (All Boreholes, except Borehole 1)

The fill generally consists of fine sand. It contains topsoil inclusions and occasional wood, concrete and asphalt debris. The fill extends to depths ranging from 0.8 to 2.3 m below the prevailing ground surface.

The water content of the samples was determined, and the results are plotted on the Borehole Logs; the values range from 3% to 33%, with a median of 4%, indicating that the fill is in a moist to saturated, generally moist condition. The high water content of 33% at Borehole 2 is likely due to the topsoil inclusions and organic debris found in the fill.

The obtained 'N' values range from 2 to 11, with a median of 3 blows per 30 cm of penetration, showing the fill was loosely placed. Due to the presence of topsoil inclusions and other deleterious material, and its loose density, the fill is considered unsuitable for bearing foundations unless it is sorted free of deleterious material and structurally recompacted.

A grain size analysis was performed on 1 representative sample; the result is plotted on Figure 7.

In excavation, the generally moist sand fill will slough to its angle of repose. Where the fill is free of deleterious materials, its engineering properties are generally similar to those of the fine sand which is described in the following section.

One must be aware that the samples retrieved from boreholes, 10 cm in diameter, may not be truly representative of the geotechnical and environmental quality of the fill, and do not indicate whether the topsoil beneath the earth fill was completely stripped. This should be further assessed by laboratory testing and/or test pits.

4.3 **Fine Sand** (All Boreholes)

The fine sand deposit dominates the soil stratigraphy and was found extending to the maximum investigated depth in all boreholes. The sand contains a trace to some silt. Sample examinations indicate it is laminated with occasional silt and silty fine sand seams and layers, with a distinct silty fine sand layer disclosed in Borehole 1 at a depth of 4.5 m below the prevailing ground surface.

Sample examinations show that the sand is non-cohesive and is found to be water bearing at depths ranging from 3.7 to 4.3 m below the prevailing ground surface.

The natural water content was determined, and the results are plotted on the Borehole Logs; the values range from 4% to 24%, with a median of 6%, indicating that it is in a moist to saturated condition, being generally moist. The saturated condition of the sand samples show that the sand is water bearing.

The obtained ‘N’ values range from 4 to 47, with a median of 23, indicating that its relative density is loose to dense, being generally compact below 1.5 to 2.3 m from grade. The loose sand is generally restricted to the weathered zone of the deposit which extends to depths ranging from 1.5 to 2.3 m below the prevailing ground surface.

Grain size analyses were performed on 2 representative samples of the fine sand, and one sample of the embedded silty fine sand layer; the results are plotted on Figures 8 and 9, respectively.

Based on the above findings, the following engineering properties of the fine sand are deduced:

- Low frost susceptibility and soil-adfreezing potential.
- High water erodibility; it is susceptible to migration through small openings under seepage pressure.
- Pervious, with an estimated coefficient of permeability of 10^{-2} cm/sec, an estimated percolation rate of 10 to 20 min/cm and runoff coefficients of:

Slope	
0% - 2%	0.04
2% - 6%	0.09
6% +	0.13

- A frictional soil, its shear strength is derived from internal friction and is soil density dependent.
- In cuts, the moist sand will slough to its angle of repose. However, the saturated sand will run with water seepage and boil under a piezometric head of about 0.4 m.
- A good pavement-supportive material, with an estimated California Bearing Ratio value of 20%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 6000 ohm·cm.

4.4 Compaction Characteristics of the Revealed Soils

The obtainable degree of compaction is primarily dependent on the soil moisture and, to a lesser extent, on the type of compactor used and the effort applied.

As a general guide, the typical water content values of the revealed soils for Standard Proctor compaction are presented in Table 1.

Table 1 - Estimated Water Content for Compaction

Soil Type	Determined Natural Water Content (%)	Water Content (%) for Standard Proctor Compaction	
		100% (optimum)	Range for 95% or +
Sand Fill	3 to 33 (median 4)	11	5 to 16
Fine Sand	4 to 24 (median 6)	11	5 to 16
Silty Fine Sand	22	11	6 to 16

The above findings show that the sand fill and fine sand are generally suitable for a 95% or + Standard Proctor compaction, provided that the sand fill is sorted free of serious topsoil inclusions and other deleterious materials prior to its use as structural fill. The silty fine sand and a portion of the sand fill are wet and will require mixing with drier soils and/or aeration prior to structural compaction. The wet sands can be stockpiled on the site to drain the excess water before placement and compaction, or they can be spread thinly on the ground in the dry, warm weather for aeration.

The sands can be compacted by a smooth drum roller with or without vibration, depending on the water content of the soils being compacted. The degree of vibration must be adjusted inversely to the moisture content of the soils being compacted. The lifts for compaction should be limited to 20 cm, or to a suitable thickness assessed by test strips performed by the equipment which will be used at the time of construction.

One should be aware that a 90%± Standard Proctor compaction of the wet fine sand and silty fine sand is achievable at the time of compaction. Further densification is prevented by the pore pressure induced by the compactive effort; however, large random voids will have been expelled and, with time, the pore pressure will dissipate and the percentage of compaction will increase. There are many cases on record where after a period of weeks to months of rest, the density of the compacted mantle has increased to over 95% of its maximum Standard Proctor dry density.

5.0 GROUNDWATER CONDITIONS

Groundwater seepage encountered during augering was recorded on the field logs. The boreholes were checked for the presence of groundwater and the occurrence of cave-in upon their completion. The data are plotted on the Borehole Logs and listed in Table 2.

Table 2 - Groundwater Levels

		Soil Colour Changes Brown to Grey	**Seepage Encountered During Augering**		**Measured Groundwater/Cave-in* Level On Completion**	
BH No.	**Borehole Depth (m)**	**Depth (m)**	**Depth (m)**	**Amount**	**Depth (m)**	**El. (m)**
1	6.6	5.5	3.5	Appreciable	3.7*	177.3*
2	6.6	5.5	4.5	Appreciable	4.3*	176.8*
3	6.6	5.5	4.5	Appreciable	4.3*	177.2*
4	6.6	5.5	4.5	Appreciable	4.3*	177.3*
5	6.6	5.5	3.5	Appreciable	3.7*	177.3*
6	6.6	5.5	4.5	Appreciable	4.0*	177.1*

*Cave-in level (In wet sand, the level generally represents the groundwater regime at the time of investigation.)

All boreholes caved at depths ranging from 3.7 to 4.3 m (El. 176.8 to 177.3 m) below the prevailing ground surface which generally represents the groundwater regime of the site at the time of investigation. The groundwater levels generally correspond to the water level of the Nottawasaga River.

The revealed soils change from brown to grey at a depth of 5.5 m below the prevailing ground surface. This indicates that the upper zone of the stratigraphy has oxidized.

The groundwater regime will be subject to seasonal fluctuation and will be impacted by the water level of the Nottawasaga River.

The groundwater yield from the water-bearing sands will be appreciable and persistent. Due to the proximity of the river, excavation into the water-bearing sands will require sheeting and must be stabilized by vigorous pumping from well-points.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath a veneer of topsoil in one location, or a layer of sand fill, the site is generally underlain by a stratum of loose to dense, generally compact fine sand extending to at least the maximum investigated depth in all boreholes. A layer of dense silty fine sand was found embedded within the fine sand stratum.

All boreholes caved at depths ranging from 3.7 to 4.3 m (El. 176.8 to 177.3 m) below the prevailing ground surface which represents the groundwater regime of the site at the time of investigation. The groundwater levels generally correspond to the water level of the Nottawasaga River. The groundwater regime will be subject to seasonal fluctuation and will be impacted by the water level of the Nottawasaga River.

The groundwater regime was detected at depths ranging from 3.5± to 4.0± m below the prevailing ground surface (or at El. 177.0 to 177.5 m), and will fluctuate with the seasons and with the water level of the Nottawasaga Bay. Excavation into the water-bearing sands should be carried out within a sheeting enclosure, and the groundwater must be lowered by vigorous pumping from well-points.

The geotechnical findings which warrant special consideration are presented below:

1. The topsoil is highly compressible and must be stripped as it is unsuitable for engineering applications. Due to its high humus content, it will generate volatile gases under anaerobic conditions. For the environmental as well as the geotechnical well-being of the development, the topsoil should not be buried within the building envelope, or deeper than a depth of 1.2 m below the exterior finished grade.

2. The existing sand fill was loosely placed and it contains topsoil inclusions and other deleterious material. Due to its unknown history and the presence of topsoil, the earth fill is unsuitable to support structures in its present state. It can, however, be upgraded to structural status by being sorted free of serious topsoil inclusions and other deleterious materials, and properly recompacted prior to its use.
3. Due to the presence of sand fill, the subgrade must be carefully inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, or a building inspector who has geotechnical background, to assess its suitability for bearing the designed foundations of the proposed structures.
4. It is understood that the finished floor of the underground parking will be founded at El. 179.7 m and the detected groundwater levels at the time of investigation were at El. 177.0 to 177.5 m. Considering that the founding soil consists of sand, perimeter foundation subdrains are not required. However, the founding level should lie at least 1.0 m above the Seasonal High Groundwater table (SHGW). This can be further assessed at the time of construction.
5. A Class ‘B’ bedding, consisting of compacted 20-mm Crusher-Run Limestone, is recommended for the construction of the underground services. In water-bearing sands, where extensive dewatering is required, a Class ‘A’ bedding consisting of concrete may be required, and the pipe joints should be leak-proof or wrapped with a waterproof membrane.

The recommendations appropriate for the project described in Section 2.0 are presented herein. One must be aware that the subsurface conditions may vary between boreholes. Should this become apparent during construction, a geotechnical engineer must be consulted to determine whether the following recommendations require revision.

6.1 **Foundations**

Based on the borehole findings, the footings must be placed below the topsoil and sand fill onto the sound natural soils. A Maximum Allowable Soil Pressure (SLS) of 150 kPa and a Factored Ultimate Soil Bearing Pressure (ULS) of 250 kPa are recommended for the design of the normal spread and strip footings founded on the sound natural soil. The recommended founding depths for normal foundations are presented in Table 3.

Table 3 - Founding Levels

	Maximum Allowable Soil Pressure (SLS)/ Factored Ultimate Soil Bearing Pressure (ULS) and Corresponding Founding Level		**Observed Groundwater/ Cave-in Levels**
	150 kPa (SLS) 250 kPa (ULS)		
BH No.	**Depth (m)**	**El. (m)**	**El. (m)**
1	1.6 or +	179.4 or -	177.3
2	2.5 or +	178.6 or -	176.8
3	2.5 or +	179.0 or -	177.2
4	1.8 or +	179.8 or -	177.3
5	2.5 or +	178.5 or -	177.3
6	1.6 or +	179.5 or -	177.1

In order to avoid an impact on the footing construction, due to the groundwater conditions, the founding level should lie at least 1.0 m above the SHGW.

Where the footing subgrade consists of wet sands, the inspected subgrade must be protected immediately by a concrete mud-slab. This will prevent construction disturbance and costly rectification.

The design of the foundations should meet the requirements specified in the Ontario Building Code 2006, and the structure should be designed to resist an earthquake force using Site Classification ‘D’ (stiff soil).

The recommended soil pressure (SLS) incorporates a safety factor of 3. The total and differential settlements of the footings are estimated to be 25 mm and 15 mm, respectively.

The footings exposed to weathering, and in unheated areas, should have at least 1.2 m of earth cover for protection against frost.

Due to the presence of earth fill, the subgrade must be carefully inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, or a building inspector who has a geotechnical background, to assess its suitability for bearing the designed foundations.

6.2 Engineered Fill

Where extended footing is required or where earth fill is required to raise the site, it is generally economical to place engineered fill for normal footing, sewer and pavement construction.

The engineering requirements for a certifiable fill for road construction, municipal services, slab-on-grade and footings designed with a 150 kPa Maximum Allowable

Soil Pressure (SLS), depending on the location and founding level, are presented below:

1. All of the topsoil and earth fill must be removed, and the subgrade must be inspected and proof-rolled prior to any fill placement. The badly weathered soils must be subexcavated, sorted free of topsoil inclusions and any deleterious materials, if any, aerated and properly compacted in layers.
2. Inorganic soils must be used, and they must be uniformly compacted in lifts 20 cm thick to 98% or + of their maximum Standard Proctor dry density up to the proposed lot grade and/or road subgrade. The soil moisture must be properly controlled on the wet side of the optimum. If the house foundations are to be built soon after the fill placement, the densification process for the engineered fill must be increased to 100% of the maximum Standard Proctor compaction.
3. If imported fill is to be used, it should be inorganic soils, free of deleterious or any material with environmental issue (contamination). Any potential imported earth fill from off site must be reviewed for geotechnical and environmental quality by the appropriate personnel as authorized by the developer or agency, before it is hauled to the site.
4. If the engineered fill is to be left over the winter months, adequate earth cover, or equivalent must be provided for protection against frost action.
5. The engineered fill must extend over the entire graded area, and the engineered fill envelope must be clearly and accurately defined in the field and precisely documented by qualified surveyors.
6. Foundations partially on engineered fill must be reinforced and designed by a structural engineer to properly distribute the stress induced by the abrupt differential settlement (estimated to be 15± mm) between the natural soils and engineered fill.

7. The engineered fill must not be placed during the period from late November to early April when freezing ambient temperatures occur either persistently or intermittently. This is to ensure that the fill is free of frozen soils, ice and snow.
8. Where the fill is to be placed on a bank steeper than 1 vertical:3 horizontal, the face of the bank must be flattened to 3 + so that it is suitable for safe operation of the compactor and the required compaction can be obtained.
9. Where the ground is wet due to subsurface water seepage, an appropriate subdrain scheme must be implemented prior to the fill placement, particularly if it is to be carried out on sloping ground.
10. The fill operation must be fully supervised and monitored by a technician under the direction of a geotechnical engineer.
11. The footing and underground services subgrade must be inspected by the geotechnical consulting firm that supervised the engineered fill placement. This is to ensure that the foundations are placed within the engineered fill envelope, and the integrity of the fill has not been compromised by interim construction, environmental degradation and/or disturbance by the footing excavation.
12. Any excavation carried out in certified engineered fill must be reported to the geotechnical consultant who supervised the fill placement in order to document the locations of excavation and/or to supervise reinstatement of the excavated areas to engineered fill status. If construction on the engineered fill does not commence within a period of 2 years from the date of certification, the condition of the engineered fill must be assessed for recertification.
13. Despite stringent control in the placement of the engineered fill, variations in soil type and density may occur in the engineered fill. Therefore, the foundations must be properly reinforced and designed by structural engineer for the project. The total and differential settlements of 25 mm and 15 mm,

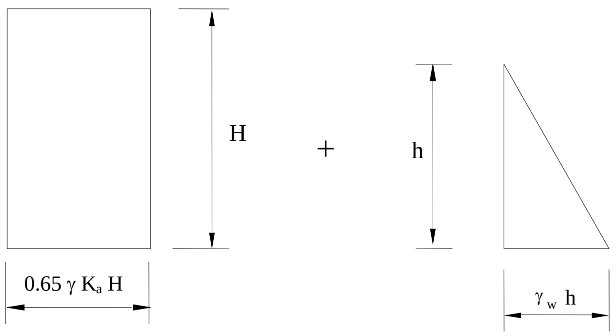
respectively, should be considered in the design of the foundations founded on engineered fill. In sewer construction, the engineered fill is considered to have the same structural proficiency as a natural inorganic soil.

6.3 Underground Services

The subgrade for the underground services should consist of properly compacted inorganic earth fill or sound natural soils. A Class ‘B’ bedding is recommended for the design of the underground services. The bedding material should consist of compacted 20-mm Crusher-Run Limestone, or equivalent. In water-bearing sands where extensive dewatering is required, a Class ‘A’ bedding consisting of concrete may be required.

Where the sewers will be constructed using the open-cut method, the construction must be carried out in accordance with Ontario Regulation 213/91. In areas where a vertical cut is necessary, the use of a trench box is considered to be appropriate. In the design of the trench box and/or shoring structure, the recommended lateral earth pressure distribution for the revealed soils is given in Diagram 1.

Diagram 1 - Lateral Earth Pressure in Sand



H - Height of Pit in metres
γ - 20.5 kN/m^3 above groundwater
γ - 10.8 kN/m^3 below groundwater
h - Height of groundwater level
γ_w - 9.8 kN/m^3
K_a - Lateral Earth Pressure Coefficient
(Table 5, Section 6.7)

In order to prevent pipe floatation when the underground services trench is deluged with water, a soil cover with a thickness equal to the diameter of the pipe should be in place at all times after completion of the pipe installation.

Underground services joints should be leak-proof, or wrapped with a waterproof membrane, to prevent subgrade migration through leakage at joints resulting from inadvertent faulty installation. Openings to subdrains and catch basins should be shielded with a fabric filter to prevent silting.

For estimation purposes for the anode weight requirements in ductile pipes, the electrical resistivity which has been determined for the disclosed soils can be used. This, however, should be confirmed by testing the soils along the water main alignment at the time of sewer construction.

6.4 **Backfilling in Trenches and Excavated Areas**

The on site inorganic soils are generally suitable for trench backfill. The backfill in trenches should be compacted to at least 95% of its maximum Standard Proctor dry density. In the zone within 1.0 m below the road subgrade, the material should be compacted with the water content 2% to 3% drier than the optimum, and the compaction should be increased to at least 98% of the respective maximum Standard Proctor dry density. This is to provide the required stiffness for pavement construction. In the lower zone, the compaction should be carried out on the wet side of the optimum; this allows a wider latitude of lift thickness.

In normal sewer construction practice, the problem areas of road settlement largely occur adjacent to manholes, catch basins and services crossings. In areas which are inaccessible to a heavy compactor, sand backfill should be used. Unless compaction of the backfill is carefully performed, the interface of the native soils and the sand backfill will have to be flooded for a period of several days.

The narrow trenches for services crossings should be cut at 1 vertical:
2 or + horizontal so that the backfill can be effectively compacted. Otherwise, soil arching will prevent the achievement of proper compaction. The lift of each backfill layer should either be limited to a thickness of 20 cm, or the thickness should be determined by test strips.

One must be aware of the possible consequences during trench backfilling and exercise caution as described below:

- When construction is carried out in freezing winter weather, allowance should be made for these following conditions. Despite stringent backfill monitoring, frozen soil layers may inadvertently be mixed with the structural trench backfill. Should the in situ soil have a water content on the dry side of the optimum, it would be impossible to wet the soil due to the freezing condition, rendering difficulties in obtaining uniform and proper compaction. Furthermore, the freezing condition will prevent flooding of the backfill when it is required, such as when the trench box is removed. The above will invariably cause backfill settlement that may become evident within 1 to several years, depending on the depth of the trench which has been backfilled.
- In areas where the underground services construction is carried out during winter months, prolonged exposure of the trench walls will result in frost heave within the soil mantle of the walls. This may result in some settlement as the frost recedes, and repair costs will be incurred prior to final surfacing of the new pavement.

- To backfill a deep trench, one must be aware that future settlement is to be expected, unless the side of the cut is flattened to at least 1 vertical: 1.5+ horizontal, and the lifts of the fill and its moisture content are stringently controlled; i.e., lifts should be no more than 20 cm (or less if the backfilling conditions dictate) and uniformly compacted to achieve at least 95% of the maximum Standard Proctor dry density, with the moisture content on the wet side of the optimum.
- It is often difficult to achieve uniform compaction of the backfill in the lower vertical section of a trench which is an open cut or is stabilized by a trench box, particularly in the sector close to the trench walls or the sides of the box. These sectors must be backfilled with sand. In a trench stabilized by a trench box, the void left after the removal of the box will be filled by the backfill. It is necessary to backfill this sector with sand, and the compacted backfill must be flooded for 1 day, prior to the placement of the backfill above this sector, i.e., in the upper sloped trench section. This measure is necessary in order to prevent consolidation of inadvertent voids and loose backfill which will compromise the compaction of the backfill in the upper section. In areas where groundwater movement is expected in the sand fill mantle, anti-seepage collars should be provided.

6.5 Underground Parking and Slab-On-Grade

The perimeter walls of the underground parking should be designed to sustain a lateral earth pressure calculated using the soil parameters given in Section 6.7, and any applicable surcharge loads adjacent to the proposed building must also be considered in the design of the basement walls.

The floor slab should be constructed on a granular base 20 cm thick, consisting of 20-mm Crusher-Run Limestone, or equivalent, compacted to its maximum Standard Proctor dry density. A Modulus of Subgrade Reaction of 25 MPa/m can be used in the design of the floor slab.

The external grading must be such that water is directed away from the building to prevent ponding adjacent to the underground parking walls.

The subgrade at the building entrances and in unheated areas should be properly insulated, or the subgrade material should be replaced with 1.2 m of non-frost-susceptible granular material and should be provided with subdrains. This will minimize frost action in these areas where vertical ground movement cannot be tolerated. The floor at the entrances and in close proximity to air shafts should be insulated, and the insulation should extend 5.0 m internally. This measure is to prevent frost action induced by cold drafts.

The in situ soil has low frost susceptibility. However, in areas where sand with a high silt content (over 15%) occurs at a shallow depth, one must realize that the ground will heave during cold weather and is susceptible to rainwash erosion.

6.6 **Pavement Design**

Where the pavement is to be built on structural slabs, such as the underground garage rooftop, a sufficient granular base and adequate drainage must be provided to prevent frost damage to the pavement. A waterproof membrane must be placed above the structural slab exposed to weathering to prevent water leakage, as well as to protect the reinforcing steel bars against brine corrosion.

The recommended pavement structure to be placed on the underground garage rooftop is presented in Table 4.

Table 4 - Pavement Design (Underground Garage Rooftop)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	60	HL-4
Granular Base	250	Granular ‘A’
Granular Sub-base	100	Free-draining Sand Fill

Based on the borehole findings, the anticipated subgrade soil will consist of fine sand. The recommended pavement design for the on-grade portion of the parking lot and access road is given in Table 5.

Table 5 - Pavement Design (On-Grade Parking Lot and Access Driveway)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	60	HL-4
Granular Base	150	Granular ‘A’
Granular Sub-base	350	Granular ‘B’

Where the subgrade of the on-grade parking consists of sand with silt content of less than 10%; the 350 mm Granular ‘B’ sub-base can be eliminated and the pavement structure shall consist of 40 mm HL-3 Asphalt Surface, 70 mm HL-4 Asphalt Binder and 200 mm Granular ‘A’ Base.

Prior to placement of the granular bases, the subgrade surface should be proof-rolled, and any soft subgrade should be subexcavated and replaced by properly compacted inorganic earth fill or granular material.

Earth fill used to raise the grade for pavement construction should consist of organic-free sand and be uniformly compacted to 95% or + of its maximum Standard Proctor dry density. The subgrade in the zone within 1.0 m below the underside of the granular sub-base should be compacted to at least 98% of its maximum Standard Proctor dry density, with the moisture content 2% to 3% drier than its optimum.

All the granular bases should be compacted to their maximum Standard Proctor dry density.

Due to the sand subgrade, subdrains are not required for the on-grade parking pavement construction.

6.7 Soil Parameters

The recommended soil parameters for the project design are given in Table 6.

Table 6 - Soil Parameters

Unit Weight and Bulk Factor

	Unit Weight (kN/m^3)		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Loose**	**Compacted**
Sand Fill	20.0	12.0	1.15	0.95
Fine Sand and Silty Fine Sand	20.5	12.4	1.20	0.98

Table 6 - Soil parameters (Cont'd)

Lateral Earth Pressure Coefficient	**Active** **K_a**	**At Rest** **K_o**	**Passive** **K_p**
Sand Fill	0.40	0.50	2.50
Fine Sand and Silty Fine Sand	0.35	0.40	3.00

Maximum Allowable Soil Pressure (SLS) For Thrust Block Design (kPa)	
Engineered Fill	75
Sound Natural Soils	100

6.8 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. For excavation purposes, the types of soils are classified in Table 7.

Table 7 - Classification of Soils for Excavation

Material	**Type**
Moist Sand Fill and Fine Sand	3
Water-bearing Sand	4

The yield of groundwater from the water-bearing sands will be appreciable and persistent.

Excavation in the water-bearing sands should be carried out within a sheeting enclosure, and the possibility of flowing sides and bottom boiling dictates that the ground be predrained by pumping from a well-point dewatering system.

In order to provide a stable subgrade for the services trenches or foundation construction, the groundwater should be maintained at least 0.5 m below the bottom of excavation.

Prospective contractors must be asked to assess the in situ subsurface conditions for soil cuts by digging test pits to at least 0.5 m below the sewer subgrade. These test pits should be allowed to remain open for a period of at least 4 hours to assess the trenching conditions.

6.9 **Bank Slope Stability Analysis**

A slope stability assessment has been carried out at the sloping ground along the northern limit of the site where it abuts the Nottawasaga River. The purpose of the assessment was to determine the erosion hazard limit with respect to the proposed development.

Four cross-sections (A-A, B-B, C-C and D-D) were selected for the analyses. Their locations are shown on Drawing No. 1.

The surface profile at the cross-sections was obtained from the contours as shown on the site survey plan, and the subsurface profile was interpreted from the findings of Boreholes 2 and 4. The cross-section details are shown on Drawing Nos. 3 to 10, inclusive.

As indicated, visual inspection revealed that the slope is well vegetated with weeds, shrubs and scattered trees. The bank is approximately 3.5± m in height with a gradient of 1 vertical:5± horizontal with no signs of seepage erosion or deep-seated failure observed on the slope face. An existing concrete retaining wall extends at the toe of the slope along the river shoreline for erosion control.

The analysis was carried out on the existing slope profile using the force-moment-equilibrium criteria of the Bishop Method and the soil strength parameters given in Table 8.

Table 8 - Soil Strength Parameters

Soil Type	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Internal Friction Angle (◦)
Sand Fill	20.0	0	25
Fine Sand	20.5	0	32

The results of the analyses are presented on Drawing Nos. 3 to 6, inclusive, and the resulting factors of safety (FOS) against deep-seated failure of the stable slope at the locations of the cross-sections are presented in Table 9.

Table 9 - FOS for Existing Slope Profile

Cross-Section	Factor of Safety
A-A	1.67
B-B	2.32
C-C	2.98
D-D	2.77

The results of the analyses indicate that the stable slope satisfies the Ontario Ministry of Natural Resources (OMNR) National Hazards Technical Guidelines and the

Nottawasaga Valley Conservation Authority Guidelines with FOS meeting the required FOS of 1.5.

It should also be noted that slope analyses at the location of the cross-sections have been carried out to model the flood condition and rapid drawdown where the water level will be at the flood level elevation (El. 178.0 m). The results of the analyses are presented on Drawing Nos. 7 to 10, inclusive, and the resulting FOS against deep-seated failure are presented in Table 10.

Table 10 - FOS for Existing Slope Profile (W.L. at Flood Level Elevation)

Cross-Section	**Factor of Safety**
A-A	1.54
B-B	2.00
C-C	2.68
D-D	2.39

The results of the analyses indicate that the stable slope, assuming flood condition and rapid water drawdown, satisfies the OMNR National Hazards Technical Guidelines and the Nottawasaga Valley Conservation Authority Guidelines with FOS meeting the required FOS of 1.2.

Furthermore, since the toe of the slope is protected against active erosion by the existing concrete retaining wall, erosion setback will not be required.

Based on the above, the natural existing top of bank can be considered as the Long-Term Stable Top of Slope (LTSTOS), and the construction of the proposed residential development on the tableland and behind the LTSTOS will not have an adverse impact on the stability of the slope and vice versa. The LTSTOS line is presented on Drawing No. 1. It should be noted that the stability and the structural integrity of the existing concrete retaining wall is outside the scope of this study. It should be further assessed to verify its condition and its ability to protect against active toe erosion.

It should be pointed out that the development setback from the established LTSTOS and any other allowances are subject to the requirements of local conservation authority.

One must be aware that in order to prevent environmentally significant slope creep and localized surface slides, and to enhance the stability of existing slope, the following geotechnical constraints should be stipulated:

1. The prevailing vegetative cover must be maintained, since its extraction would deprive the slope of the rooting system that is reinforcement against soil erosion by weathering. If for any reason the vegetation cover is stripped, it must be reinstated to its original, or better than its original, protective condition.
2. The leafy topsoil cover on the slope face should not be disturbed, since this provides an insulation and screen against frost wedging and rainwash erosion.
3. Grading of the land adjacent to the slope must be such that concentrated runoff is not allowed to drain onto the slope face. Landscaping features which may cause runoff to pond at the top of the slope, as well as frequent lawn watering which will saturate the crown of the slope, must not be allowed.

The above recommendations should be reviewed by and are subject to the approval of the Nottawasaga Valley Conservation Authority.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of Dreamwood Developments Inc., and for review by its designated agents, financial institutions, and government agencies. Use of the report is subject to the conditions and limitations of the contractual agreement. The material in the report reflects the judgment of Basim Al Ali, P.Eng., and Bernard Lee, P.Eng., in light of the information available to it at the time of preparation. Any use which a Third Party makes of this report, and/or any reliance on decisions to be made based on it are the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

SOIL ENGINEERS LTD.

Basim Al-Ali, P.Eng.

Bernard Lee, P.Eng.
BAA/BL:dd

LICENSED PROFESSIONAL ENGINEER
A. 18, 2017
B. B. Y. LEE
100104568
PROVINCE OF ONTARIO

LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms commonly employed on the borehole logs and figures, and in the text of the report, are as follows:

SAMPLE TYPES

AS	Auger sample
CS	Chunk sample
DO	Drive open (split spoon)
DS	Denison type sample
FS	Foil sample
RC	Rock core (with size and percentage recovery)
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

PENETRATION RESISTANCE

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows for each foot of penetration of a 2-inch diameter, 90° point cone driven by a 140-pound hammer falling 30 inches.

Plotted as '—●—'

Standard Penetration Resistance or 'N' Value:

The number of blows of a 140-pound hammer falling 30 inches required to advance a 2-inch O.D. drive open sampler one foot into undisturbed soil.

Plotted as '○'

WH	Sampler advanced by static weight
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
NP	No penetration

SOIL DESCRIPTION

Cohesionless Soils:

'N' (blows/ft)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

Cohesive Soils:

Undrained Shear Strength (ksf)	'N' (blows/ft)	Consistency
less than 0.25	0 to 2	very soft
0.25 to 0.50	2 to 4	soft
0.50 to 1.0	4 to 8	firm
1.0 to 2.0	8 to 16	stiff
2.0 to 4.0	16 to 32	very stiff
over 4.0	over 32	hard

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0	Field vane test in borehole; the number denotes the sensitivity to remoulding
△	Laboratory vane test
□	Compression test in laboratory

For a saturated cohesive soil, the undrained shear strength is taken as one half of the undrained compressive strength

METRIC CONVERSION FACTORS

1 ft = 0.3048 metres
1lb = 0.454 kg
1 inch = 25.4 mm
1ksf = 47.88 kPa

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JOB NO.: 1707-C037

LOG OF BOREHOLE NO.: 1

FIGURE NO.: 1

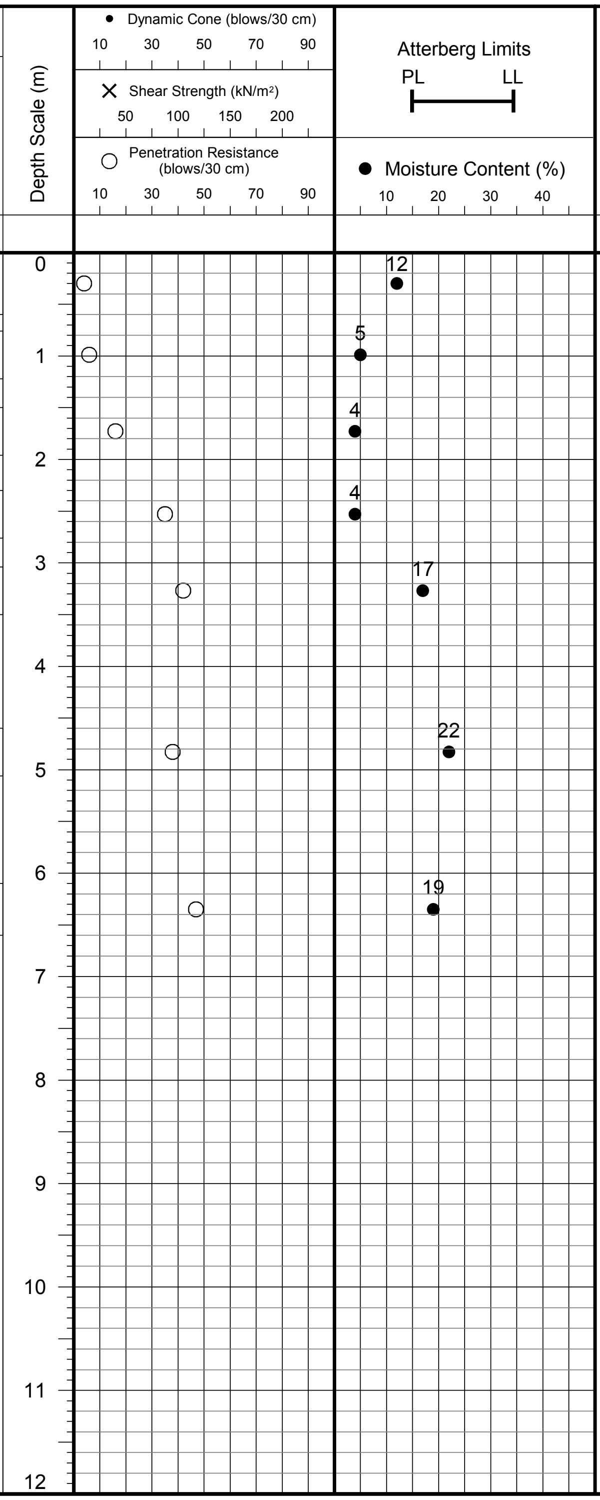
PROJECT DESCRIPTION: Proposed 4-Storey Apartment Building with 1-Level Underground Parking

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 60-69 River Road East - Town of Wasaga Beach

DRILLING DATE: July 15, 2013

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
181.0	Ground Surface			
0.0 0.2	**23 cm TOPSOIL**	1	DO	4
	Loose to dense	2	DO	6
	weathered	3	DO	16
	FINE SAND a tr. of silt	4	DO	35
		5	DO	42
	silty fine sand layer	6	DO	38
	brown grey	7	DO	47
174.4 6.6	**END OF BOREHOLE**			



WATER LEVEL

Cave-in @ El. 177.3 m on completion

Soil Engineers Ltd.

JOB NO.: 1707-C037

LOG OF BOREHOLE NO.: 2

FIGURE NO.: 2

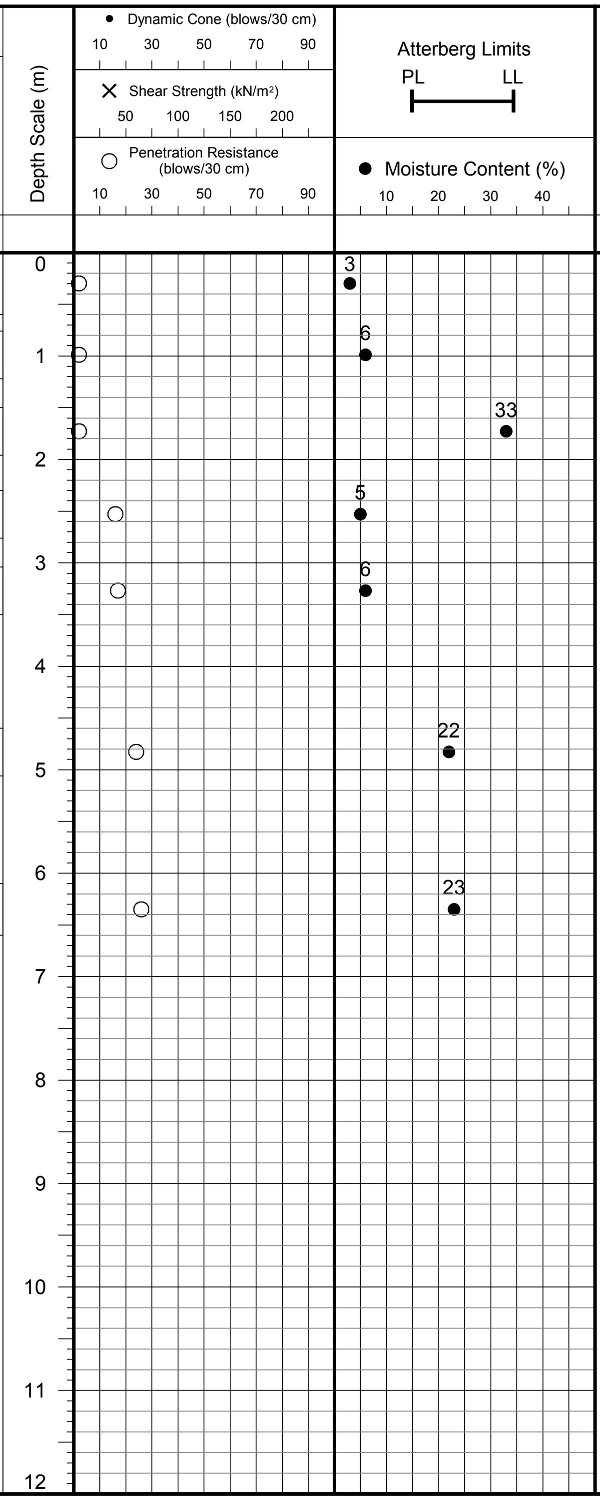
PROJECT DESCRIPTION: Proposed 4-Storey Apartment Building with 1-Level Underground Parking

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 60-69 River Road East - Town of Wasaga Beach

DRILLING DATE: July 15, 2013

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
181.1	Ground Surface			
0.0	Brown **EARTH FILL** sand with occ. topsoil inclusions and wood debris	1	DO	2
		2	DO	2
		3	DO	2
178.8 2.3	Compact **FINE SAND** a tr. to some silt	4	DO	16
		5	DO	17
		6	DO	24
	— brown grey			
		7	DO	26
174.5 6.6	**END OF BOREHOLE**			



WATER LEVEL

Cave-in @ El. 176.8 m on completion

Soil Engineers Ltd.

JOB NO.: 1707-C037

LOG OF BOREHOLE NO.: 3

FIGURE NO.: 3

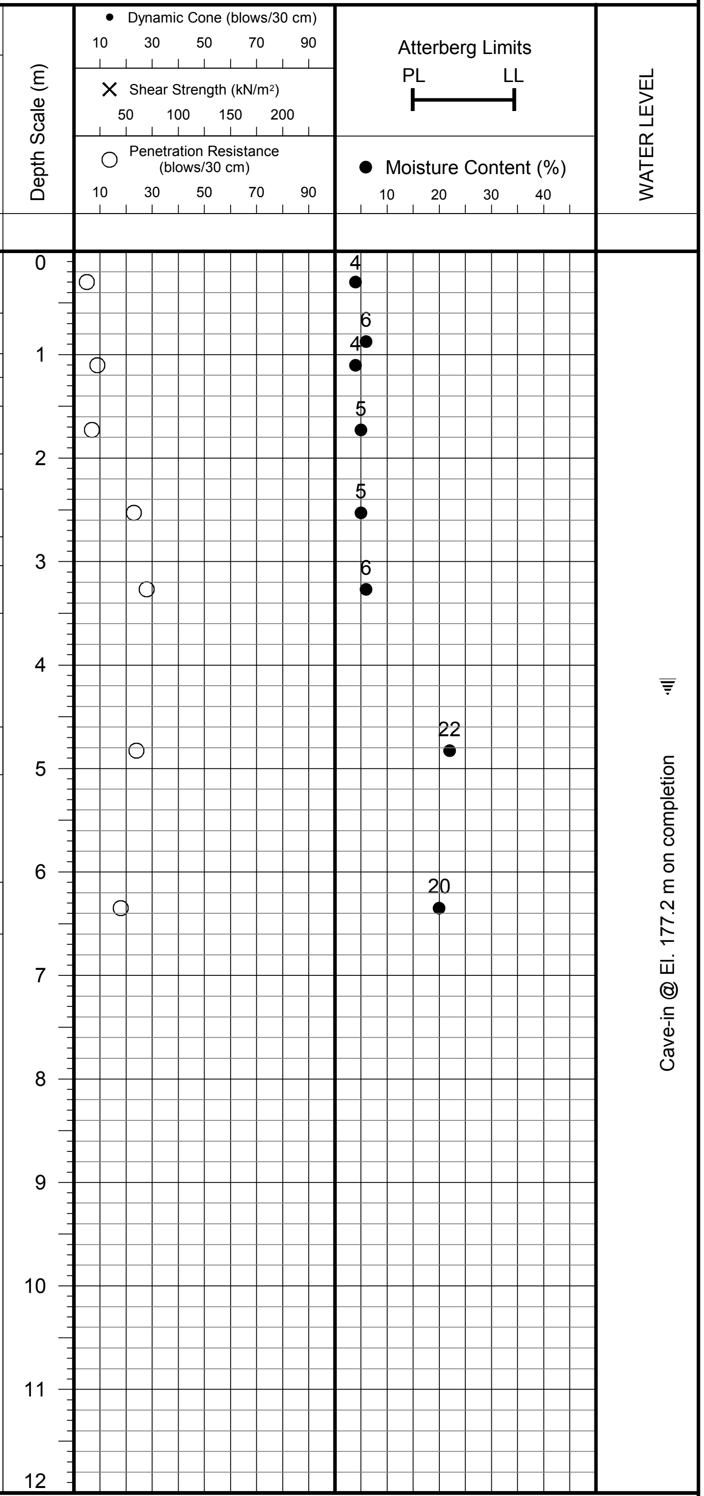
PROJECT DESCRIPTION: Proposed 4-Storey Apartment Building with 1-Level Underground Parking

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 60-69 River Road East - Town of Wasaga Beach

DRILLING DATE: July 15, 2013

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES		
		Number	Type	N-Value
181.5	Ground Surface			
0.0	Brown **EARTH FILL** a tr. sand with occ. topsoil inclusions concrete and asphalt debris	1	DO	5
180.5		2A	DO	
1.0	Loose to compact	2B	DO	9
		3	DO	7
	weathered			
	FINE SAND traces of silt and shells	4	DO	23
		5	DO	28
		6	DO	24
	brown grey			
174.9		7	DO	18
6.6	**END OF BOREHOLE**			



Soil Engineers Ltd.

JOB NO.: 1707-C037

LOG OF BOREHOLE NO.: 4

FIGURE NO.: 4

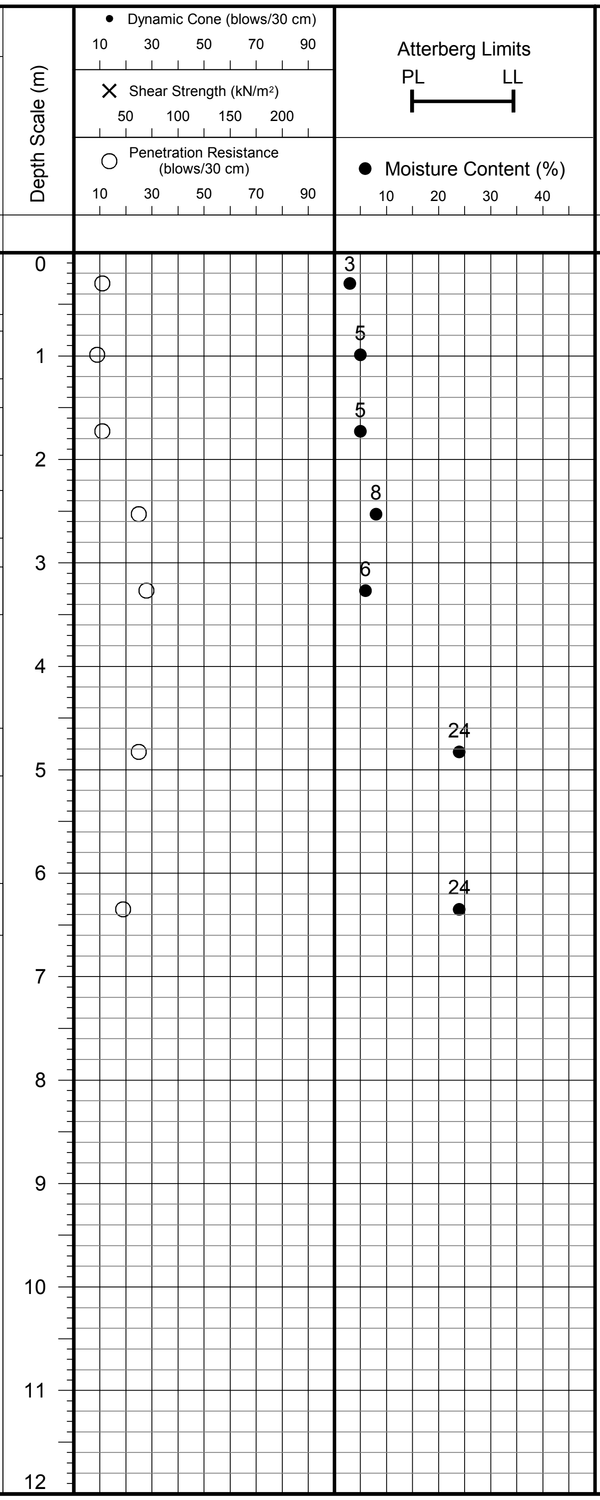
PROJECT DESCRIPTION: Proposed 4-Storey Apartment Building with 1-Level Underground Parking

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 60-69 River Road East - Town of Wasaga Beach

DRILLING DATE: July 15, 2013

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
181.6	Ground Surface			
0.0	Brown **EARTH FILL** sand with occ. topsoil inclusions traces of gravel and cobbles	1	DO	11
180.8 0.8	Loose to compact	2	DO	9
	weathered			
		3	DO	11
	FINE SAND a tr. of silt	4	DO	25
		5	DO	28
		6	DO	25
	brown grey			
175.0		7	DO	19
6.6	**END OF BOREHOLE**			



WATER LEVEL

Cave-in @ El. 177.3 m on completion

Soil Engineers Ltd.

JOB NO.: 1707-C037

LOG OF BOREHOLE NO.: 5

FIGURE NO.: 5

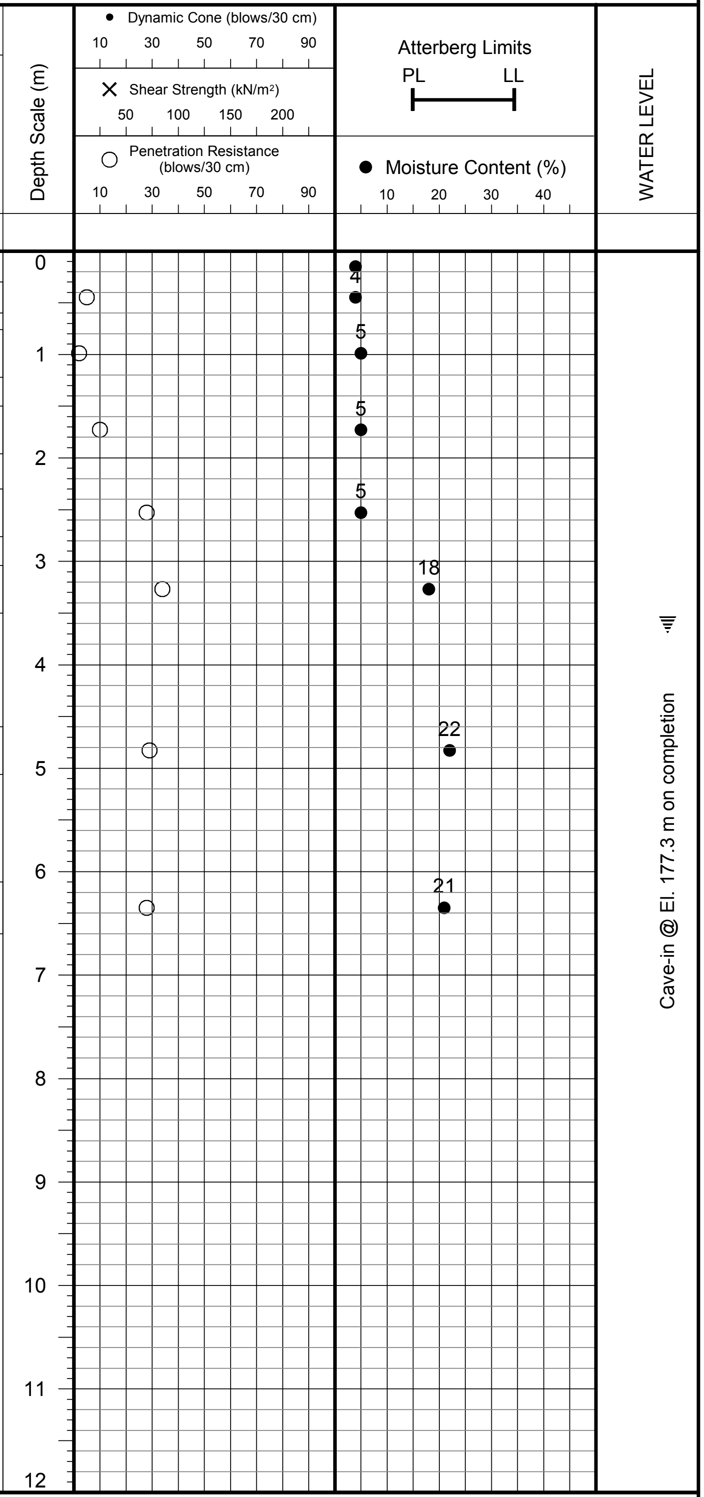
PROJECT DESCRIPTION: Proposed 4-Storey Apartment Building with 1-Level Underground Parking

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 60-69 River Road East - Town of Wasaga Beach

DRILLING DATE: July 15, 2013

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
181.0	Ground Surface			
0.0	Brown	1A	DO	
		1B	DO	5
	EARTH FILL sand with occ. topsoil inclusions	2	DO	2
179.5 1.5	Loose to dense weathered	3	DO	10
		4	DO	28
	FINE SAND traces of silt and shells	5	DO	34
		6	DO	29
	brown grey	7	DO	28
174.4 6.6	**END OF BOREHOLE**			



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JOB NO.: 1707-C037

LOG OF BOREHOLE NO.: 6

FIGURE NO.: 6

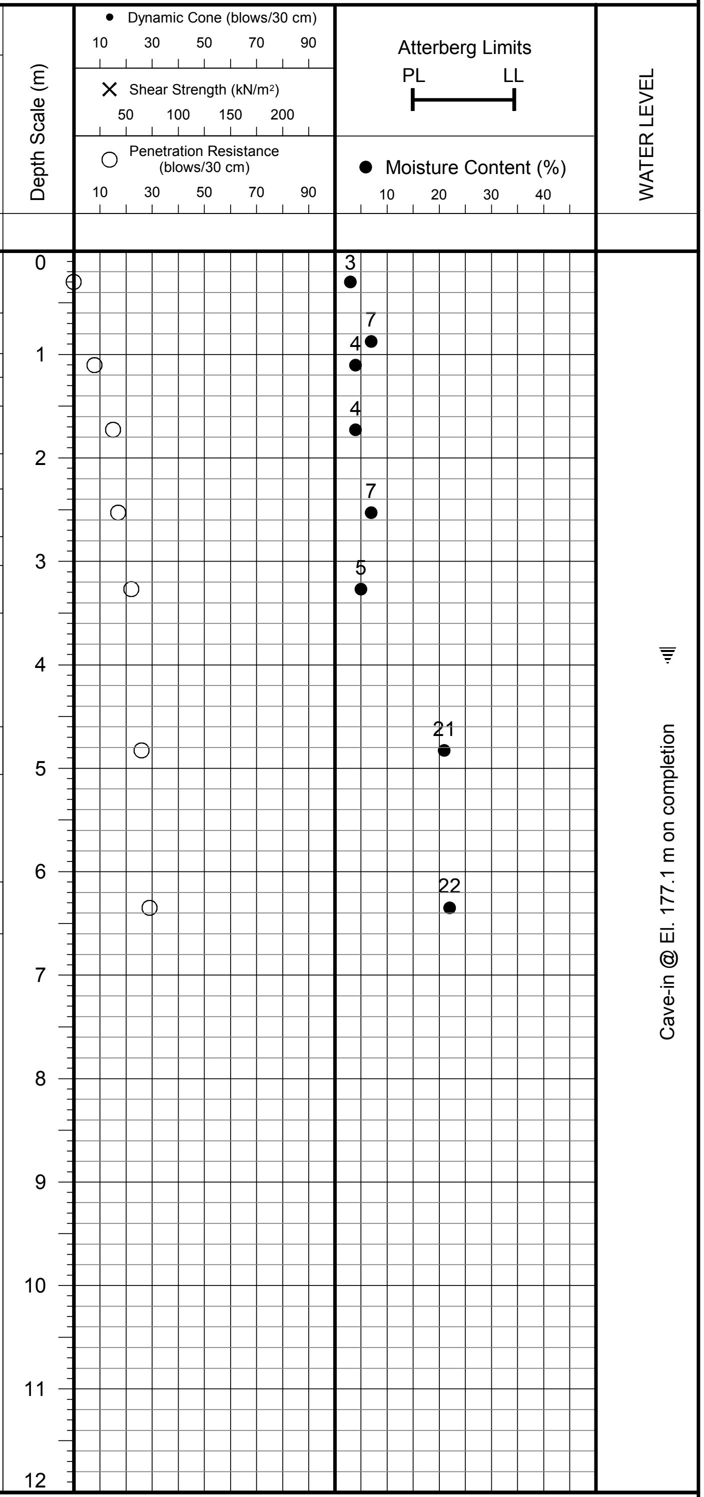
PROJECT DESCRIPTION: Proposed 4-Storey Apartment Building with 1-Level Underground Parking

METHOD OF BORING: Flight-Auger

PROJECT LOCATION: 60-69 River Road East - Town of Wasaga Beach

DRILLING DATE: July 15, 2013

El. (m) Depth (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
181.1	Ground Surface			
0.0	Brown **EARTH FILL** sand with occ.topsoil inclusions	1	DO	0
		2A	DO	
180.1 1.0	Loose to compact weathered	2B	DO	8
		3	DO	15
	FINE SAND traces of silt and shells	4	DO	17
		5	DO	22
		6	DO	26
	brown grey			
		7	DO	29
174.5 6.6	**END OF BOREHOLE**			



Cave-in @ El. 177.1 m on completion

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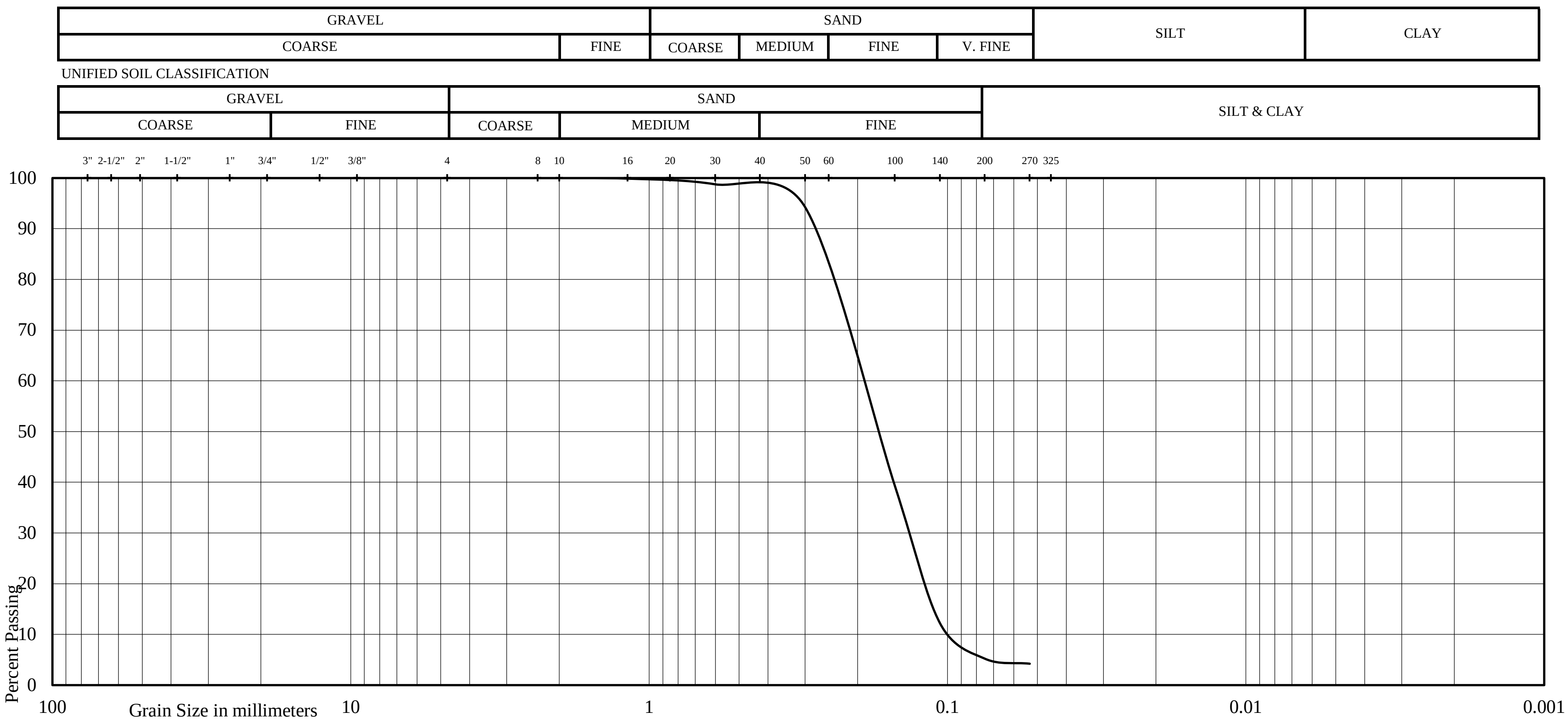
GRAIN SIZE DISTRIBUTION

Reference No: 1306-S162



Project: Proposed 4-Storey Appartment Building with 1-Level Underground Parking

Location: 60-69 River Road East, Town of Wasaga Beach

Liquid Limit (%) = -

Plastic Limit (%) = -

Borehole No: 2

Plasticity Index (%) = -

Sample No: 1

Moisture Content (%) = 3

Depth (m): 0.3

Estimated Permeability (cm./sec.) = 10^{-2}

Elevation (m): 180.8

Classification of Sample [& Group Symbol]: FINE SAND FILL, a trace of silt

Figure: 7

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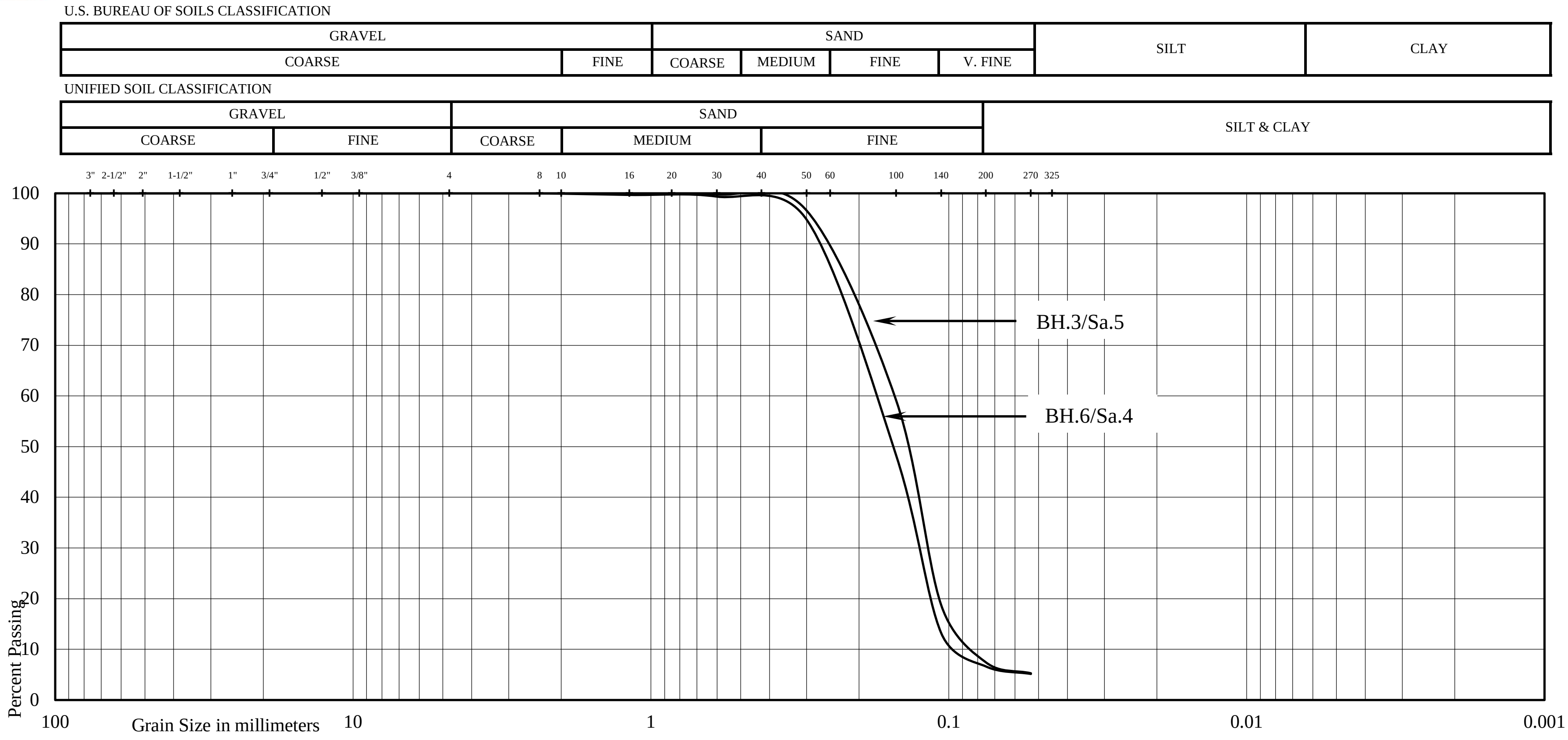
GRAIN SIZE DISTRIBUTION

Reference No: 1306-S162



Project: Proposed 4-Storey Appartment Building with 1-Level Underground Parking

Location: 60-69 River Road East, Town of Wasaga Beach

Borehole No:	3	6
Sample No:	5	4
Depth (m):	3.3	2.6
Elevation (m):	178.2	178.5

BH./Sa.	3/5	6/4
Liquid Limit (%) =	-	-
Plastic Limit (%) =	-	-
Plasticity Index (%) =	-	-
Moisture Content (%) =	6	7
Estimated Permeability (cm./sec.) =	10^{-2}	10^{-2}

Classification of Sample [& Group Symbol]: FINE SAND, a trace of silt

Figure: 8

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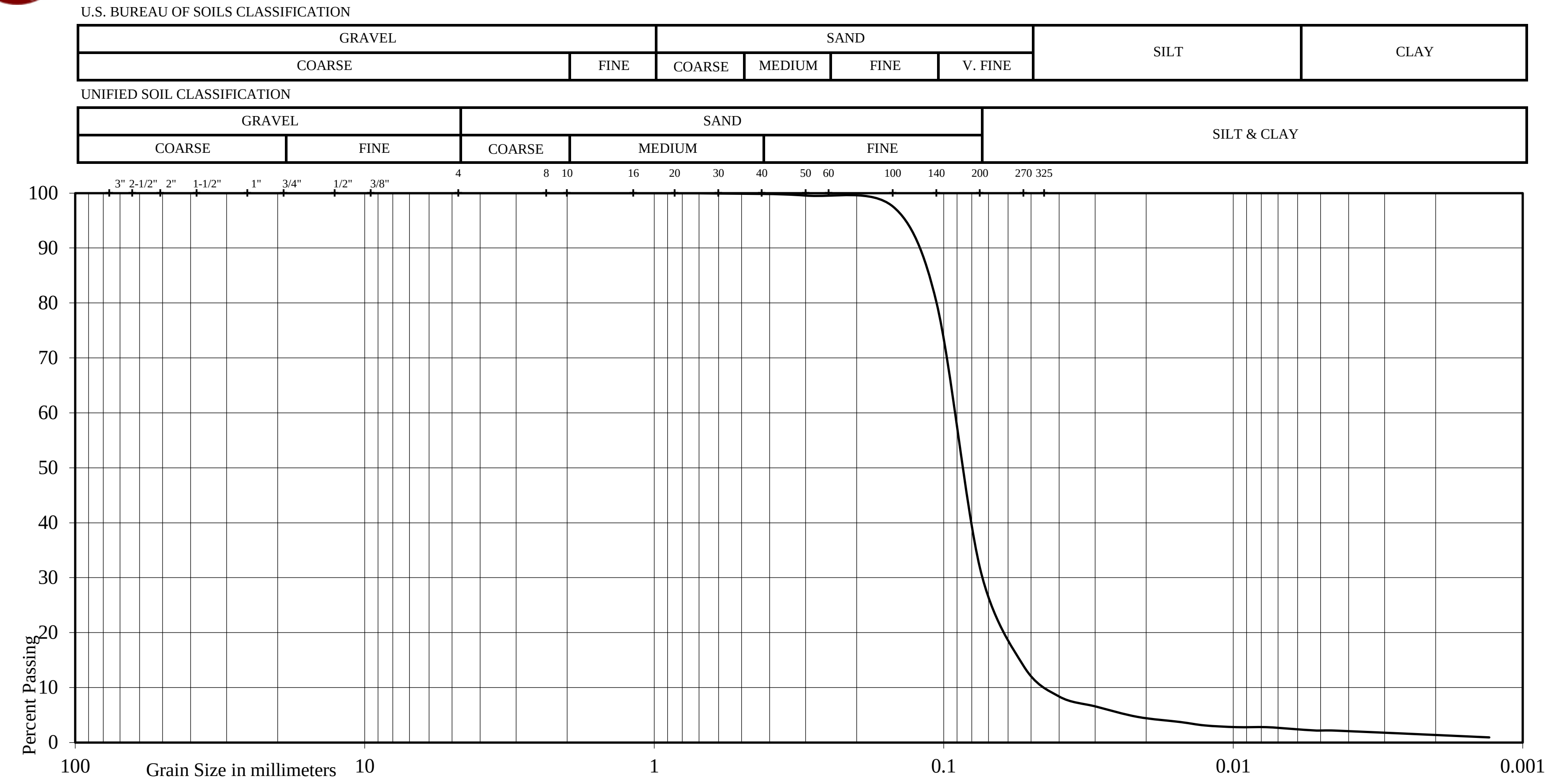
GRAIN SIZE DISTRIBUTION

Reference No: 1306-S162



Project: Proposed 4-Storey Appartment Building with 1-Level Underground Parking

Location: 60-69 River Road East, Town of Wasaga Beach

Borehole No: 1

Sample No: 6

Depth (m): 4.9

Elevation (m): 176.1

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

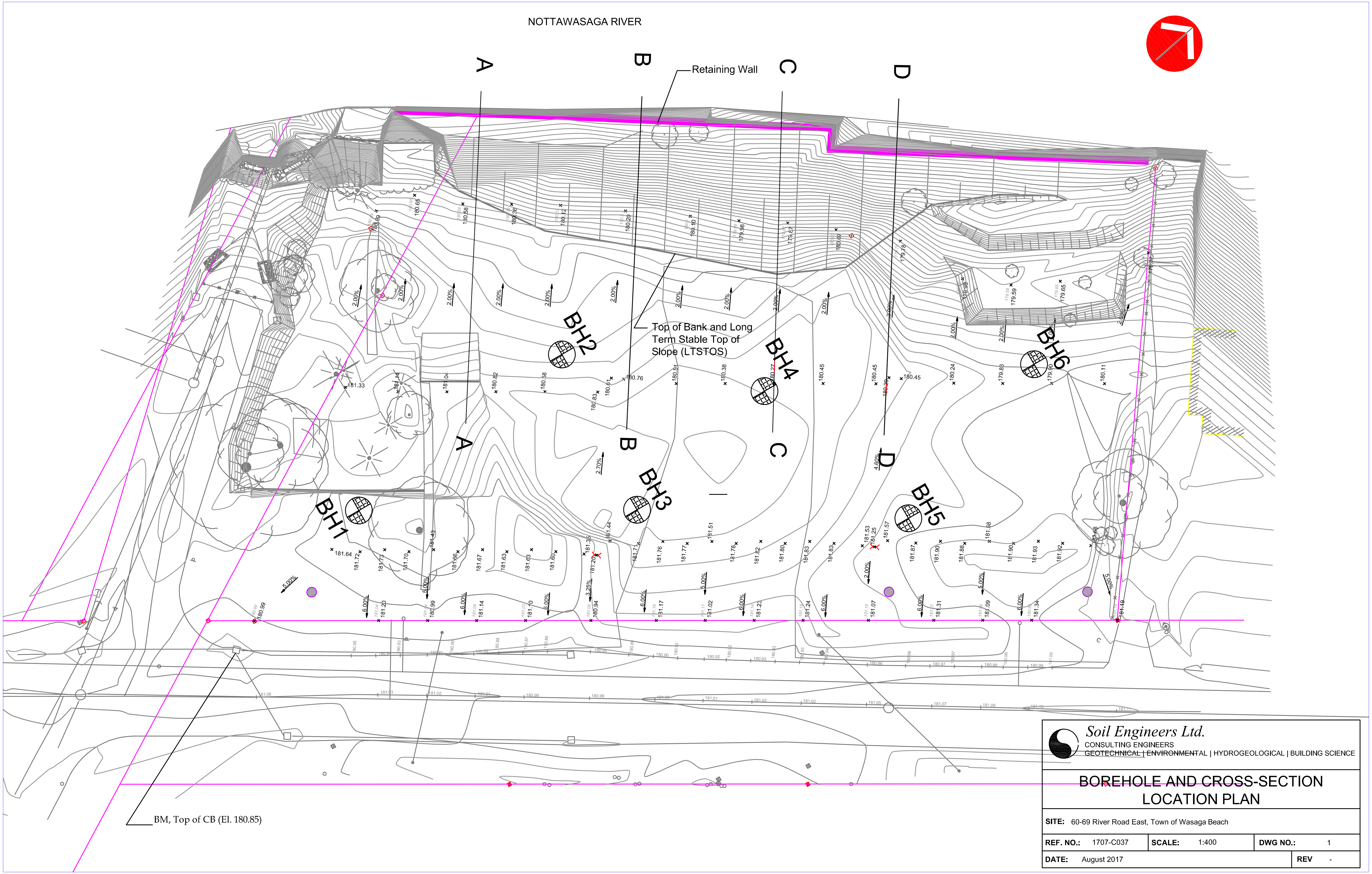
Moisture Content (%) = 22

Estimated Permeability

(cm./sec.) = 10^{-3}

Classification of Sample [& Group Symbol]: SILTY FINE SAND

Figure: 9



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BOREHOLE AND CROSS-SECTION LOCATION PLAN

SITE: 60-69 River Road East, Town of Wasaga Beach

REF. NO.:	1707-C037	**SCALE:**	1:400	**DWG NO.:**	1
DATE:	August 2017			**REV**	-

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SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

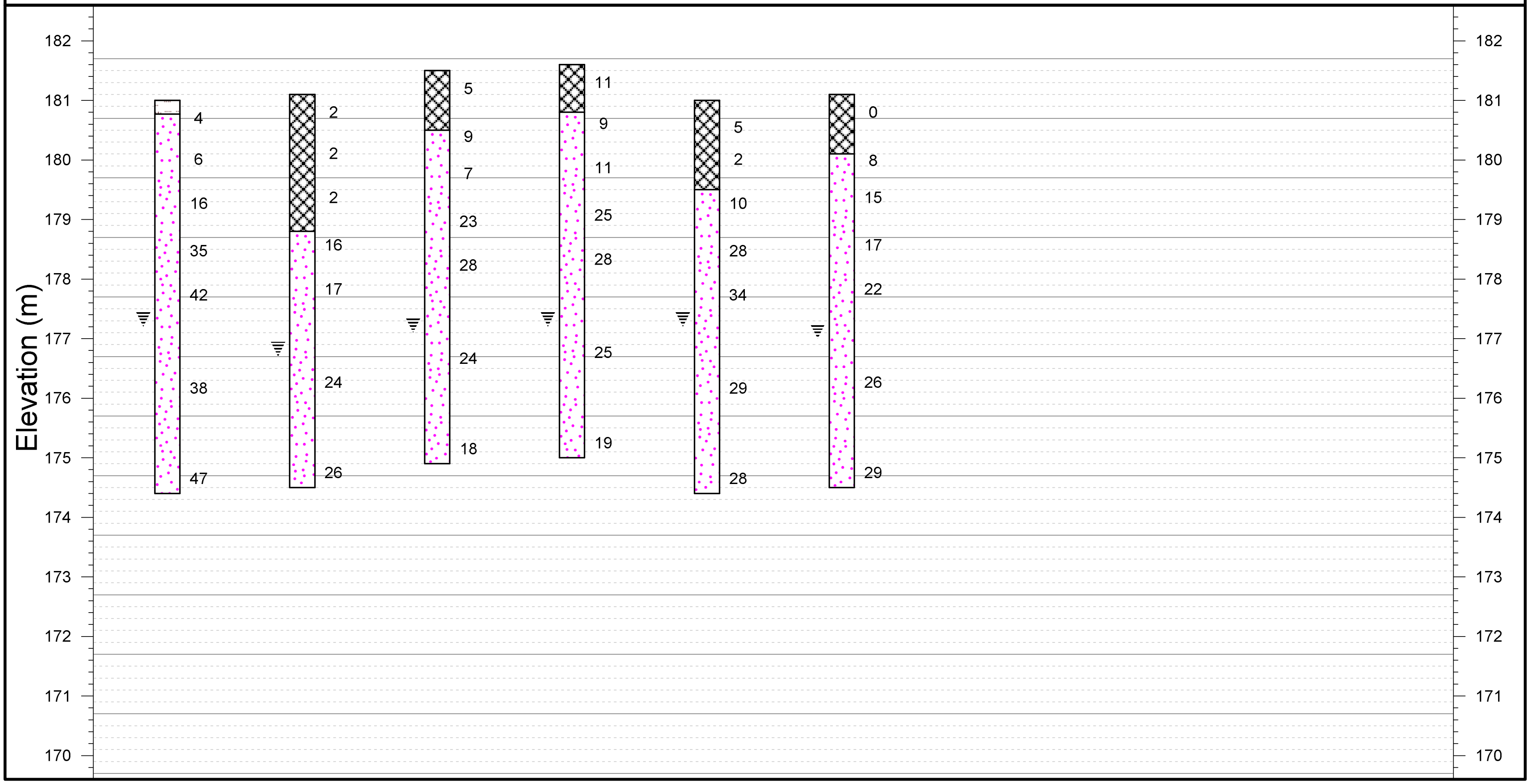
JOB NO.: 1707-C037
REPORT DATE: August 2017
PROJECT DESCRIPTION: Proposed 4-Storey Apartment Building with 1-Level Underground Parking
PROJECT LOCATION: 60-69 River Road East - Town of Wasaga Beach

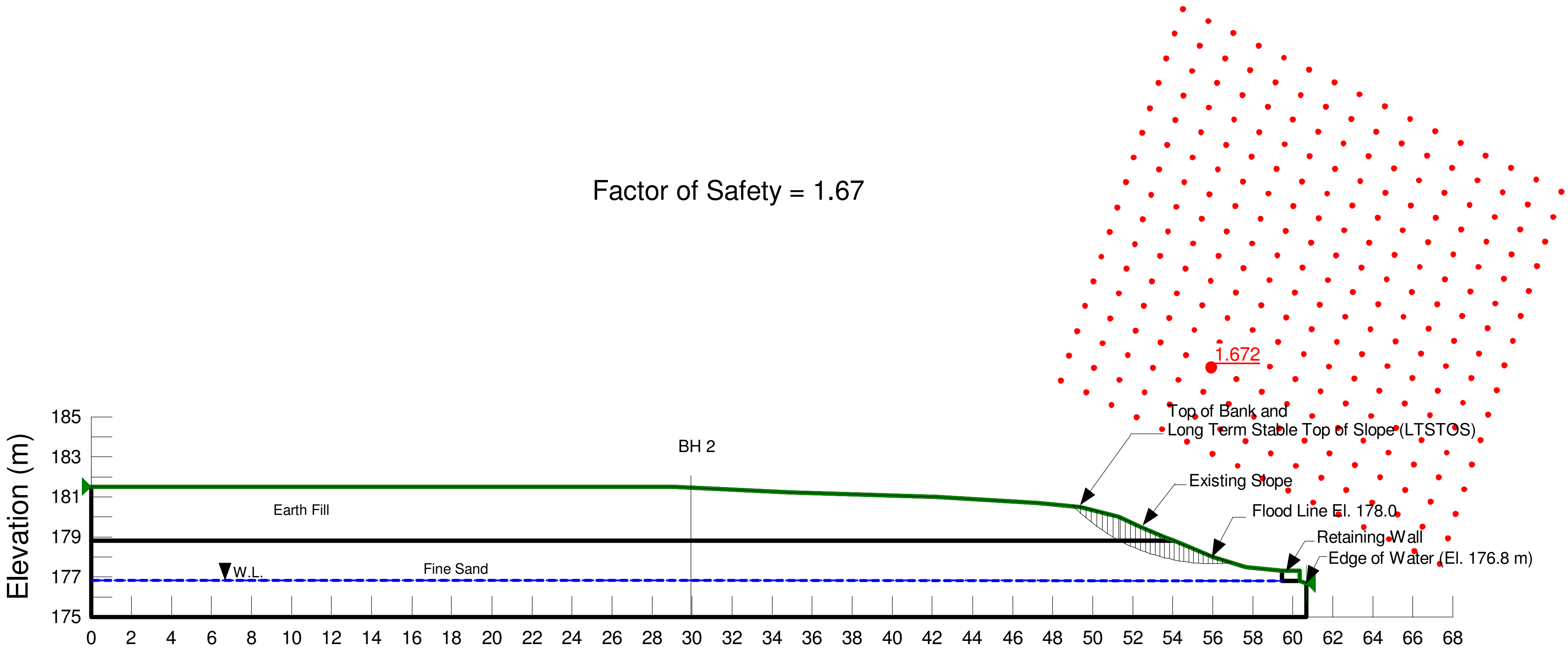
LEGEND

▭ TOPSOIL ▩ FILL ⁚ SAND

CAVE-IN ≡

BH No.:	1	2	3	4	5	6
El. (m):	181	181.1	181.5	181.6	181	181.1





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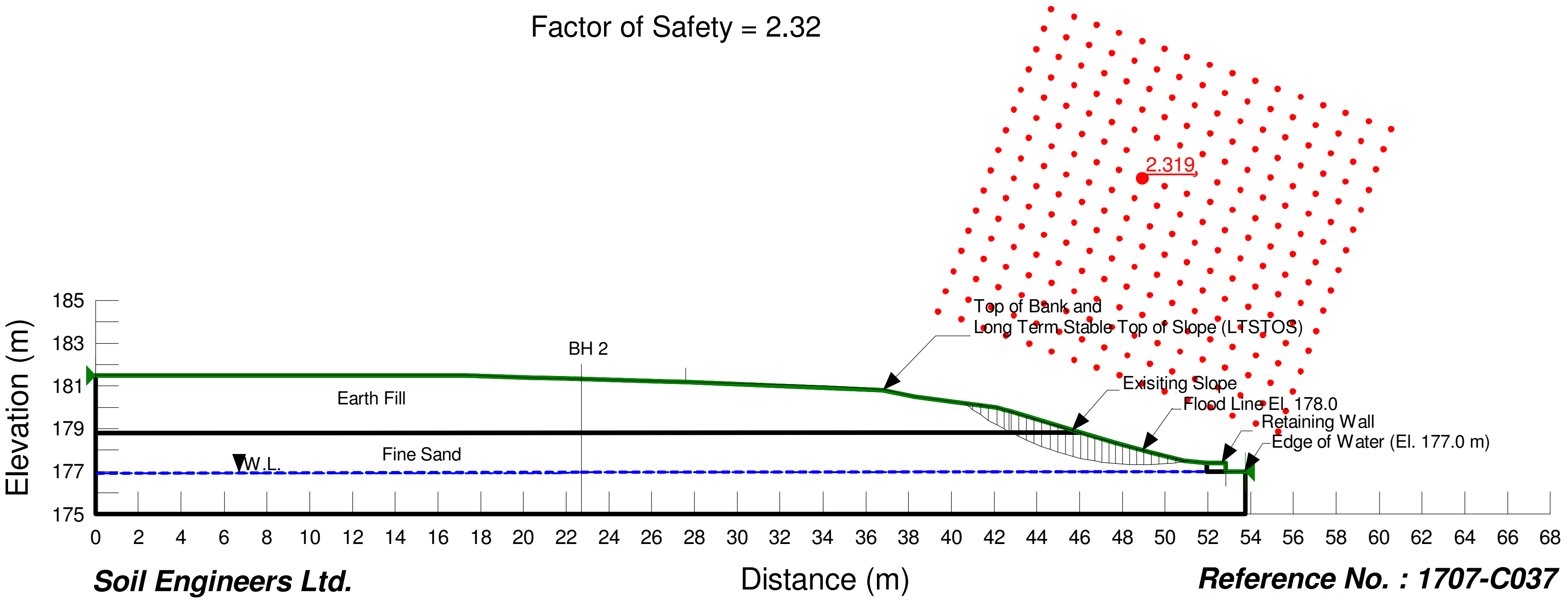
Cross-Section A-A
(Existing Slope)

Reference No. : 1707-C037
Drawing No. : 3
Scale : Horz. 1:300
Vert. 1:300

Factor of Safety = 2.32

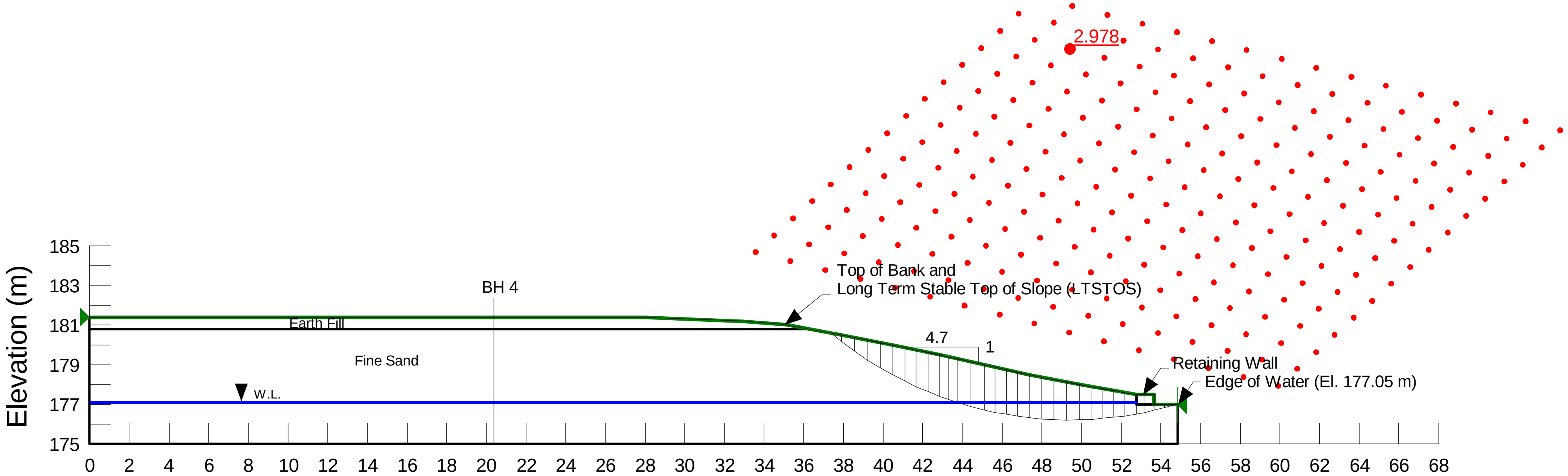


Cross-Section B-B
(Existing Slope)

Soil Engineers Ltd.

Reference No. : 1707-C037
Drawing No. : 4
Scale : Horz. 1:300
Vert. 1:300

Factor of Safety = 2.98



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Distance (m)

Cross-Section C-C

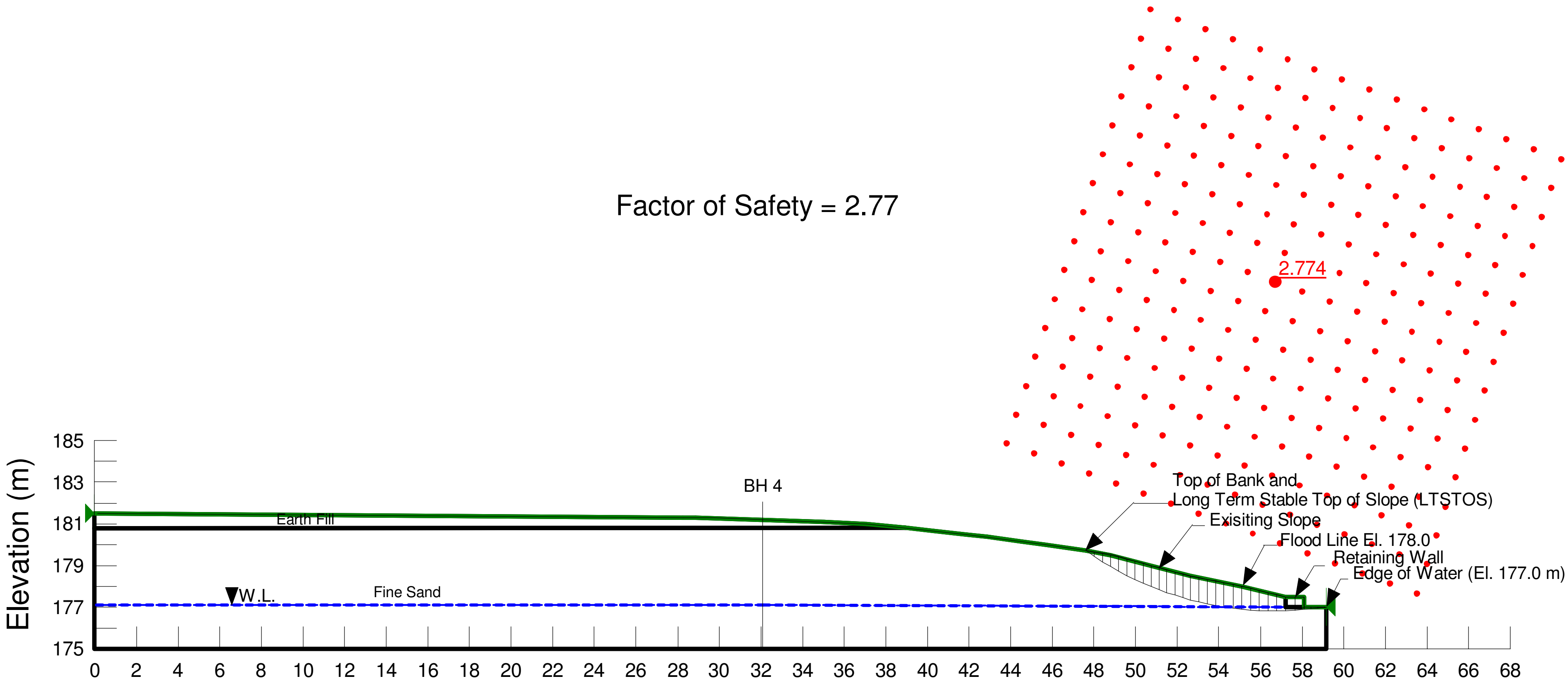
(Existing Slope)

Reference No. : 1707-C037

Drawing No. : 5

Scale : Horz. 1:300

Vert. 1: 300



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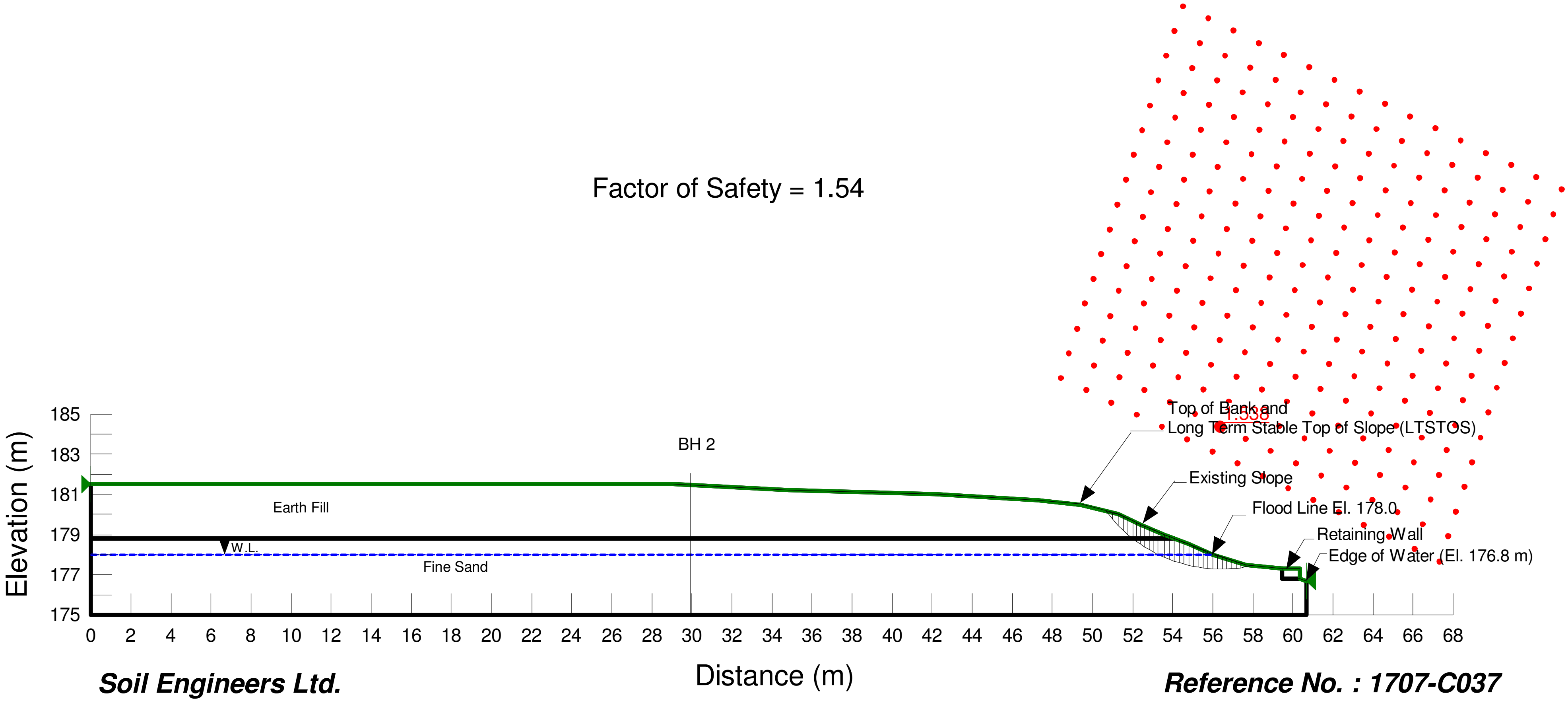
Cross-Section D-D
(Existing Slope)

Reference No. : 1707-C037
Drawing No. : 6
Scale : Horz. 1:300
Vert. 1:300



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Cross-Section A-A

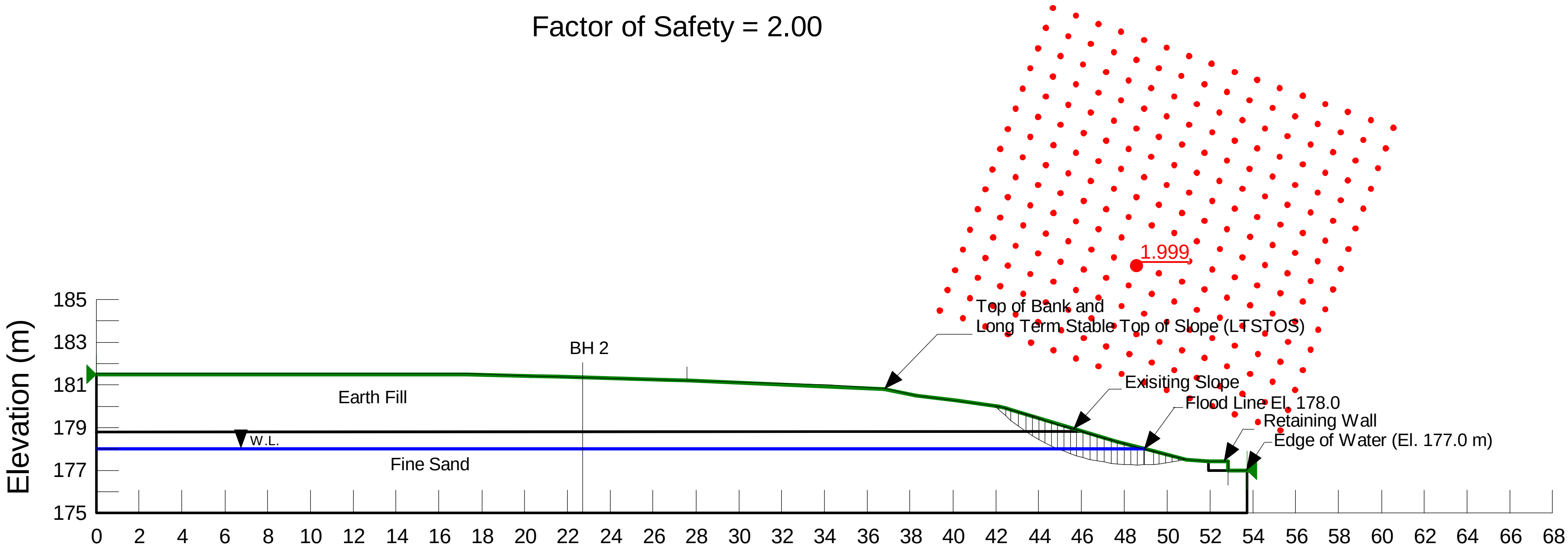
(Existing Slope with W.L at Flood Line Elev. 178.00 m)

Reference No. : 1707-C037

Drawing No. : 7

Scale : Horz. 1:300
Vert. 1:300

Factor of Safety = 2.00



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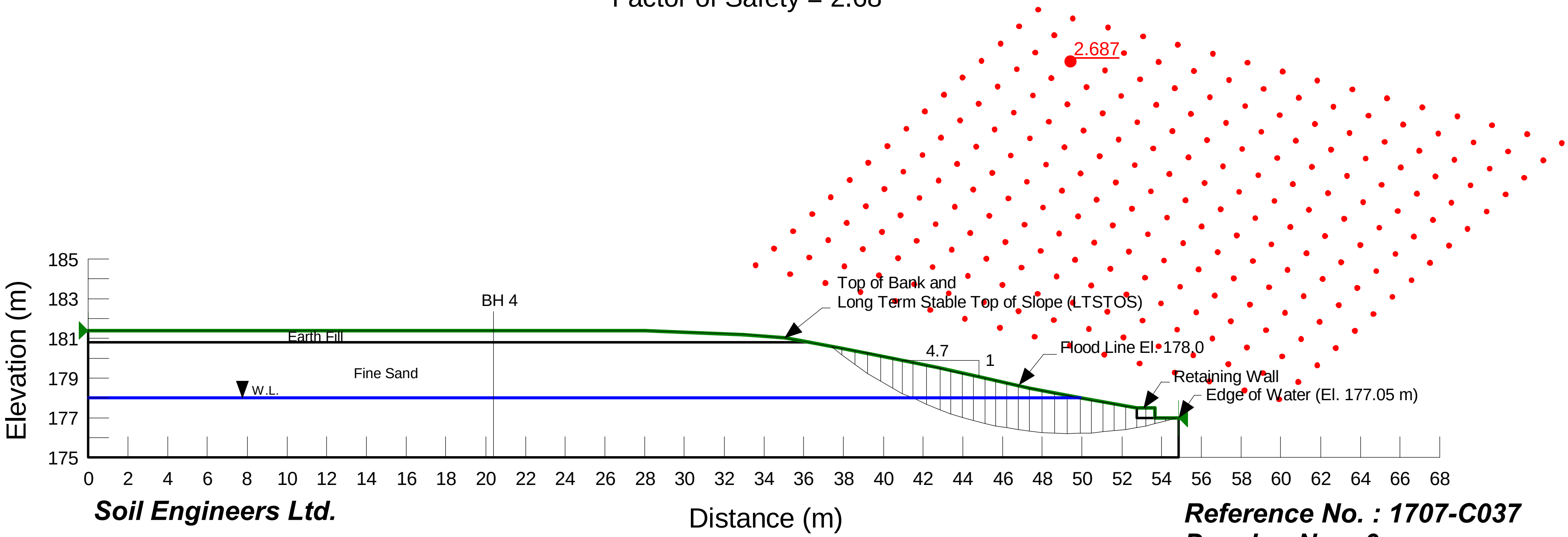
Cross-Section B-B
(Existing Slope with WL at Flood Line Elev. 178.00 m)

Reference No. : 1707-C037
Drawing No. : 8
Scale : Horz. 1:300
Vert. 1:300

Factor of Safety = 2.68



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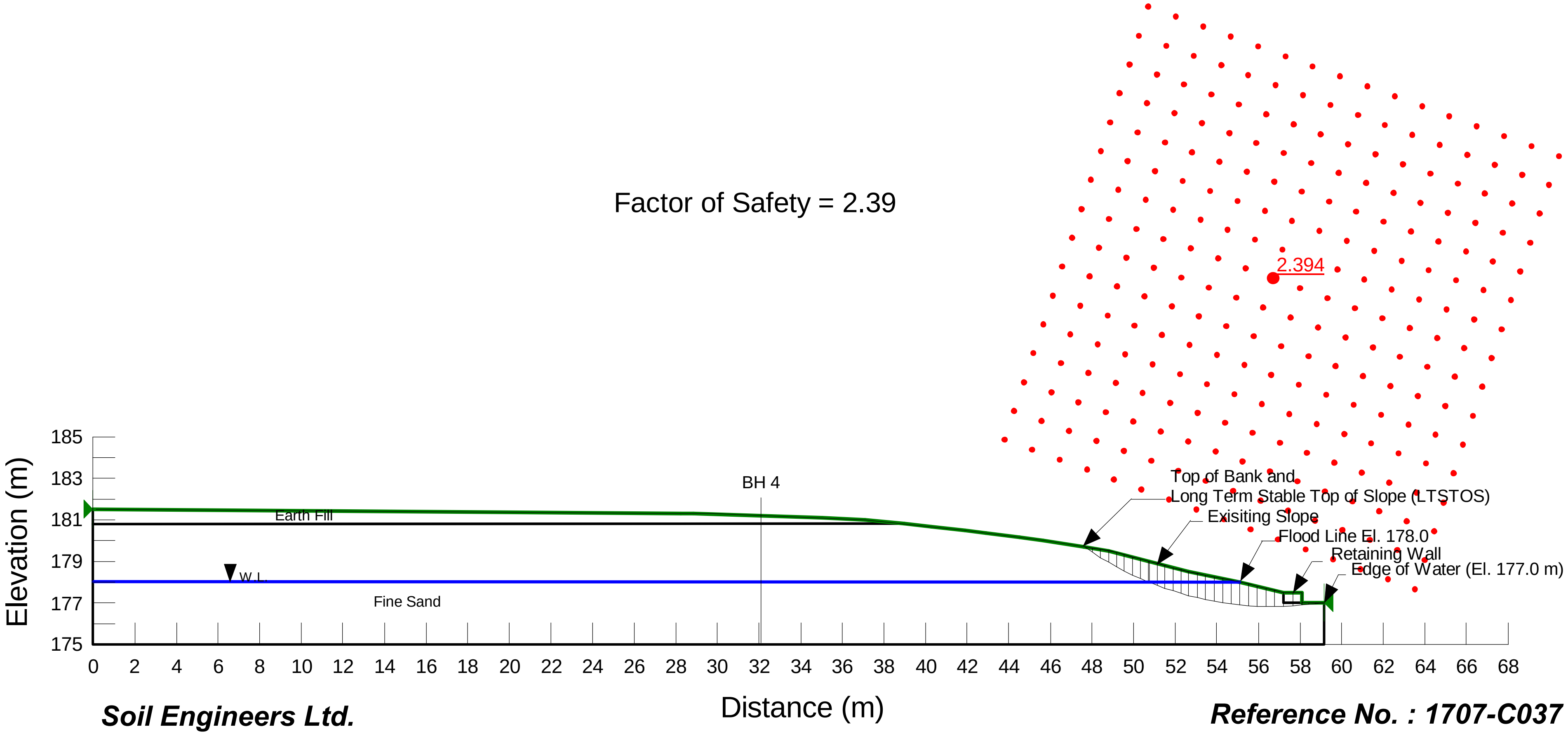
Cross-Section C-C
(Existing Slope with WL at Flood Line Elev. 178.00 m)

Reference No. : 1707-C037
Drawing No. : 9
Scale : Horz. 1:300
Vert. 1: 300



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Cross-Section D-D

(Existing Slope with WL at Flood Line Elev. 178.00 m)

Reference No. : 1707-C037

Drawing No. : 10

Scale : Horz. 1:300

Vert. 1:300