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A REPORT TO
HEN & CHICKENS BREWING CO. LTD.

A SOIL INVESTIGATION FOR PROPOSED INDUSTRIAL DEVELOPMENT

MOUNTAIN ROAD

TOWN OF COLLINGWOOD

Reference No. 1110-S118

DECEMBER 2011

DISTRIBUTION

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1.0 **INTRODUCTION**

In accordance with instructions from Mr. Dan Hurley, P.Eng., Vice President of C.C. Tatham & Associates, and written authorization dated October 21, 2011, from Mr. Garnet Siddall, President, of Hen & Chickens Brewing Co. Ltd., a soil investigation was carried out at a parcel of land located on Mountain Road, between 10th Line and High Street, in the Town of Collingwood, for a proposed Industrial Development.

The purpose of the investigation was to reveal the subsurface conditions and to determine the engineering properties of the disclosed soils for the design and construction of the proposed project.

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

2.0 SITE AND PROJECT DESCRIPTION

The Town of Collingwood is situated in the Simcoe Lowlands bordering the Niagara Escarpment where lacustrine sand, silt and clay deposits, outwash sands and glacial till have bedded onto undulated Black River and Trenton Group of bedrock.

The subject site consists of a vacant, wooded parcel of land, encompassing an approximate area of 4 ha (10 ac). The site is located on the south side of Mountain Road. The neighbouring properties consist of Mountain Road and a vegetated field to the north, industrial properties to the northeast and east, Black Ash Creek to the southeast, and vegetated fields to the south and west. The ground surface is relatively flat with minor undulations.

The proposed project will consist of the construction of an industrial development which will be provided with municipal services, and paved-on-grade parking areas and access roadways meeting urban standards.

3.0 FIELD WORK

The field work, consisting of 9 boreholes to depths ranging from 2.6 to 3.4 m, was performed on November 14, 2011, at the locations shown on the Borehole 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 interpolated from the spot elevations and contours on the conceptual site plan provided by C.C. Tatham & Associates.

4.0 **SUBSURFACE CONDITIONS**

Detailed descriptions of the encountered subsurface conditions are presented on the Borehole Logs, comprising Figures 1 to 9, inclusive. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2, and the engineering properties of the disclosed soils are discussed herein.

The investigation has disclosed that beneath a topsoil veneer, the site is generally underlain by strata of fine sand, silty fine sand, silty clay till, silty sand till and fine to coarse sand.

4.1 **Topsoil** (All Boreholes)

The thickness of the topsoil ranges from 23 to 30 cm. It is dark brown in colour, indicating it contains an appreciable amount of roots and humus. These materials are unstable and compressible under loads; therefore, they are considered to be void of engineering value. Due to its humus content, it will generate an offensive odour and may produce volatile gases under anaerobic conditions. Therefore, the topsoil must not be buried deeper than 1.5 m below the finished grade so it will not have an adverse impact on the environmental well-being of the developed areas.

Since the topsoil is considered void of engineering value, it can only be used for general landscaping and landscape contouring purposes. A fertility analysis should be carried out to determine the suitability of the topsoil for general planting material.

4.2 **Fine Sand** (Boreholes 3, 4, 5 and 8) and **Silty Fine Sand** (Boreholes 1 and 2)

The fine sand and silty fine sand deposits were encountered in the upper zone of the revealed stratigraphy. Sample examinations show that the fine sand and silty fine sand are non-cohesive, and generally in saturated condition. The wet samples became dilatant when shaken by hand.

The natural water content of the soil samples was determined and the results are plotted on the Borehole Logs. The values range from 12% to 26%, with a median of 24%, indicating that the deposits are likely water-bearing.

The obtained ‘N’ values range from 2 to 6, with a median of 4 blows per 30 cm of penetration, indicating that the relative density of the sands is very loose to loose, being generally very loose. The high water content and low ‘N’ values show the deposits have been loosened by the weathering process.

A grain size analysis was performed on 1 representative sample of the silty fine sand, and the result is plotted on Figure 10.

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

- High frost susceptibility with high soil-adfreezing potential.
- High water erodibility.
- Pervious to relatively pervious, with an estimated coefficient of permeability of 10^{-3} to 10^{-5} cm/sec, and runoff coefficients of:

Slope	
0% - 2%	0.04 to 0.11
2% - 6%	0.09 to 0.16
6% +	0.13 to 0.23

- Frictional soils, their shear strength is derived from internal friction and is density dependent. Due to their dilatancy, the shear strength of the wet soils is susceptible to impact disturbance; i.e., the disturbance will induce a build-up of pore pressure within the soil mantle, resulting in soil dilation and a reduction of shear strength.
- In cuts, the wet soil will slough readily, run with seepage and boil with a piezometric head of about 0.4 m.
- Fair materials to support pavement, with an estimated California Bearing Ratio (CBR) value of 8% to 20%.
- Moderately low corrosivity to buried metal, with an estimated electrical resistivity of 5000 ohm·cm

4.3 **Silty Clay Till** (Boreholes 1, 2, 4, 6, 7, 8 and 9)

The silty clay till is encountered through out the property. It consists of a random mixture of soils; the particle sizes range from clay to gravel, with the clay fraction exerting the dominant influence on its soil properties. It is heterogeneous in structure, showing it is a glacial deposit. Occasional wet sand and silt seams and layers were found laminated in the till mantle. In places, the uppermost layer of the till stratum is weathered.

Hard resistance to augering was encountered, indicating the presence of cobbles and/or boulders in the till mantle.

The obtained 'N' values range from 1 blow per 30 cm to refusal, with a median of 24 blow per 30 cm. This indicates that the consistency of the till is very soft to hard, being generally very stiff.

The Atterberg Limits of 1 representative sample and the water content of all the samples were determined. The results are plotted on the Borehole Logs and summarized below:

Liquid Limit	24%
Plastic Limit	15%
Natural Water Content	5% to 29% (median 8%)

The above values show that the clay till is a cohesive material with low plasticity. The natural water content generally lies below its plastic limit, confirming the generally very stiff consistency as disclosed by the ‘N’ values. The high water content and low ‘N’ values are generally restricted the surficial weathered zone of the clay till.

A grain size analysis was performed on 1 representative sample; the result is plotted on Figure 11.

Based on the above findings, the soil engineering properties pertaining to the project are given below:

- Moderate frost susceptibility and low soil-adfreezing potential.
- Low water erodibility.
- Low permeability, with an estimated coefficient of permeability of 10^{-7} cm/sec, and runoff coefficients of:

Slope	
0% - 2%	0.15
2% - 6%	0.20
6% +	0.28

- A cohesive soil, its shear strength is primarily derived from consistency which is inversely related to its moisture content. It contains sand; therefore, its shear strength is augmented by internal friction.
- It will generally be stable in a relatively steep cut; however, prolonged exposure will allow the fissures in the weathered zone and the wet sand seams and layers to become saturated which may lead to localized sloughing.
- A very poor pavement-supportive material, with an estimated CBR value of less than 3%.
- Moderate corrosivity to buried metal, with an estimated electrical resistivity of 4000 ohm·cm.

4.4 **Silty Sand Till** (Boreholes 1 to 5, inclusive, and Borehole 9)

The silty sand till was found interstratified within the soil stratigraphy at various depths and locations. It consists of a random mixture of soils; the particle size range from clay to gravel, with the sand fraction exerting the dominant influence on the soil properties. The sand till is heterogeneous in structure, indicating that it is a glacial deposit. Occasional wet sand and silt seams and layers, cobbles and boulders were detected in the till mantle. In places, the till contains occasional silty clay till layers.

Hard resistance to augering was encountered, showing that occasional cobbles and boulders are embedded in the till mantle.

Sample examinations disclosed that the till, in places, is cemented and it displayed some cohesion when remoulded, indicating that it contains variable amounts of clay. The samples slaked readily when placed in water, and when shaken the wet samples displayed a low to moderate dilatancy.

The natural water content was determined, and the results are plotted on the Borehole Logs. The values range from 6% to 12%, with a median of 8%, showing the sandy silt till is in a moist to very moist condition, being generally moist.

The obtained 'N' values range from 4 blows per 30 cm to refusal, with a median of 17 blows per 30 cm of penetration. This indicates the relative density of the till is very loose to very dense, being generally compact. The very loose and loose conditions of the silty sand till found at a depth of 1.0+ m below the prevailing ground surface in Boreholes 1, 2 and 5, were likely resulted from the suction effect during retracting of the augers that loosened the in situ soil.

Grain size analyses were performed on 4 representative samples; the results are plotted on Figures 12 and 13.

Based on the laboratory and field findings, the engineering properties related to the project are as follows:

- Moderately high frost susceptibility and low soil-adfreezing potential.
- Moderate water erodibility.
- Relatively low permeability, with an estimated coefficient of permeability of 10^{-5} to 10^{-6} cm/sec, and runoff coefficients of:

Slope	
0% - 2%	0.11 to 0.15
2% - 6%	0.16 to 0.20
6% +	0.23 to 0.28

- A frictional soil, its shear strength is density dependent and is augmented by cementation and cohesion.
- The moist soil will be stable in relatively steep cuts; however, with prolonged exposure, localized sheet collapse will likely occur, particularly in

the zone where wet sand and silt layers are prevalent. An open cut-face in the very moist soil will slough slowly.

- A poor pavement-supportive material, with an estimated CBR value of 7%.
- Moderate corrosivity to buried metal, with an estimated electrical resistivity of 4500 ohm·cm.

4.5 **Fine to Coarse Sand** (Boreholes 2, 3 and 4)

The sand deposits occur at 3 locations in the north sector of the property and sand seams and layers are laminated within the soil stratigraphy at various depths and locations. The sand contain a trace of gravel and some silt with occasional silt seams and layers.

The obtained ‘N’ values range from 11 per 30 cm to refusal, with a median of 42 blows per 15 cm of penetration, which indicates that the relative density of the sand is compact to very dense, being generally very dense. The loose sand is restricted to the weathered zone.

The natural water content of the soil samples was determined, and the results are plotted on the Borehole Logs. The values range from 8% to 15%, with a median of 10%, and show that the sand is likely water bearing. It should be noted that due to the pervious nature of the sand, some of the water may have drained during sampling; therefore, the determined values may not represent the actual water content as determined from sample examinations.

A grain size analysis was performed on 1 representative sample. The result is plotted on Figure 13.

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

- Low frost susceptibility and heaving potential.
- High water erodibility.
- Pervious, with an estimated coefficient of permeability of 10^{-3} cm/sec, 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, thus being density dependent.
- In cuts, the wet sand will slough. It will run with seepage and boil with a piezometric head of about 0.4 m.
- A fair pavement-supportive material, with an estimated CBR value of 20%.
- Moderately low corrosivity to buried material, with an estimated electrical resistivity of 5500 ohm·cm.

4.6 Interpretation of Refusal to Augering

Refusal to augering or Standard Penetration Tests was encountered in 7 of the 9 boreholes at depths ranging from 2.6 to 3.4 m below the prevailing ground surface. It is inferred that boulders and/or, most likely, bedrock occur at these levels.

4.7 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 soil for Standard Proctor compaction are presented in Table 1.

Table 1 - Estimated Water Content for Compaction

		Water Content (%) for Standard Proctor Compaction	
Soil Type	**Determined Natural Water Content (%)**	**100% (optimum)**	**Range for 95% or +**
Fine Sand and Silty Fine Sand	12 to 26 (median 24)	12	8 to 16
Silty Clay Till	5 to 29 (median 8)	14	10 to 19
Silty Sand Till	6 to 12 (median 8)	9 and 10	6 to 15
Fine to Coarse Sand	8 to 15 (median 10)	10	5 to 14

The above values show that the tills are generally suitable for a 95% or + Standard Proctor compaction. The sands and some portions of the tills are wet and will require mixing with drier soils and/or aeration by spreading the soils thinly on the ground in dry, warm weather prior to structural compaction. The wet sands can also be properly stockpiled to drain them of excess water.

The tills should be compacted using a heavy-weight, kneading-type roller. The sands can be compacted by a smooth roller with or without vibration, depending on the water content of the soils being compacted. The lifts for compaction should be limited to 20 cm, or to a suitable thickness as assessed by test strips performed by the equipment which will be used at the time of construction.

When compacting the hard and very dense tills on the dry side of the optimum, the compactive energy will frequently bridge over the chunks in the soil and be transmitted laterally into the soil mantle. Therefore, the lifts must be limited to 20 cm or less (before compaction). It is difficult to monitor the lifts of backfill placed in deep trenches; therefore, it is preferable that the compaction of backfill at depths over 1.0 m below the road subgrade be carried out on the wet side of the optimum. This would allow a wider latitude of lift thickness. Wetting of the sound tills will be necessary to achieve this requirement.

If the compaction of the soils is carried out with the water content within the range for 95% Standard Proctor dry density but on the wet side of the optimum, the surface of the compacted soil mantle will roll under the dynamic compactive load. This is unsuitable for road construction since each component of the pavement structure is to be placed under dynamic conditions which will induce the rolling action of the subgrade surface and cause structural failure of the new pavement. The foundations or bedding of the sewer and slab-on-grade will be placed on a subgrade which will not be subjected to impact loads. Therefore, the structurally compacted soil mantle with the water content on the wet side or dry side of the optimum will provide an adequate subgrade for the construction.

The presence of boulders will prevent transmission of the compactive energy into the underlying material to be compacted. If an appreciable amount of boulders over 15 cm in size is mixed with the material, it must either be sorted or must not be used for construction of engineered fill and/or structural backfill.

5.0 GROUNDWATER CONDITIONS

Groundwater seepage encountered during augering was recorded on the field logs. The groundwater and/or cave-in levels were measured upon completion of the boreholes, and the data are plotted on the Borehole Logs and listed in Table 2.

Table 2 - Groundwater Levels

	Borehole	Soil Colour Changes Brown to Grey	Seepage Encountered During Augering		Measured Groundwater/ Cave-in* Level On Completion	
BH No.	**Depth (m)**	**Depth (m)**	**Depth (m)**	**Amount**	**Depth (m)**	**El. (m)**
1	3.4	1.5	1.5	Some	2.1	180.2
2	3.4	1.5	2.3	Appreciable	1.8/2.7*	180.5/179.6*
3	2.6	-	1.5	Appreciable	1.5*	180.7*
4	3.4	1.5	1.5	Appreciable	2.1/2.4*	180.0/179.7*
5	2.6	2.3	1.5	Some	0.5/0.6*	181.7/181.6*
6	3.2	2.3	-	-	Dry	-
7	3.2	1.5	3.0	Some	2.1	180.4
8	3.4	2.3	1.5	Some	0.6	181.7
9	2.9	2.3	2.5	Some	2.6	179.7

* Cave-in level (In wet sands, the level generally represents the groundwater regime at the borehole location).

Groundwater was detected in 8 of the 9 boreholes, and the measured groundwater/ cave-in levels range from 0.5 to 2.7± m below the prevailing ground surface (El. 181.7 m to El. 179.7 m). Borehole 6 remained dry upon completion of the field work. The groundwater encountered at shallow depths is likely derived from infiltrated precipitation trapped in the fissures of the weathered soils and in the sand and silt seams and layers, rendering perched groundwater at shallow depths. It should be also noted that the shallow groundwater levels, in places, are likely attributable to the surface water ponding at the borehole locations at the time of investigation.

The colour of the revealed soils changes from brown to grey at depths of 1.5 m and 2.3 m below the ground surface. This shows that the soils in the upper zone of the stratigraphy have oxidized and the groundwater regime lies in the grey saturated soils. The groundwater level will be subject to seasonal fluctuation.

The yield of groundwater is expected to be small and limited from the silty clay till. From the silty sand till the yield is expected to be moderate; from the water-bearing sands, the yield will likely be appreciable and may be persistent, depending on the extent and continuity of the sand deposits. Excavations in the silty clay till and silty sand till can be stabilized by normal pumping from sumps. Excavations in the water-bearing sands will need to be stabilized by vigorous pumping from closely-spaced sumps and/or the use of a well-point dewatering system; alternatively, sheeting of the excavation will be required to control the groundwater. The appropriate dewatering method should be further assessed by test pits and/or test pumping prior to the project construction.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation has disclosed that beneath a topsoil veneer, the site is generally underlain by strata of very loose to loose, generally very loose fine sand and silty fine sand; very soft to hard, generally very stiff silty clay till; very loose to very dense, generally compact silty sand till; and compact to very dense, generally very dense fine to coarse sand. The very loose to loose sands and very soft to firm clay are generally restricted to the weathered zone, which extends to depths ranging from 0.8 to 1.2 m from the prevailing ground surface.

Refusal to augering or Standard Penetration Tests, occurred at depths ranging from 2.6 to 3.4 m below the prevailing ground surface, in 7 of the 9 boreholes. It is inferred that boulders and/or bedrock likely occur at these levels.

Groundwater was detected in eight of the nine boreholes, and the measured groundwater/cave-in levels range from 0.5 to 2.7± m below the prevailing ground surface (El. 181.7 m to El. 179.7 m). The shallow groundwater levels are likely attributable to infiltrated precipitation held by capillarity in the silt and silty fine sand, and in places, to water ponded at the surface at the borehole location.

The colour of the revealed soils changes from brown to grey at depths of 1.5 m and 2.3 m below the ground surface. This indicates that the soils in the upper zone of the stratigraphy have oxidized and the groundwater regime lies in the grey saturated soils. The groundwater level will be subject to seasonal fluctuation.

The geotechnical findings and constraints 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 future development, the topsoil should not be buried within the building envelope, or deeper than 1.2 m below the exterior finished grade. A fertility analysis should be carried out to confirm the suitability of the topsoil for use as a general planting material.
2. The weathered soils extend to depths ranging from 0.8 to 1.2± m from the prevailing ground surface. The highly weathered soils are weak and will consolidate under surcharge loads. To upgrade the weathered soils to engineered status suitable for normal footing construction, they must be subexcavated, aerated and properly compacted.
3. The sound natural soils are suitable for normal spread and strip footings and caisson foundation construction.
4. Due to the presence of topsoil and weathered soils, the footing subgrade must be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to assess its suitability for bearing the designed foundations.
5. A Class ‘B’ bedding is recommended for the underground services construction. The bedding material should consist of compacted 20-mm Crusher-Run Limestone. 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.
6. The tills are generally suitable for a 95% or + Standard Proctor compaction. The sands are wet and will require mixing with drier soils and/or aeration prior to structural compaction.
7. The tills contain occasional boulders. Extra effort and a properly equipped backhoe will be required for excavation. Boulders larger than 15 cm in size are not suitable for structural backfill.

8. Where rock excavation is required, this can be achieved by using a backhoe equipped with a rock-ripper or pneumatic hammering in the weathered zone of the bedrock. Excavation into the sound rock will likely require blasting to break up the sound rock mass. A consultant who specializes in rock blasting must be consulted.

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 for the proposed building foundations must be placed below the topsoil and weathered soils, onto the sound natural soils. The recommended soil pressures for use in the design of the normal spread and strip footings, together with the corresponding suitable founding levels are presented in Table 3.

Table 3 - Founding Levels

BH No.	**Maximum Allowable Soil Pressure (SLS)/ Factored Ultimate Soil Bearing Pressure (ULS) and Corresponding Founding Level**					
	75 kPa (SLS) 120 kPa (ULS)		**150 kPa (SLS) 250 kPa (ULS)**		**500 kPa (SLS) 1100 kPa (ULS)**	
	Depth (m)	**El. (m)**	**Depth (m)**	**El. (m)**	**Depth (m)**	**El. (m)**
2	1.6 or +	180.7 or -	2.0 or +	180.3 or -	3.0 or +	179.3 or -
3	-	-	1.2 or +	181.0 or -	1.5 or +	180.7 or -
5	1.8 or +	180.4 or -	-	-	2.3 or +	179.9 or-
7	-	-	1.2 or +	181.3 or -	2.2 or +	180.3 or -
8	1.2 or +	181.1 or -	1.8 or +	180.5 or -	3.0 or +	179.3 or -

If the limestone bedrock which commonly occurs in this area is encountered within the founding depths, a Maximum Allowable Rock Pressure (SLS) of 1500 kPa can be used for the design of normal spread and strip foundations founded on the bedrock.

Where the footing subgrade consists of wet sand, it must be protected by a concrete mud-slab immediately after inspection and approval to prevent construction traffic from impacting its structural integrity.

The foundation loads of the building can also be supported by drilled caisson foundations. The suitable founding levels for caisson foundations lie at depths of 3.0 m or + from the prevailing ground surface. To facilitate efficient subgrade inspection and cleaning, the caissons should be at least 80 cm in diameter and the excavation should be temporarily lined.

Where extended footings or stepped footings will be required in the moist natural soils, it may be more cost-effective to subexcavate to a size 30% larger than the designed footing width and fill with structural concrete up to the proposed footing elevation immediately after the suitable founding soil is exposed. In order to allow a minor amount of side sloughing of the incidental in situ material on the approved subgrade, the sequence of footing excavation, subgrade inspection and concreting must be carried out simultaneously. The stepped footings must be sloped at 7 vertical:10 horizontal.

The foundations subjected to weathering and in unheated areas should have an earth cover of at least 1.5 m for protection against frost action.

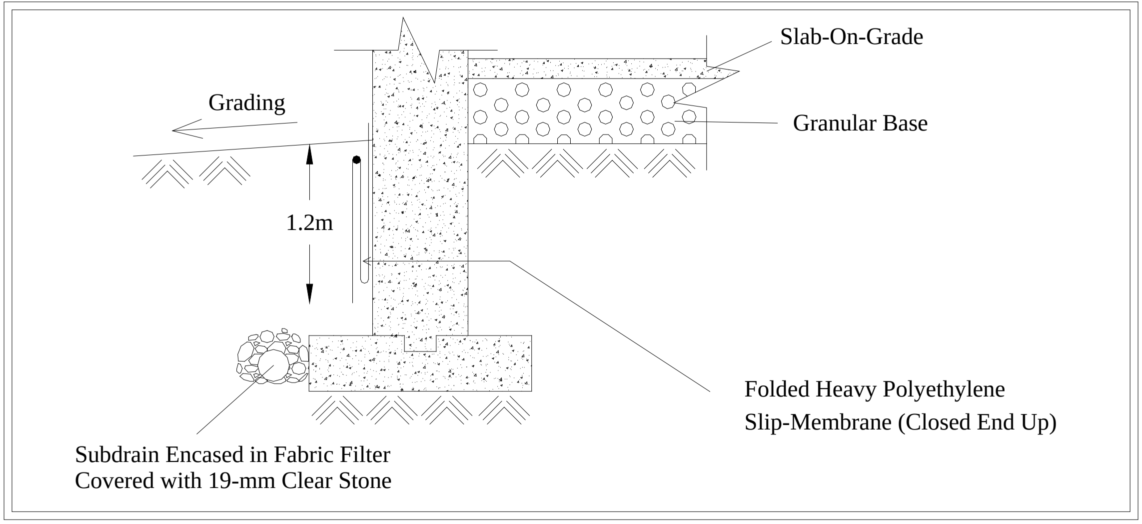
The recommended soil pressures incorporate a safety factor of 3 against shear failure of the underlying soils. The total and differential settlements of the footings are estimated to be 25 mm and 15 mm, respectively.

The footings must meet the requirements specified in the 2006 Ontario Building Code. As a guide, the structure should be designed to resist an earthquake force using Site Classification ‘C’ (very stiff soil).

Due to the presence of topsoil and weathered soils, the footing subgrade should be checked by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that the revealed conditions are compatible with the foundation design requirements.

The fine sand and silty fine sand are high in frost heave and soil-adfreezing potential. If these soils are to be used for the foundation backfill, the foundation walls should be shielded by a polyethylene slip-membrane for protection against soil adfreezing. The recommended measures are schematically illustrated in Diagram 1.

Diagram 1 - Frost Protection Measures (Foundations)



The exterior grading adjacent to the building must be designed to slope away from the building to prevent any water ponding close to the building envelope.

6.2 Slab-On-Grade

The subgrade for the slab-on-grade must consist of sound natural soil or properly compacted inorganic engineered fill.

The subgrade should be inspected and assessed by proof-rolling prior to slab-on-grade construction. Where the subgrade consists of loose soils or earth containing serious topsoil inclusions, it should be subexcavated and replaced with inorganic material uniformly compacted to 98% or + of its maximum Standard Proctor dry density.

Any new material for raising the grade should consist of organic-free soil compacted to at least 98% of its maximum Standard Proctor dry density.

The slab should be constructed on a granular base 20 cm thick, consisting of 20-mm Crusher-Run Limestone, or equivalent, compacted to 100% of its maximum Standard Proctor dry density.

A Modulus of Subgrade Reaction of 30 MPa/m can be used for the design of the floor slab.

If the subgrade has been loosened due to construction traffic, it must be proof-rolled prior to placement of the granular base.

Where the subgrade is wet or where groundwater seepage is encountered during slab-on-grade construction, floor subdrains will be required. The subdrains should be encased in fabric to protect them against blockage by silting and they must be connected to a positive outlet. The necessity to implement these measures should be further assessed by a geotechnical engineer at the time of construction.

Insulation consisting of 60-mm Styrofoam, or equivalent, should be provided beneath the floor slab in non-heated areas for protection against frost heave. The slab at the entrance doors should be insulated with 60-mm Styrofoam, or its thermal equivalent, extending 3.0 m internally. This measure is to prevent cold drafts in the winter months from inducing frost action on the subgrade and thereby causing damage to the floor slab.

6.3 **Underground Services**

The subgrade for the underground services should consist of sound natural soil or properly compacted inorganic earth fill. Where loose soil is encountered, it must be properly densified or replaced with properly compacted bedding material.

Where services are to 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. The soil parameters given in Section 6.8 can be used for the design of the trench box and/or shoring structure.

A Class ‘B’ bedding is recommended for the underground services construction. The bedding material should consist of compacted 20-mm Crusher-Run Limestone, or equivalent. In water-bearing sandy material or silts, where extensive dewatering is required, the pipe joints must be leak-proof or wrapped with a waterproof membrane.

In order to prevent pipe floatation when the sewer 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.

Openings to subdrains and catch basins should be shielded by a fabric filter to prevent blockage by silting.

Since the occurring soils have moderate to moderately low corrosivity to buried metal, the water main should be protected against corrosion. For estimation purposes for the anode weight requirements, the estimated electrical resistivity for the disclosed soils can be used. This, however, should be confirmed by testing the soil along the water main alignment at the time of construction.

6.4 Backfilling in Trenches and Excavated Areas

The backfill in the trenches and excavated areas should be compacted to at least 95% and 98% of its maximum Standard Proctor dry density below the pavement and slab-on-grade, respectively. In the zone within 1.0 m below the pavement 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. Wetting of the sound tills will likely be required to achieve this requirement.

In normal construction practice, the problem areas of pavement settlement largely occur adjacent to foundation walls, columns, 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 sand backfill will have to be flooded for a period of at least 1 day.

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 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, seepage collars should be provided.

6.5 Perimeter Walls at Truck Loading Dock

The perimeter walls at the loading area must be designed to sustain the lateral earth pressure calculated using the soil parameters given in Section 6.8. Any surcharge loads adjacent to the walls must also be considered in the design of the perimeter walls. The backfill against the wall must consist of free-draining granular fill. The loading dock must be provided with subdrains connected to a positive outlet to relieve possible groundwater seepage.

A concrete apron is recommended at the truck loading area and ramp. The apron should be designed using the Modulus of Subgrade Reaction given for the design of the slab-on-grade, and it should be insulated with 60-mm Styrofoam, or equivalent, on a compacted granular base 300 mm thick, consisting of 20-mm Crusher-Run Limestone, or equivalent.

6.6 Pavement Design

The subgrade of the proposed pavement, according to the borehole findings, will generally consist of silty material, which is highly susceptible to frost heave. In order to minimize the frost damage to the pavement, the pavement design presented in Table 4 is recommended.

Table 4 - Pavement Design

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL-3
Asphalt Binder	50	HL-8
Granular Base	150	Granular ‘A’

Table 4 - Pavement Design (Cont'd)

Course	Thickness (mm)	OPS Specifications
Granular Sub-base		Granular 'B'
Parking	350	
Access Roads	500	

In preparation of the subgrade, the topsoil must be removed and the surface should be proof-rolled. The badly weathered soil should be subexcavated, sorted free of any concentrated topsoil and deleterious materials, and properly compacted. Any soft subgrade should be subexcavated and properly compacted, or replaced with uniformly compacted, inorganic earth fill or granular material.

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

In the zone within 1.0 m below the pavement subgrade, the backfill should be compacted to at least 98% of its maximum Standard Proctor dry density, with the water content 2% to 3% drier than the optimum. In the lower zone, a 95% or + Standard Proctor compaction is considered adequate.

Subdrains connected to the catch basins in the paved areas must be installed to minimize the risk of frost heave which would affect the structural stability of the roads.

The pavement subgrade will suffer a strength regression if water is allowed to infiltrate prior to paving. The following measures should therefore be incorporated into the construction procedures and road design:

- If the pavement construction does not immediately follow the trench backfilling, the subgrade should be properly crowned and smooth-rolled to allow interim precipitation to be properly drained.
- Site areas adjacent to the pavement should be properly graded to prevent the ponding of large amounts of water during the interim construction period.
- If the pavement is to be constructed during wet seasons and extremely soft subgrade occurs, the granular sub-base may require thickening. This can be assessed during construction.
- In order to prevent infiltrated precipitation from seeping into the granular bases (since this may inflict frost damage on the flexible pavement), an intercept subdrain system should be installed along the perimeter where runoff may drain onto the pavement. In the paved areas, catch basins should be provided; they should drain into the storm sewer or suitable outlet.

6.7 Sidewalks, Interlocking Stone Pavement and Landscaping

Interlocking stone pavement, sidewalks and landscaping structures in areas sensitive to frost-induced ground movement, such as entrances, must be constructed on a free-draining, non-frost-susceptible granular material such as Granular ‘B’. It must extend to 1.5 m below the pavement surface and be provided with positive drainage such as weeper subdrains connected to manholes or catch basins. Alternatively, the landscaping structures, sidewalks and interlocking stone pavement should be insulated with 60-mm Styrofoam, or equivalent.

6.8 Soil Parameters

The recommended soil parameters for the project design are given in Table 5.

Table 5 - Soil Parameters

Unit Weight and Bulk Factor

	Unit Weight (kN/m^3)	**Estimated Bulk Factor**	
	Bulk	**Loose**	**Compacted**
Fine Sand and Silty Fine Sand	20.5	1.10	0.95
Silty Clay Till	22.0	1.33	1.05
Silty Sand Till	22.5	1.25	1.05
Fine to Coarse Sand	20.0	1.20	1.03

Lateral Earth Pressure Coefficients

	Active K_a	**At Rest K_o**	**Passive K_p**
Fine Sand and Silty Fine Sand	0.50	0.55	2.33
Silty Clay Till	0.45	0.50	2.50
Silty Sand Till	0.40	0.45	2.86
Fine to Coarse Sand	0.33	0.40	3.00

Coefficient of Friction

Between Concrete and Granular Base	0.60
Between Concrete and Sound Natural Soil	0.40

Maximum Allowable Soil Pressures (SLS)
For Thrust Block Design

Sound Natural Soil	100 kPa

6.9 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91.

For excavation purposes, the types of soils are classified in Table 6.

Table 6 - Classification of Soils for Excavation

Material	Type
Sound Tills	2
Weathered Soils and Dewatered Sands	3
Water-bearing Sands	4

Excavation into the hard and very dense tills containing boulders will require extra effort and the use of a heavy, properly equipped backhoe. If rock excavation is required, this can be achieved by using a backhoe equipped with a rock-ripper or pneumatic hammering in the weathered zone of the bedrock. Excavation into the sound rock will likely require blasting to break up the sound rock mass. A consultant who specializes in rock blasting must be consulted.

The yield of groundwater is expected to be small to moderate from the tills and will be controllable by normal pumping from sumps. Deep excavations into the water-bearing sands will need to be stabilized by vigorous pumping from closely-spaced sumps and/or the use of a well-point dewatering system; alternatively, sheeting of the excavation will be required to control the groundwater. The appropriate dewatering method should be further assessed by test pits and/or test pumping prior to the project construction.

In order to provide a stable base for footing and underground services construction; the water level in the water-bearing sands must be lowered to 0.5 m below the working base of the 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.

7.0 LIMITATIONS OF REPORT

It should be noted that chemical analyses of soil samples was conducted to determine its environmental quality, and a Phase One Environmental Site Assessment was carried out for the property. The results of the chemical analyses were presented in our letter dated November 23, 2011, and the findings of the Phase One Assessment were presented in our report dated November 24, 2011. Therefore, this report deals only with a study of the geotechnical aspects of the proposed project.

This report was prepared by Soil Engineers Ltd. for the account of Hen & Chickens Brewing Co. Ltd., and for review by its designated consultants and government agencies. The material in it reflects the judgement of Basim Al-Ali, P.Eng., and Victor S. Chan, 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, 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.

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12/21/2011

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:

1. SAMPLE TYPES

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

2. PENETRATION RESISTANCE/'N'

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 'O'

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

3. SOIL DESCRIPTION

a) 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

b) 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

c) 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
1 lb. = 0.453 kg
1 inch = 25.4 mm
1 ksf = 47.88 kN/m^2

JOB NO: 1110-S118

LOG OF BOREHOLE NO: 1

FIGURE NO: 1

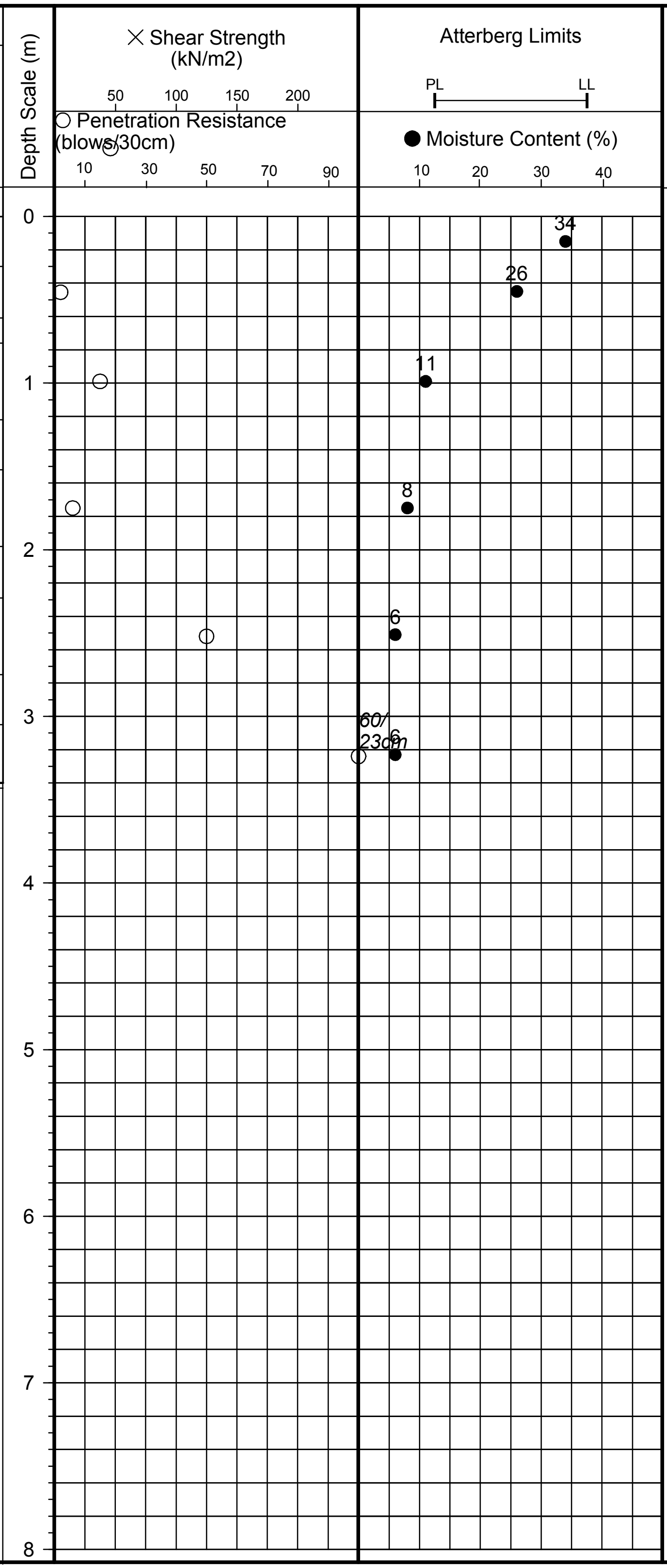
JOB DESCRIPTION: Proposed Industrial Development

JOB LOCATION: Mountain Road, Town of Collingwood

METHOD OF BORING: Flight-Auger

DATE: November 14, 2011

Depth Elev. (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
0.0	Ground Surface			
182.3	**30 cm TOPSOIL**	1A	DO	-
	Brown, very loose **SILTY FINE SAND** some clay occ. silt seams and layers	1B	DO	2
0.8 181.5	Brown, stiff **SILTY CLAY, Till** sandy, a tr. of gravel occ. wet sand and silt seams and layers, cobbles and boulders weathered	2	DO	15
1.5 180.8	Grey, loose to very dense **SILTY SAND, Till** some clay, a tr. of gravel occ. wet sand and silt seams and layers, cobbles and boulders	3	DO	6
		4	DO	50
		5	DO	60/ 23
3.4 178.9	**Refusal to SPT** **END OF BOREHOLE**			



WATER LEVEL

∇

W.L. @ El. 180.2 m on completion

JOB NO: 1110-S118

LOG OF BOREHOLE NO: 2

FIGURE NO: 2

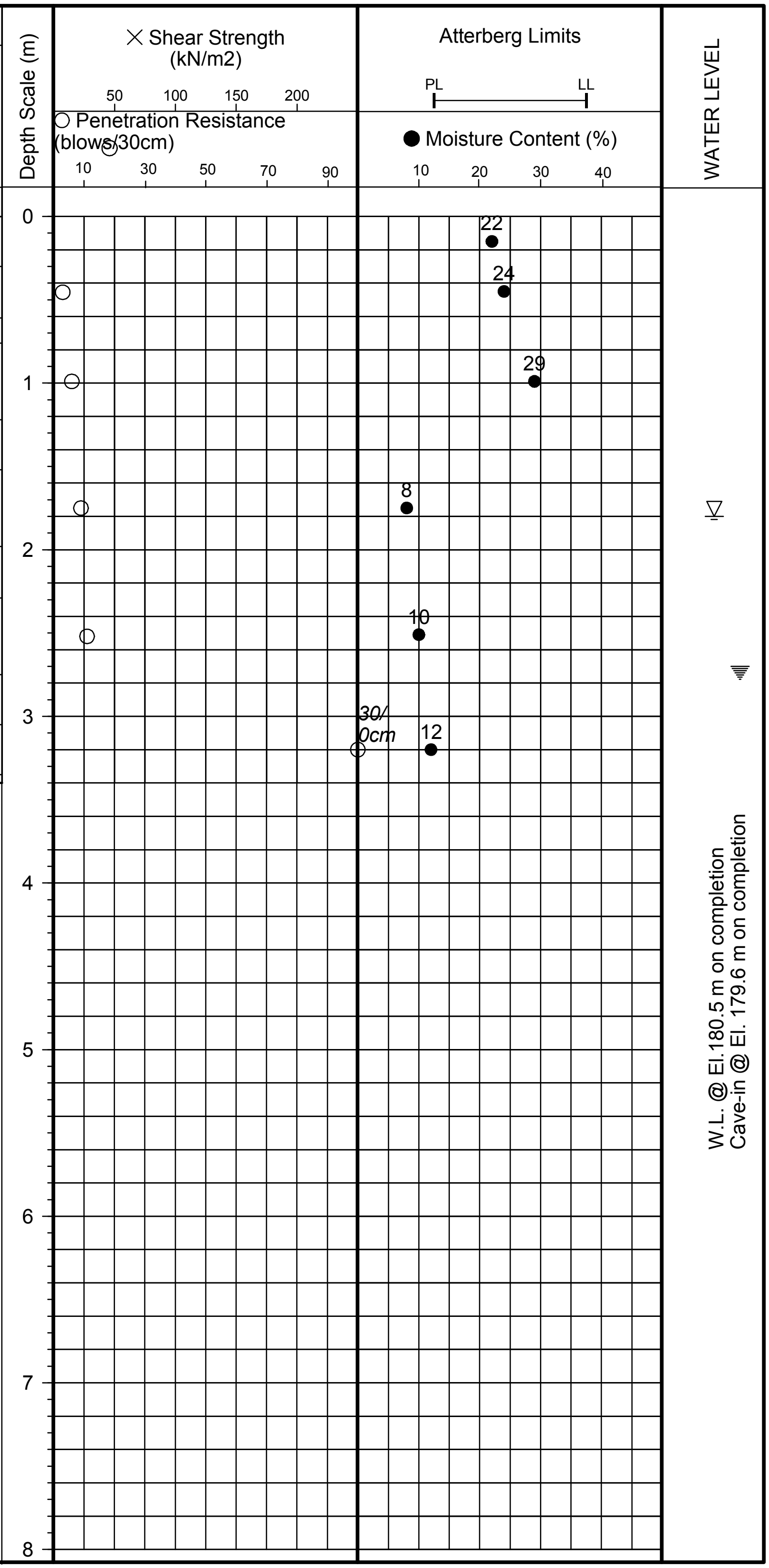
JOB DESCRIPTION: Proposed Industrial Development

JOB LOCATION: Mountain Road, Town of Collingwood

METHOD OF BORING: Flight-Auger

DATE: November 14, 2011

Depth Elev. (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
0.0	Ground Surface			
182.3	**30 cm TOPSOIL**	1A	DO	-
	Brown, very loose **SILTY FINE SAND** some clay occ. silt seams and layers	1B	DO	3
0.8 181.5	Brown, firm, weathered **SILTY CLAY, Till** sandy, a tr. of gravel occ. wet sand and silt seams and layers, cobbles and boulders	2	DO	6
1.5 180.8	Grey, loose **SILTY SAND, Till** some clay, a tr. of gravel occ. wet sand and silt seams and layers, cobbles and boulders	3	DO	9
2.3 180.0	Brown, compact to very dense **FINE TO COARSE SAND** some silt	4	DO	11
		5	DO	30/0
3.4 178.9	**Refusal to Augering** **END OF BOREHOLE**			



W.L. @ El.180.5 m on completion
Cave-in @ El. 179.6 m on completion

JOB NO: 1110-S118

LOG OF BOREHOLE NO: 3

FIGURE NO: 3

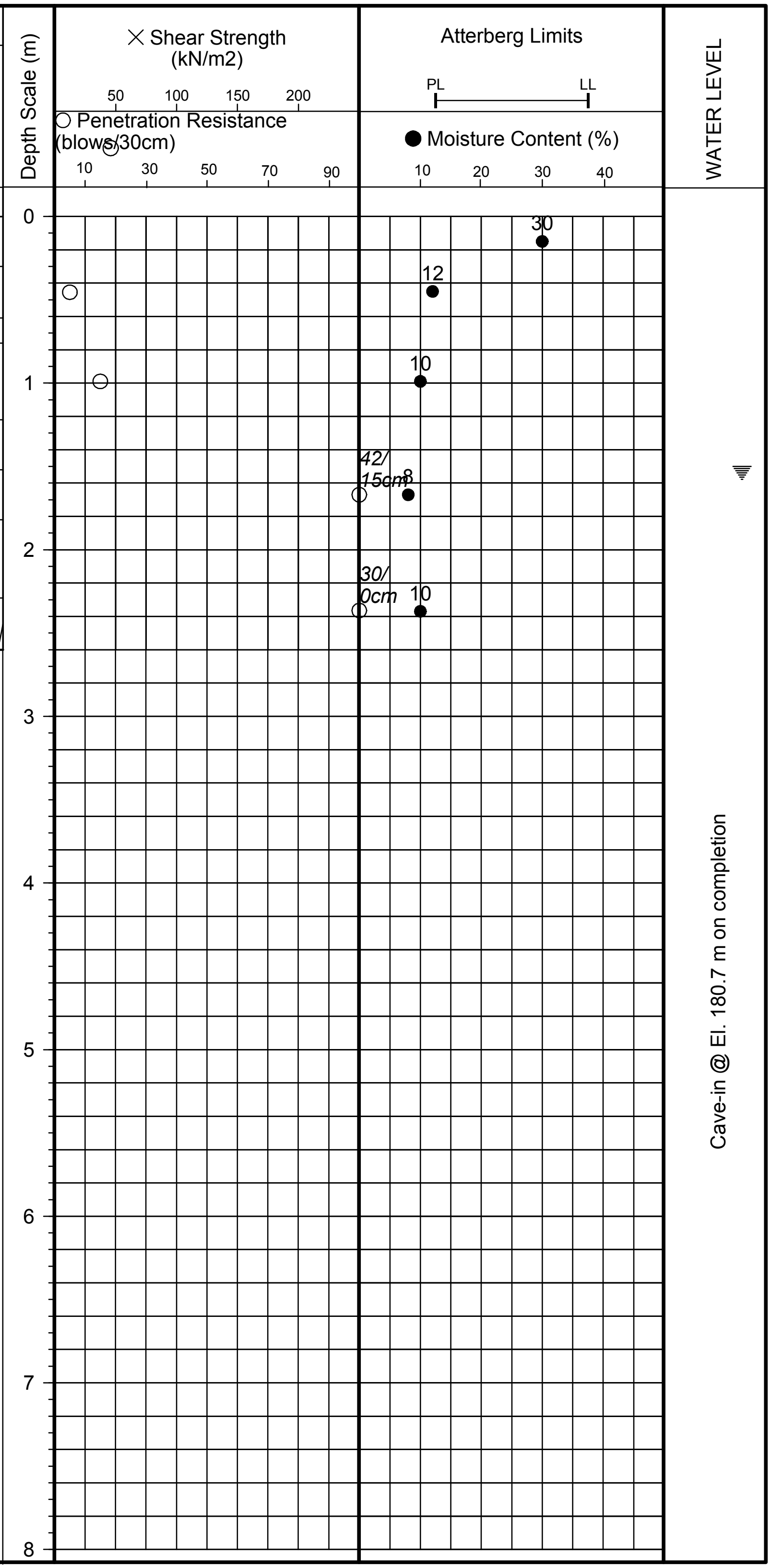
JOB DESCRIPTION: Proposed Industrial Development

JOB LOCATION: Mountain Road, Town of Collingwood

METHOD OF BORING: Flight-Auger

DATE: November 14, 2011

Depth Elev. (m)	SOIL DESCRIPTION	SAMPLES		
		Number	Type	N-Value
0.0	Ground Surface			
182.2	**28 cm TOPSOIL**	1A	DO	-
	Brown, loose **FINE SAND** some silt occ. silt seams and layers	1B	DO	5
0.8 181.4	Brown, compact **SILTY SAND, Till** traces of clay and gravel occ. wet sand and silt seams and layers, cobbles and boulders weathered	2	DO	15
1.5 180.7	Brown, compact to very dense **FINE TO COARSE SAND** some silt, a tr. of gravel occ. rock fragments	3	DO	42/15
		4	DO	30/0
2.6 179.6	**Refusal to Augering** **END OF BOREHOLE**			



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JOB NO: 1110-S118

LOG OF BOREHOLE NO: 4

FIGURE NO: 4

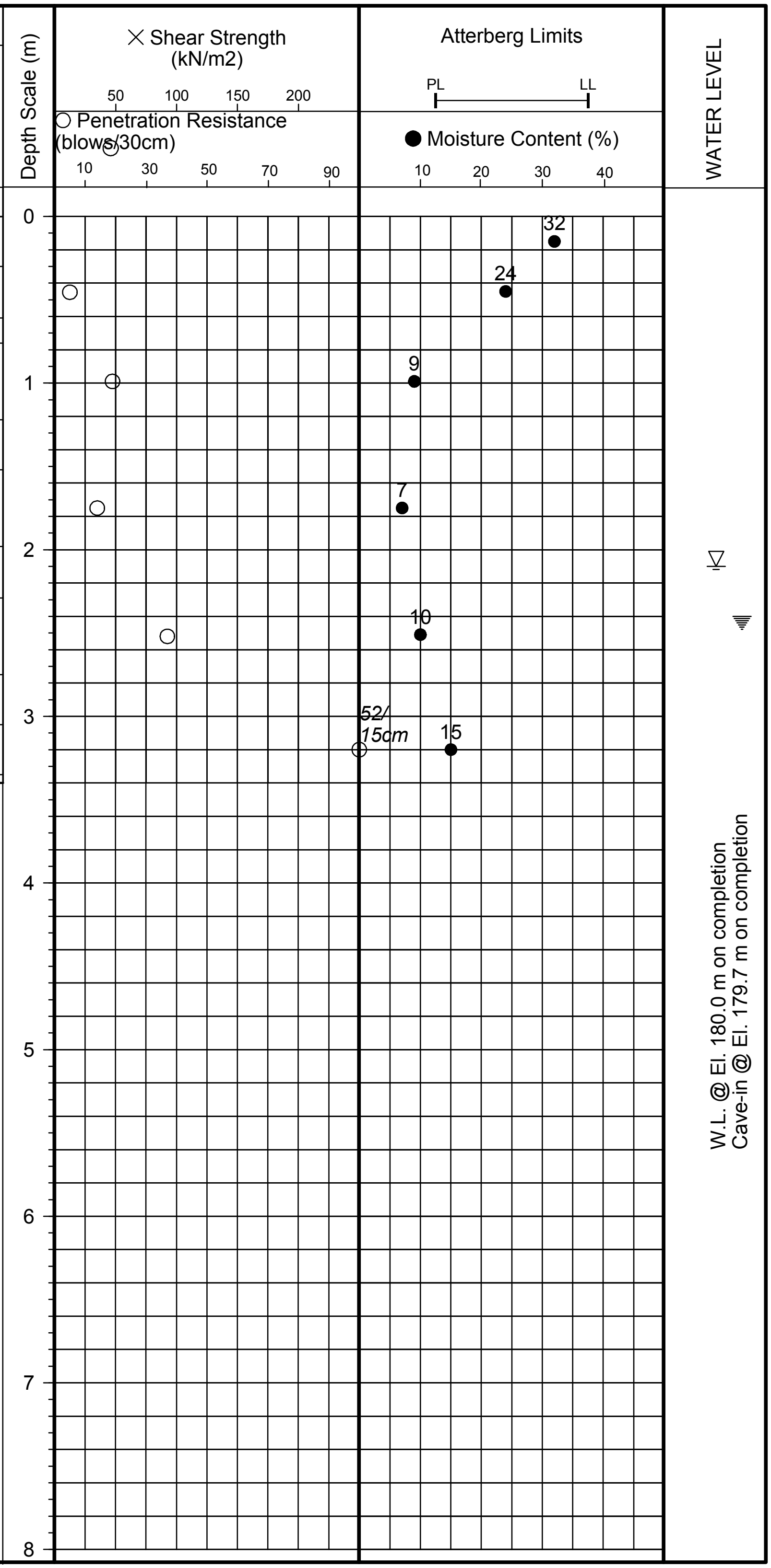
JOB DESCRIPTION: Proposed Industrial Development

JOB LOCATION: Mountain Road, Town of Collingwood

METHOD OF BORING: Flight-Auger

DATE: November 14, 2011

Depth Elev. (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
0.0	Ground Surface			
182.1	**25 cm TOPSOIL**	1A	DO	-
	Brown, loose **FINE SAND** some silt occ. silt seams and layers	1B	DO	5
0.8 181.3	Stiff to very stiff **SILTY CLAY, Till** sandy, a tr. of gravel occ. wet sand and silt seams and layers, cobbles and boulders weathered brown grey	2	DO	19
		3	DO	14
2.0 180.1	Brown, dense **SILTY SAND, Till** a tr. of clay and gravel occ. wet sand and silt seams and layers, cobbles and boulders	4	DO	37
3.0 179.1	Brown, compact to very dense **FINE TO COARSE SAND** a tr. of silt	5	DO	52/ 15
3.4 178.7	**END OF BOREHOLE**			



W.L. @ El. 180.0 m on completion

Cave-in @ El. 179.7 m on completion

Soil Engineers Ltd.

JOB NO: 1110-S118

LOG OF BOREHOLE NO: 5

FIGURE NO: 5

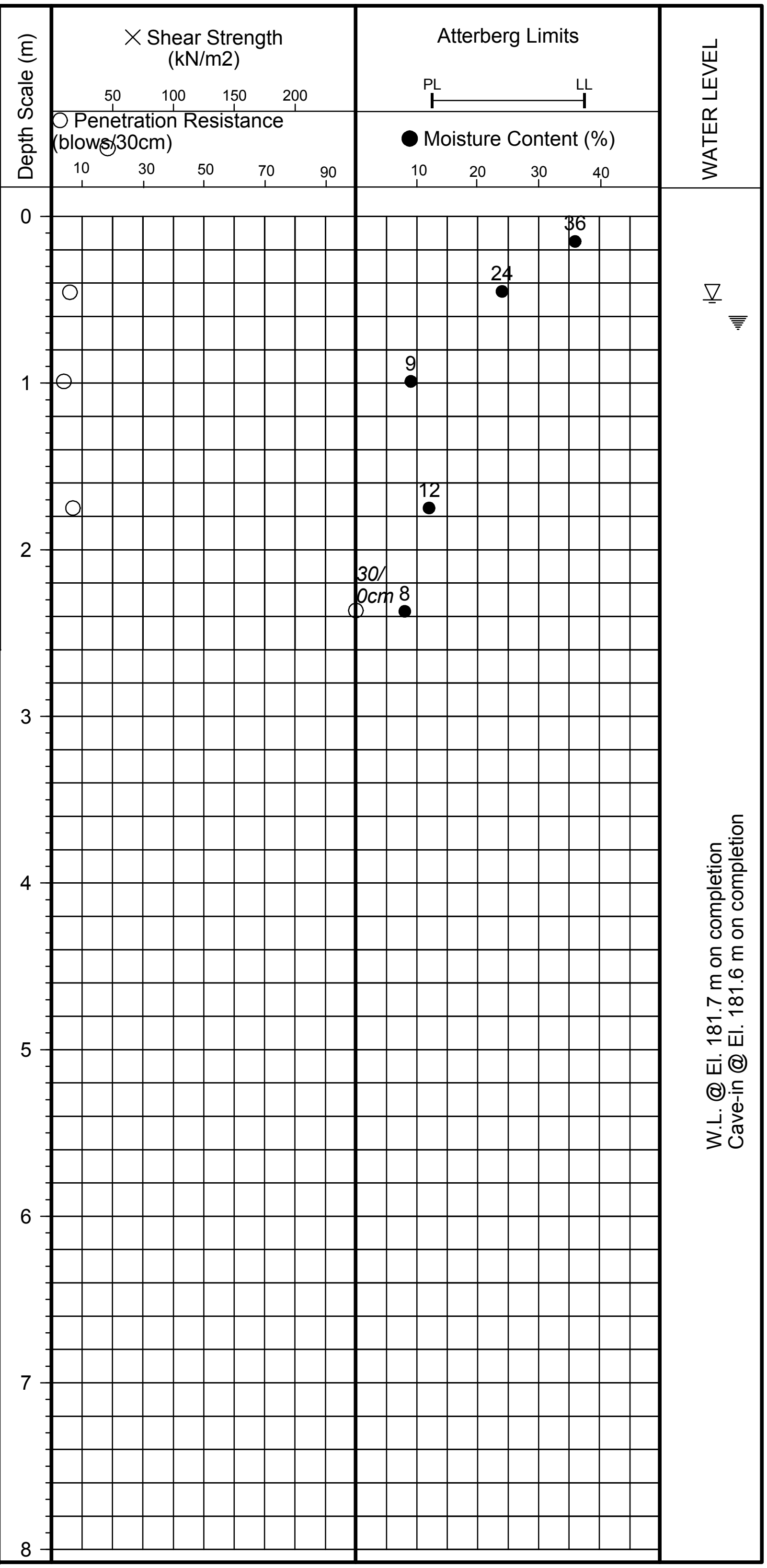
JOB DESCRIPTION: Proposed Industrial Development

JOB LOCATION: Mountain Road, Town of Collingwood

METHOD OF BORING: Flight-Auger

DATE: November 14, 2011

Depth Elev. (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
0.0	Ground Surface			
182.2	**23 cm TOPSOIL**	1A	DO	-
	Brown, loose **FINE SAND** some silt occ. silt seams and layers	1B	DO	6
0.8 181.4	Very loose to very dense **SILTY SAND, Till** weathered	2	DO	4
	some clay, a tr. of gravel occ. wet sand and silt seams and layers, cobbles and boulders	3	DO	7
	brown grey	4	DO	30/0
2.6 179.6	**Refusal to Augering** **END OF BOREHOLE**			



W.L. @ El. 181.7 m on completion
Cave-in @ El. 181.6 m on completion

JOB NO: 1110-S118

LOG OF BOREHOLE NO: 6

FIGURE NO: 6

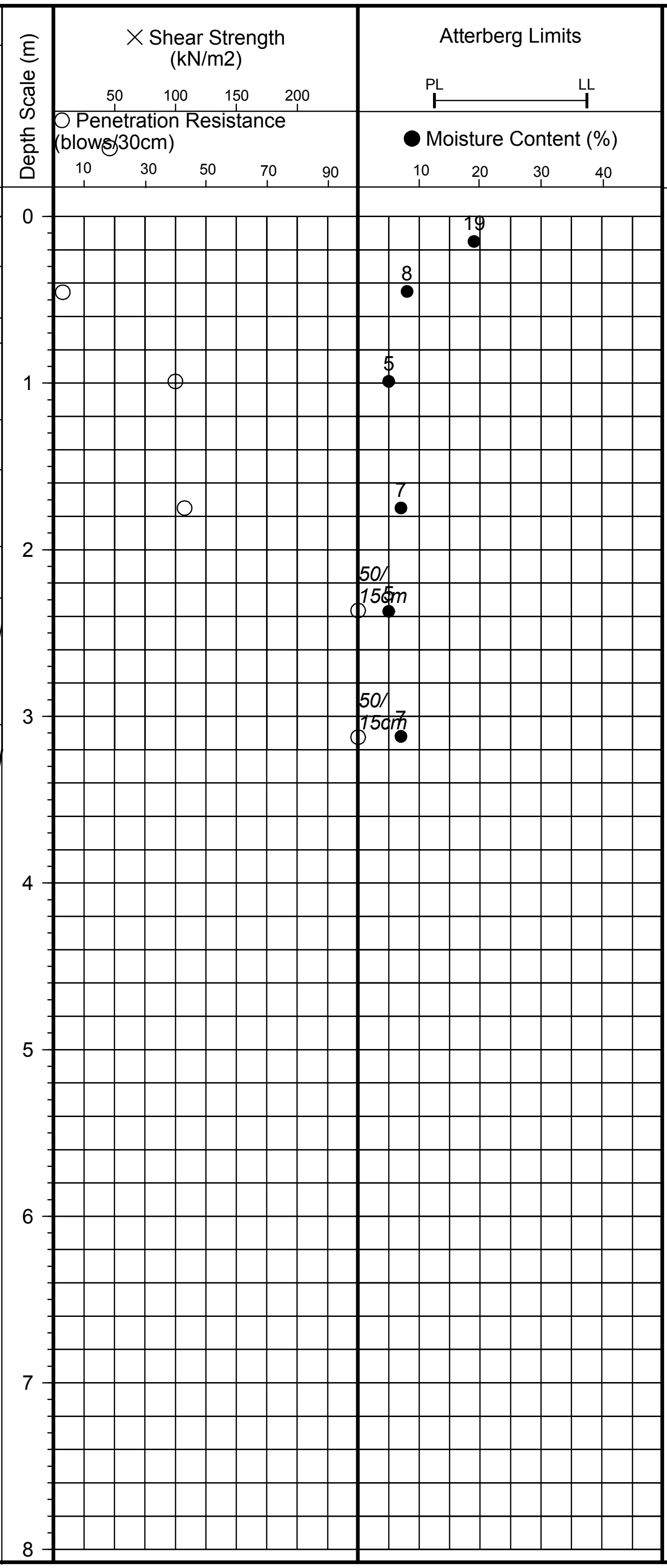
JOB DESCRIPTION: Proposed Industrial Development

JOB LOCATION: Mountain Road, Town of Collingwood

METHOD OF BORING: Flight-Auger

DATE: November 14, 2011

Depth Elev. (m)	SOIL DESCRIPTION	SAMPLES		
		Number	Type	N-Value
0.0	Ground Surface			
183.0	**25 cm TOPSOIL**	1A	DO	-
	Soft to hard	1B	DO	3
	SILTY CLAY, Till weathered	2	DO	40
	sandy, a tr. of gravel occ. wet sand and silt seams and layers, cobbles and boulders	3	DO	43
	brown grey	4	DO	50/ 15
3.2		5	DO	50/ 15
179.8	**END OF BOREHOLE**			



WATER LEVEL

Dry on completion

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JOB NO: 1110-S118

LOG OF BOREHOLE NO: 7

FIGURE NO: 7

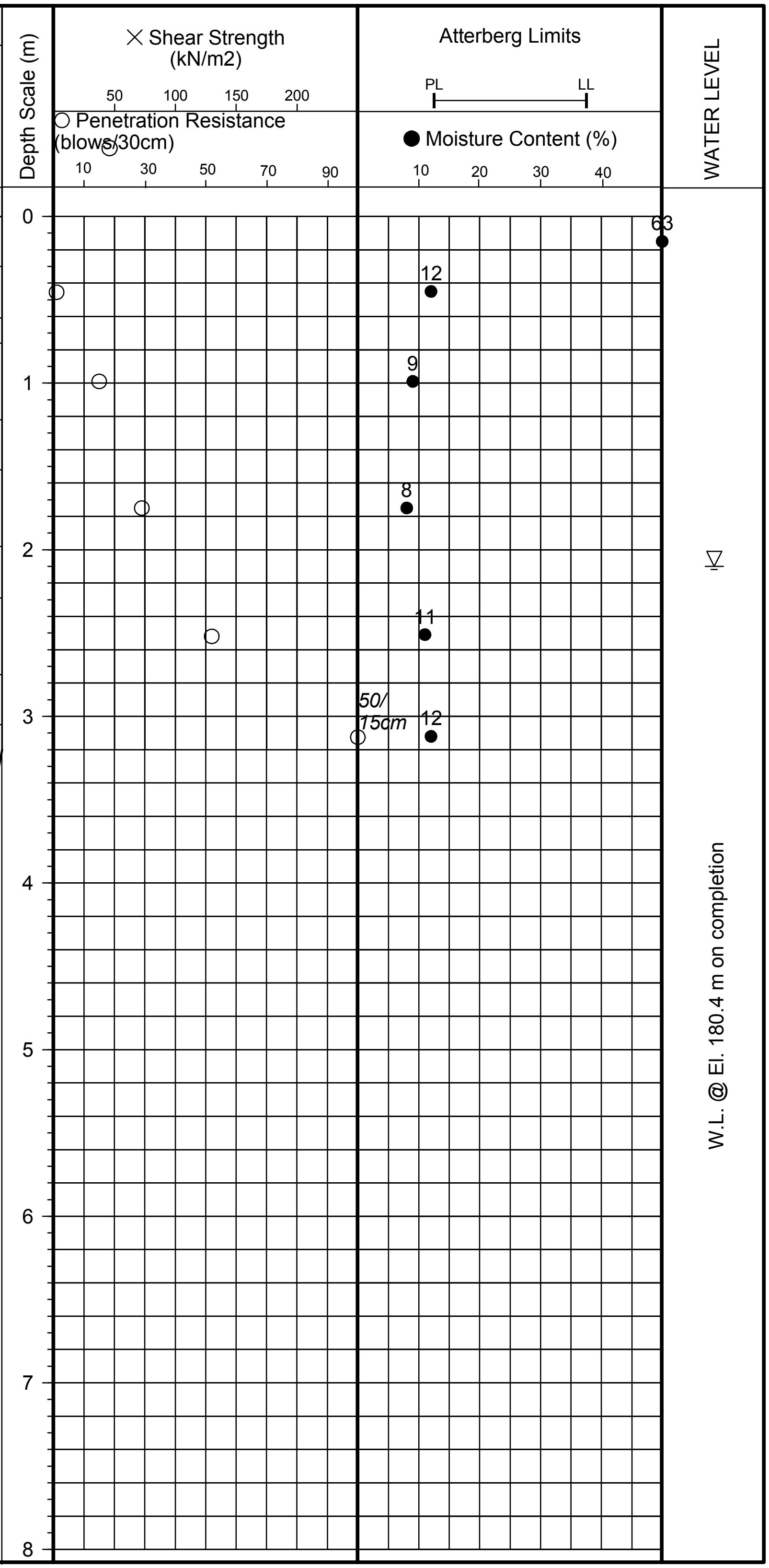
JOB DESCRIPTION: Proposed Industrial Development

JOB LOCATION: Mountain Road, Town of Collingwood

METHOD OF BORING: Flight-Auger

DATE: November 14, 2011

Depth Elev. (m)	SOIL DESCRIPTION	SAMPLES		
		Number	Type	N-Value
0.0	Ground Surface			
182.5	**30 cm TOPSOIL**	1A	DO	-
	Very soft to hard	1B	DO	1
	SILTY CLAY, Till weathered	2	DO	15
	brown grey sandy, a tr. of gravel occ. wet sand and silt seams and layers, cobbles and boulders	3	DO	29
		4	DO	52
3.2 179.3	**Refusal to Augering** **END OF BOREHOLE**	5	DO	50/ 15



W.L. @ El. 180.4 m on completion

Soil Engineers Ltd.

JOB NO: 1110-S118

LOG OF BOREHOLE NO: 8

FIGURE NO: 8

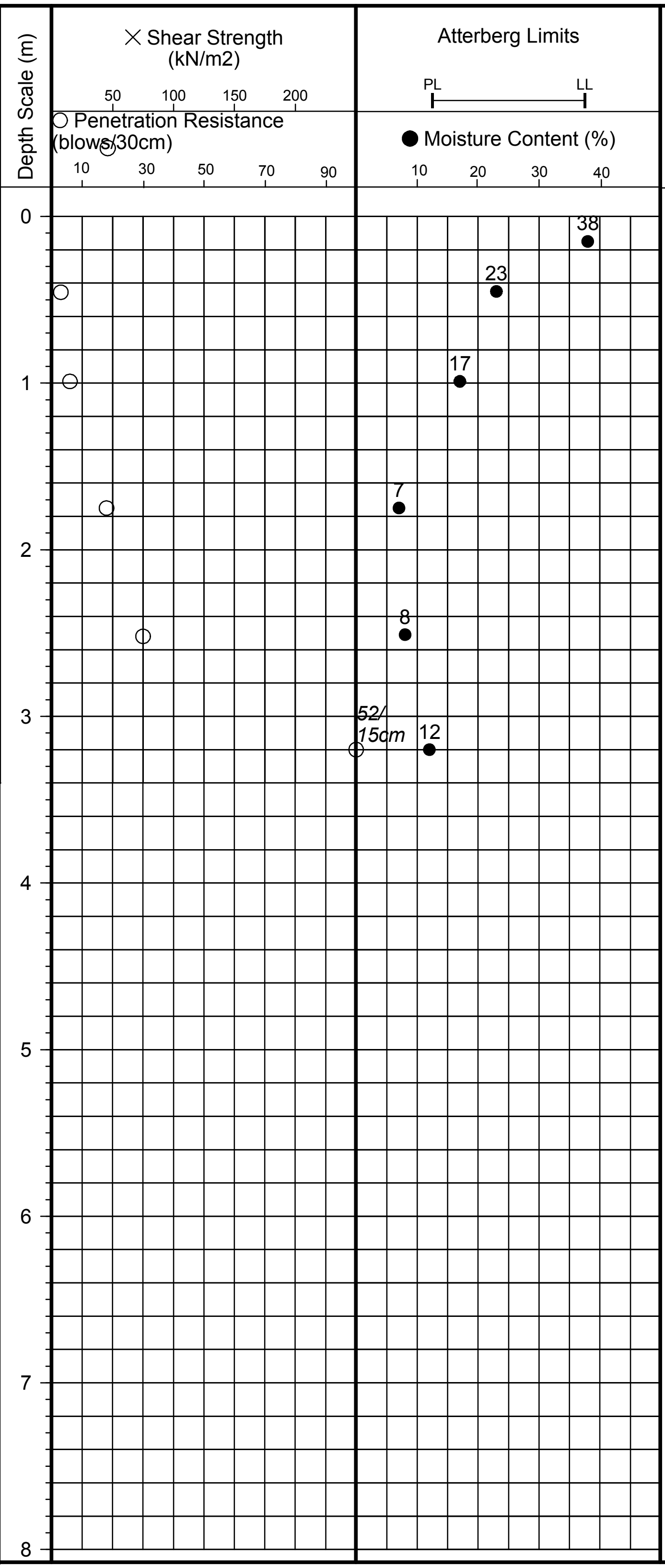
JOB DESCRIPTION: Proposed Industrial Development

JOB LOCATION: Mountain Road, Town of Collingwood

METHOD OF BORING: Flight-Auger

DATE: November 14, 2011

Depth Elev. (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value
0.0	Ground Surface			
182.3	**25 cm TOPSOIL**	1A	DO	-
	Brown, very loose **FINE SAND** occ. silt seams and layers	1B	DO	3
0.8 181.5	Firm to hard weathered	2	DO	6
	SILTY CLAY, Till brown grey	3	DO	18
	sandy, a tr. of gravel occ. wet sand and silt seams and layers, cobbles and boulders	4	DO	30
		5	DO	52/ 15
3.4 178.9	**Refusal to Augering** **END OF BOREHOLE**			



WATER LEVEL

∇

W.L. @ El. 181.7 m on completion

Soil Engineers Ltd.

JOB NO: 1110-S118

LOG OF BOREHOLE NO: 9

FIGURE NO: 9

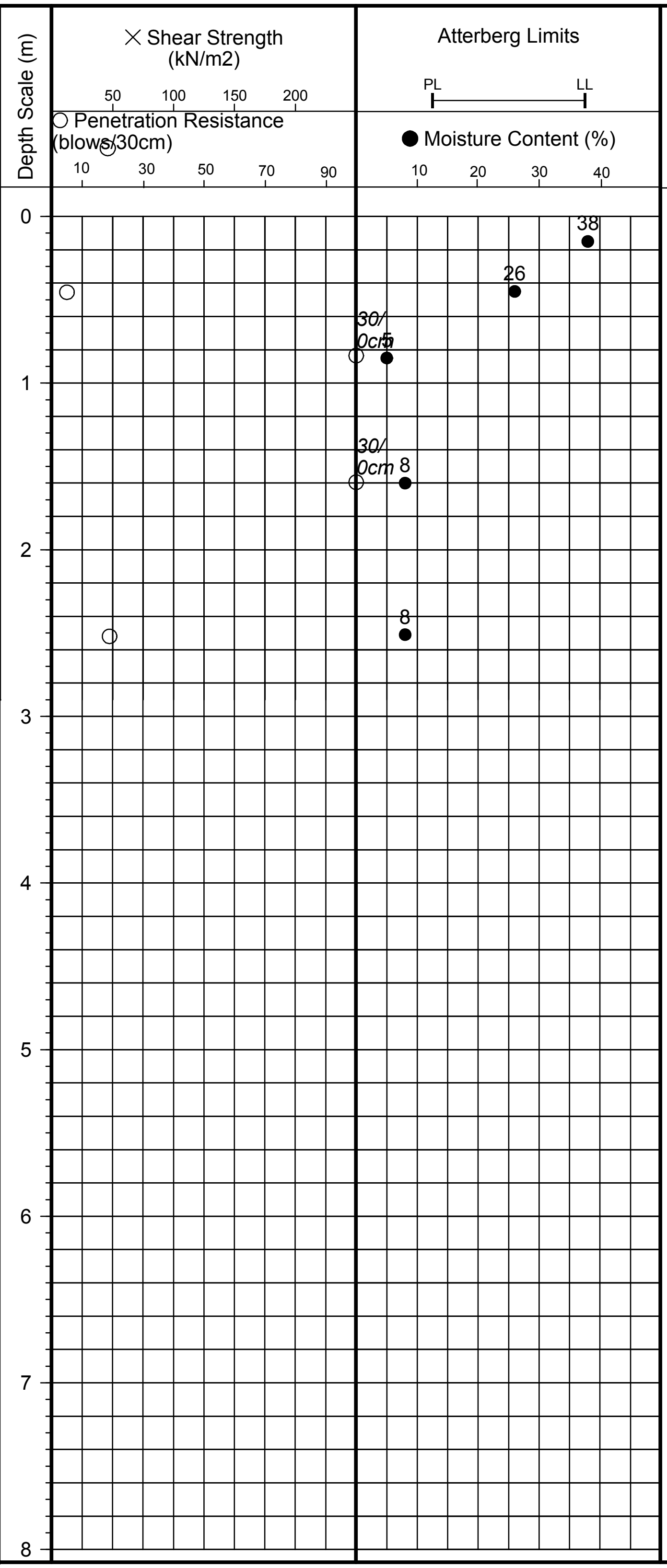
JOB DESCRIPTION: Proposed Industrial Development

JOB LOCATION: Mountain Road, Town of Collingwood

METHOD OF BORING: Flight-Auger

DATE: November 14, 2011

Depth Elev. (m)	SOIL DESCRIPTION	SAMPLES Number	SAMPLES Type	SAMPLES N-Value
0.0	Ground Surface			
182.3	**25 cm TOPSOIL**	1A	DO	-
	Brown, soft to hard	1B	DO	5
	weathered **SILTY CLAY, Till**	2	DO	30/ 0
	sandy, a tr. of gravel occ. wet sand and silt seams and layers, cobbles and boulders	3	DO	30/ 0
2.3 180.0	Grey, compact **SILTY SAND, Till** a tr. of clay and gravel occ. wet sand and silt seams and layers, cobbles and boulders	4	DO	19
2.9 179.4	**Refusal to Augering** **END OF BOREHOLE**			



WATER LEVEL

∇

W.L. @ El. 179.7 m on completion

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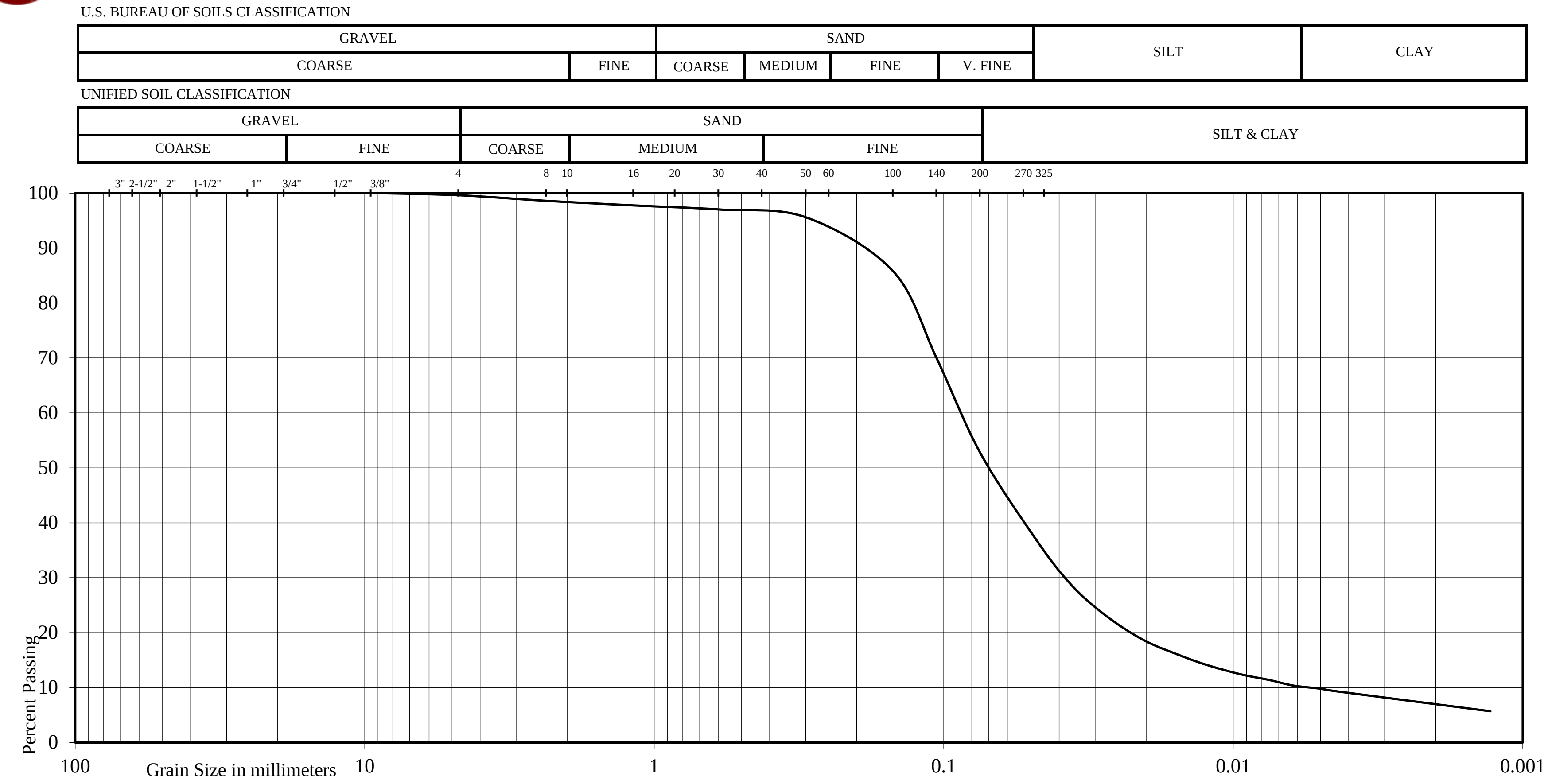
GRAIN SIZE DISTRIBUTION

Reference No: 1110-S118



Project: Proposed Industrial Development

Location: Mountain Road, Town of Collingwood

Borehole No: 1

Sample No: 1B

Depth (m): 0.4

Elevation (m): 181.9

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = -

Estimated Permeability (cm./sec.) = 10^{-5}

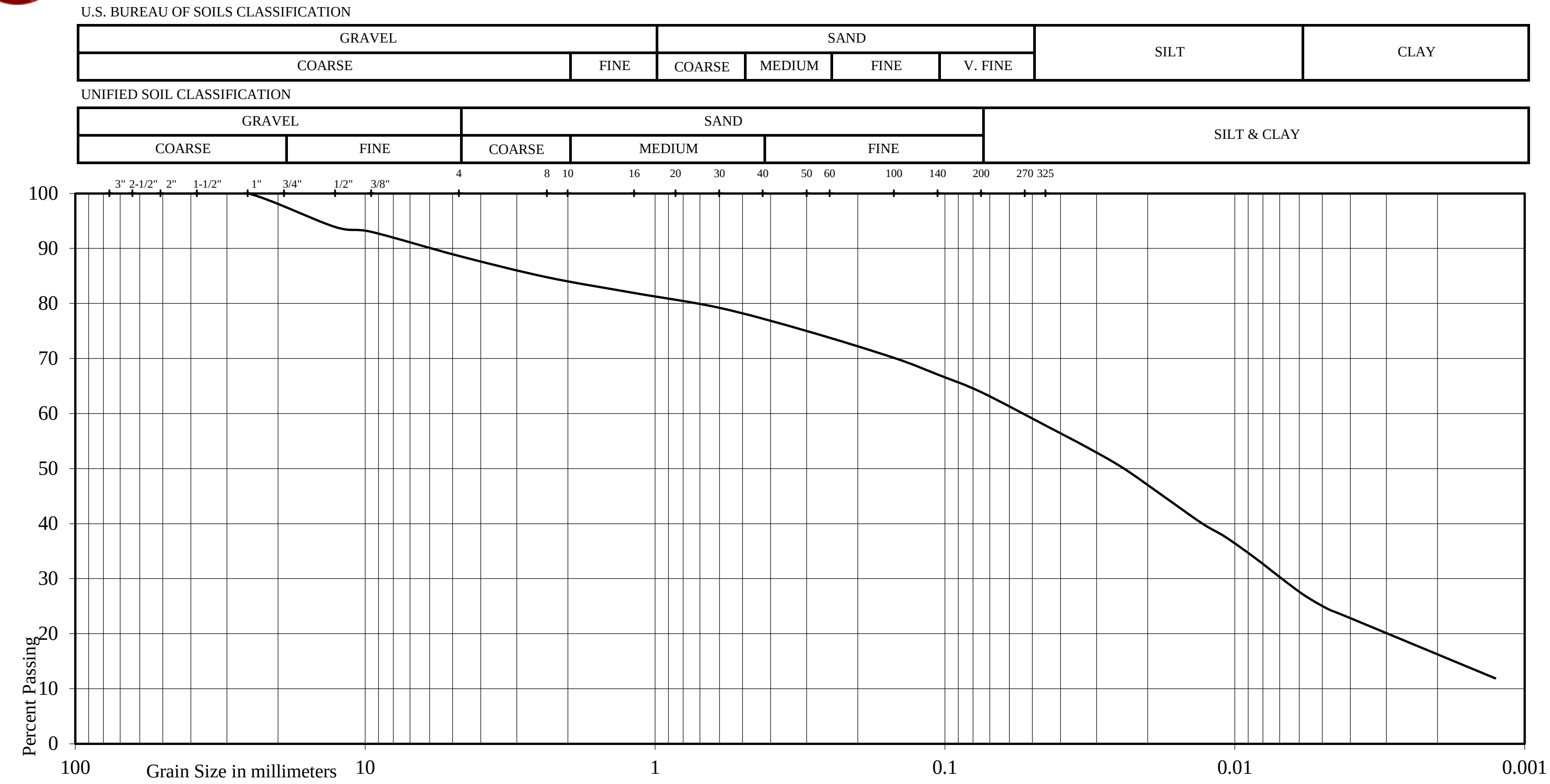
Classification of Sample [& Group Symbol]: SILTY FINE SAND
some clay

Figure: 10

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 1110-S118



Project:	Proposed Industrial Development		
Location:	Mountain Road, Town of Collingwood	Liquid Limit (%) =	24
		Plastic Limit (%) =	15
Borehole No:	7	Plasticity Index (%) =	9
Sample No:	3	Moisture Content (%) =	-
Depth (m):	1.8	Estimated Permeability	
Elevation (m):	180.7	(cm./sec.) =	10^{-7}

Classification of Sample [& Group Symbol]: SILTY CLAY, Till
sandy, a trace of gravel

Figure: 11

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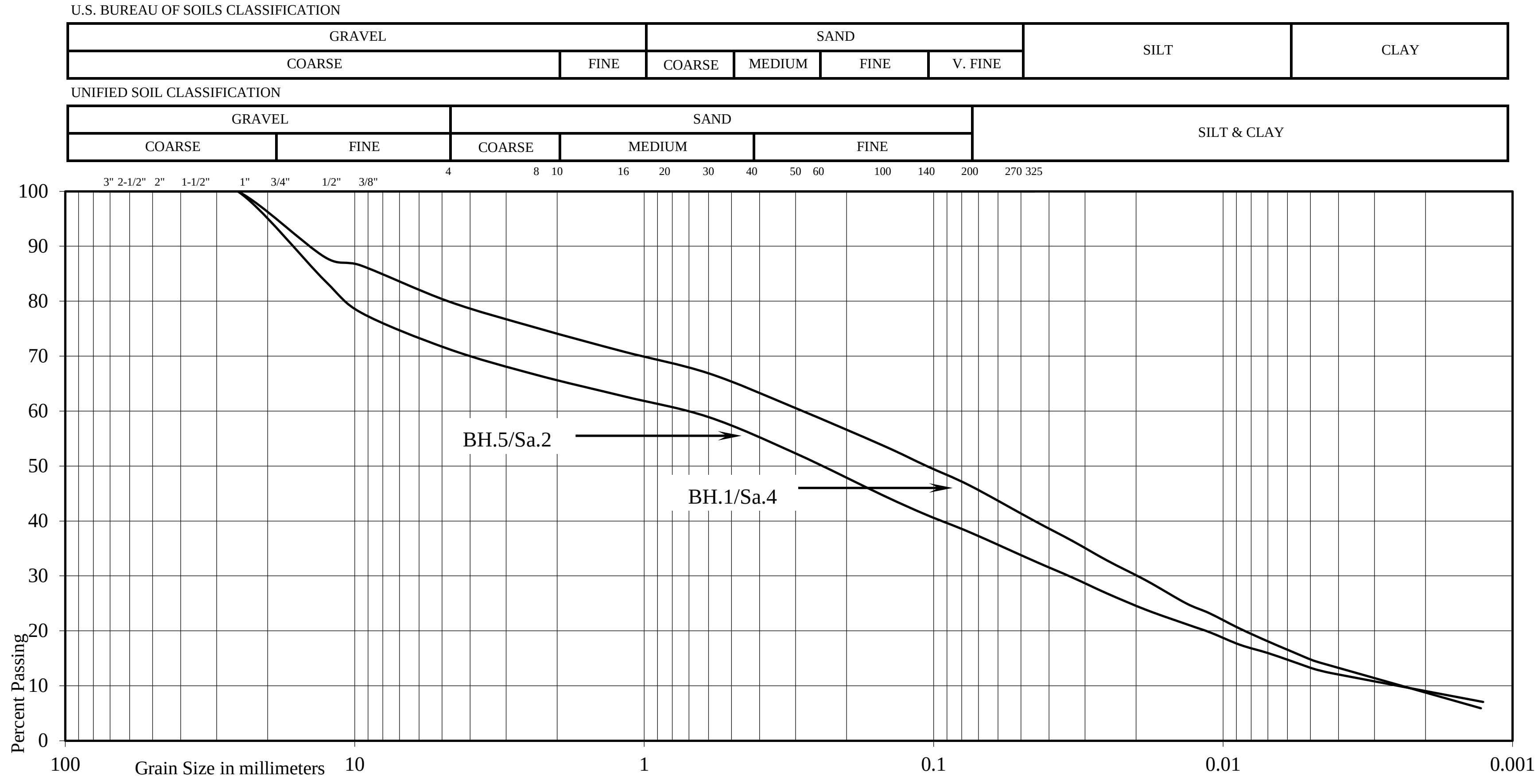
GRAIN SIZE DISTRIBUTION

Reference No: 1110-S118



Project: Proposed Industrial Development

Location: Mountain Road, Town of Collingwood

Borehole No:	1	5
Sample No:	4	2
Depth (m):	2.6	1.0
Elevation (m):	179.7	181.2

BH./Sa.	1/4	5/2
Liquid Limit (%) =	-	-
Plastic Limit (%) =	-	-
Plasticity Index (%) =	-	-
Moisture Content (%) =	-	-
Estimated Permeability (cm./sec.) =	10^{-6}	10^{-6}

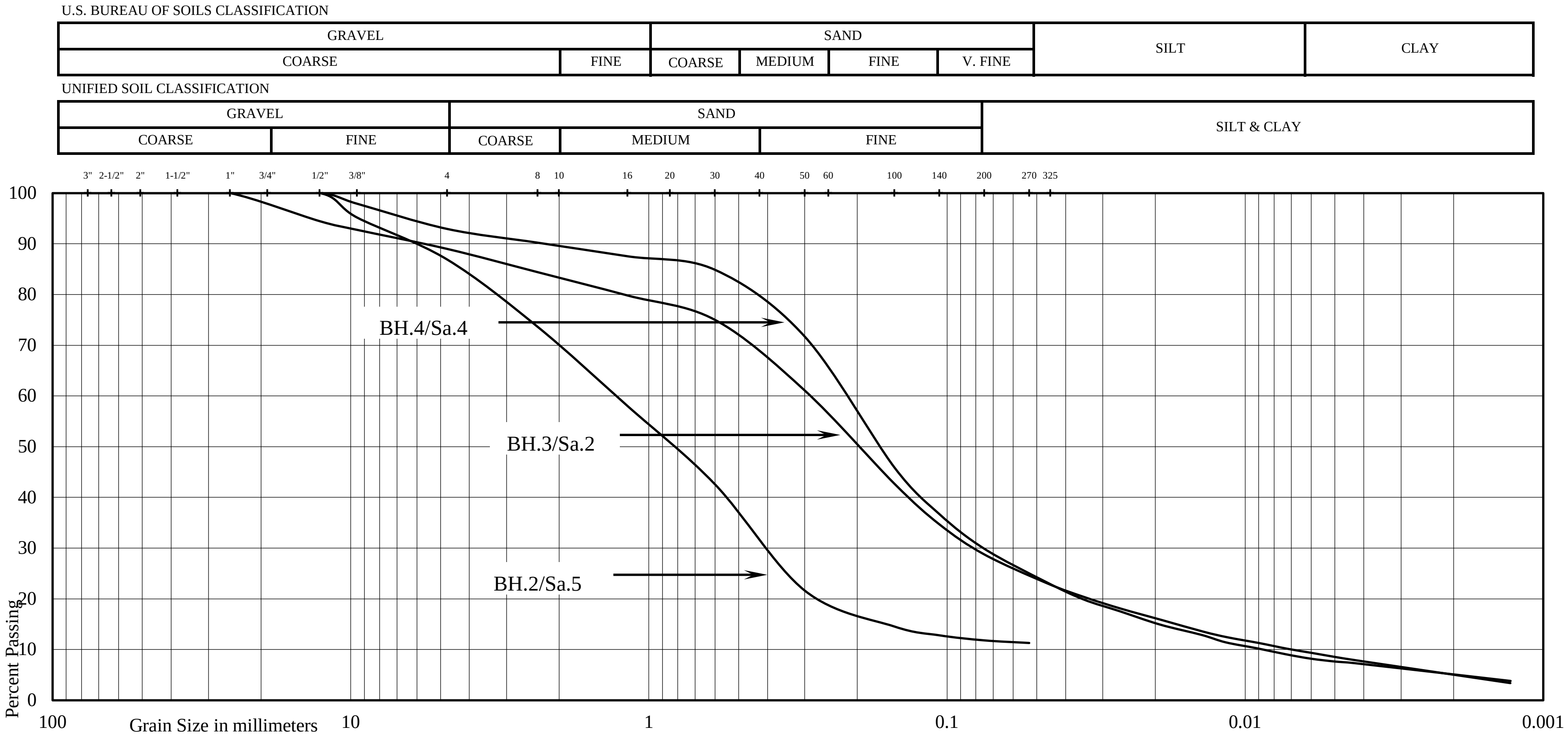
Classification of Sample [& Group Symbol]: SILTY SAND, Till
some clay

Figure: 12

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 1110-S118



Project: Proposed Industrial Development

Location: Mountain Road, Town of Collingwood

Borehole No:	2	3	4
Sample No:	5	2	4
Depth (m):	3.2	1.0	2.5
Elevation (m):	179.1	181.2	179.6

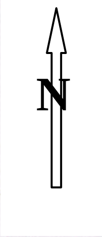
BH./Sa.	2/5	3/2	4/4
Liquid Limit (%) =	-	-	-
Plastic Limit (%) =	-	-	-
Plasticity Index (%) =	-	-	-
Moisture Content (%) =	-	-	-
Estimated Permeability (cm./sec.) =	10^{-3}	10^{-5}	10^{-5}

Classification of Sample [& Group Symbol]:

BH.2/Sa.5 - FINE TO COARSE SAND, some silt

BH.3/Sa.2 & BH.4/Sa.4 - SILTY SAND, Till, traces of clay and gravel

Figure: 13



BOREHOLE LOCATION PLAN

Ref. No.:		1110-S118
Date:		December 2011
Drawing No.:		1
Scale:	Horiz.:	N.T.S.

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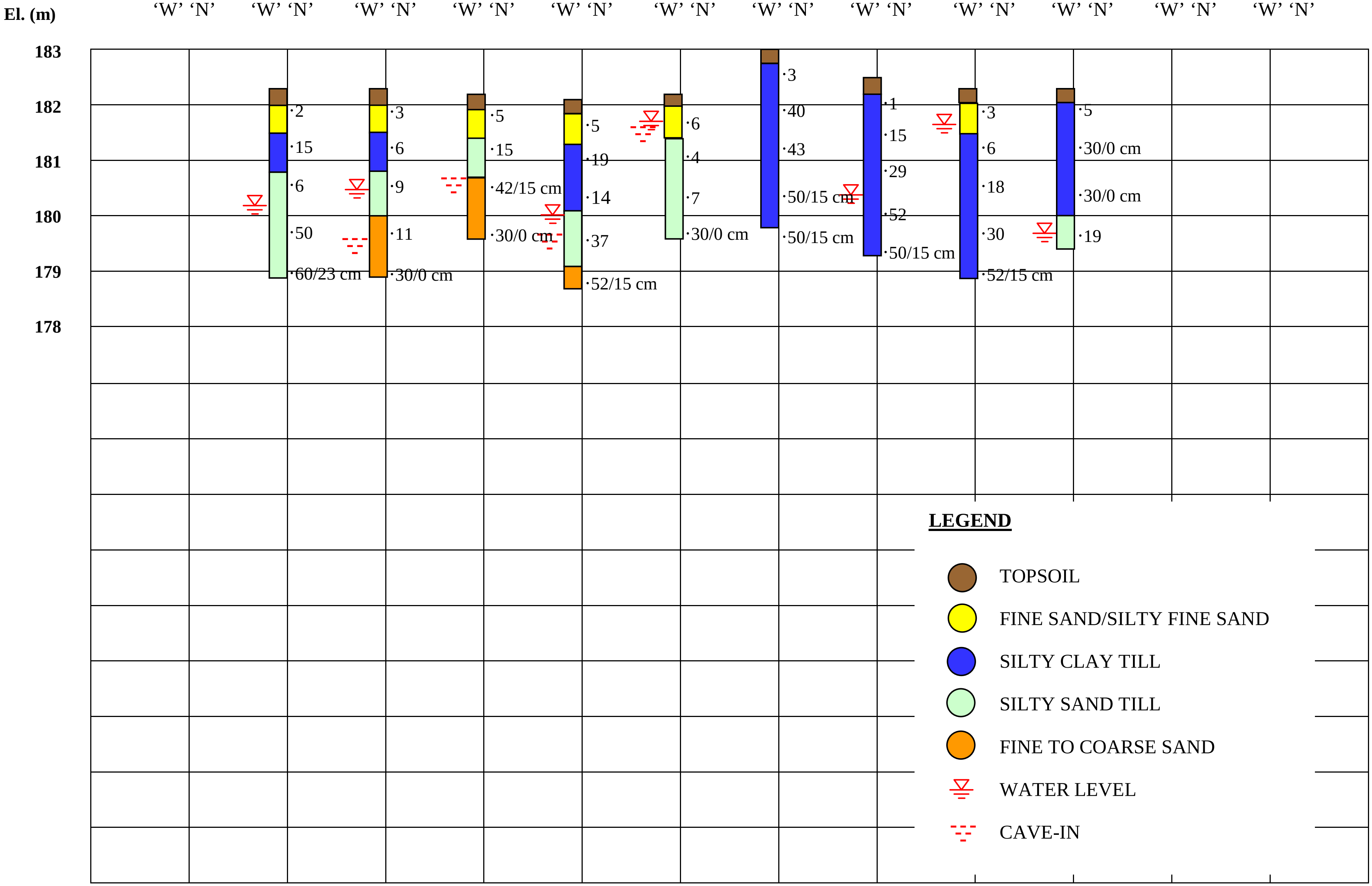
BH. No.		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**		
Topsoil (cm)		**30**	**30**	**28**	**25**	**23**	**25**	**30**	**25**	**25**		
Elevation (m)		**182.3**	**182.3**	**182.2**	**182.1**	**182.2**	**183.0**	**182.5**	**182.3**	**182.3**		



SUBSURFACE PROFILE

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Scale: ***Horiz.:*** *N.T.S.*
Vert.: *1:100*

Ref. No.: *1110-S118*
Drawing No. *2*