

GEOTECHNICAL INVESTIGATION/SLOPE STABILITY

**RESIDENTIAL DEVELOPMENT, AREA B
HILTON HEAD HEIGHTS (CONDOMINIUMS)
408 RIDGE ROAD
MEAFORD, ONTARIO**

CMT Project 19-623.R01

Prepared for:

Meaford Golf Course Inc.

February 5, 2020





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February 5, 2020

19-623.R01

Meaford Golf Course Inc.
408 Ridge Road
Meaford, Ontario
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Attention: Mr. Ivan Alderdice

Dear Sir:

**Re: Geotechnical Investigation/Slope Stability
Residential Development, Area B
Hilton Head Heights (Condominiums)
408 Ridge Road
Meaford, Ontario**

As requested, CMT Engineering Inc. conducted a geotechnical investigation and slope stability assessment at the above referenced site, and we are pleased to present the enclosed report.

We trust that this information meets your present requirements and we thank you for allowing us to undertake this project. Should you have any questions, please do not hesitate to contact our office.

Yours truly,

A handwritten signature in blue ink, appearing to read 'Brandon R Figg'.

Brandon R Figg, C.Tech.
Senior Soil Technician

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1.0 INTRODUCTION

The services of CMT Engineering Inc. (CMT Inc.) were retained by Mr. Ivan Alderdice of Meaford Golf Course Inc., to conduct a geotechnical investigation and slope stability assessment for the proposed Residential Development, Area B, Hilton Head Heights Condominiums, to be located at 408 Ridge Road in Meaford, Ontario. The location of the site is shown on Drawing 1.

It is understood that the project will involve the construction of several new residential condominium buildings with associated roadways, underground utilities and stormwater management facilities.

The purpose of the geotechnical investigation was to assess the existing soil and groundwater conditions encountered in the boreholes and assess the characteristics of the slope around the unnamed tributary to Pete's Creek area. Included in the assessment are the soil classification and groundwater observations, as well as comments and recommendations regarding geotechnical resistance (bearing capacity); serviceability limit states (anticipated settlement); dewatering considerations; site classification for seismic site response; recommendations for site grading, site servicing, excavations and backfilling; recommendations for slab-on-grade construction; pavement design/drainage; soil design properties; slope stability assessment; and a summary of the laboratory results.

2.0 EXISTING SITE CONDITIONS

The site of the proposed new residential development (condominiums) is located on the west side of Ridge Road and currently comprises an existing operating golf course. The site is bounded by Ridge Road to the east, residential development to the north and south, and agricultural property to the west.

3.0 FIELD AND LABORATORY PROCEDURES

The field investigation was conducted on January 9, 2020 and comprised the advancement of two (2) boreholes (referenced as Boreholes 1 and 2), utilizing a Geoprobe 7822DT drillrig operated by employees of CMT Drilling Inc. The boreholes were advanced at the top of the existing slope near the location of the proposed residential development to support the slope assessment. The boreholes were advanced to approximate depths of 8.23 m (27.0 ft) below the ground surface.

Standard penetration testing and sampling was carried out in all boreholes using 38 mm inside diameter split spoon sampling equipment and an automatic hammer, in accordance with ASTM D 1586 "Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils". SPT soil sampling was generally conducted at 0.76 m (2.5 ft) intervals to 3.05 m (10.0 ft), and every 1.52 m (5.0 ft) thereafter, to borehole termination. Macro core (MC5) direct push sampling was conducted between the SPT soil samples conducted below 3.05 m (10.0 ft) depth.

Technical staff from CMT Inc. observed the drilling operation and collected and logged the recovered soil samples. A small portion of each sample was placed in a sealed, marked jar for moisture content determinations.

Representative samples from the boreholes at the following depths were submitted to our laboratory for grain size analyses:

- Borehole 1 - depth 0.76 m to 1.37 m (2.5 ft to 4.5 ft)
- Borehole 1 - depth 2.29 m to 2.90 m (7.5 ft to 9.5 ft)
- Borehole 2 - depth 6.70 m to 7.62 m (22.0 ft to 24.0 ft)

The borehole logs are provided in Appendix A and the resulting grain size analyses can be found in Appendix B.

The ground surface elevations of the boreholes were surveyed by CMT Inc. using laser survey equipment following the completion of drilling. An existing spike in the ground adjacent to Borehole 1 was utilized as a temporary benchmark, with a reported elevation of 198.929 m. The ground surface elevations at the boreholes ranged from approximately 199.45 m to 199.17 m. The locations of the boreholes are shown on Drawing 2.

4.0 SUBSOIL CONDITIONS

The following paragraphs have been simplified in terms of major soil strata for the purposes of geotechnical design. The soil boundaries indicated have been inferred from non-continuous samples and observations of sampling and drilling resistance, and typically represent transitions from one soil type to another rather than exact planes of geological change. Further, the subsurface conditions are anticipated to vary between and beyond the borehole locations.

The soils encountered in the boreholes are described briefly below and a more detailed stratigraphic description is provided on the borehole logs in Appendix A.

4.1. Topsoil

Dark brown to black, silty, organic topsoil was encountered at the surface of the boreholes. The thickness of the topsoil was observed to range from about 50 mm to 250 mm (average 150 mm) at the borehole locations. The thickness of the topsoil is anticipated to vary outside of the borehole locations. Materials noted as topsoil in this report were classified based on visual and textural evidence. Testing of organic content or for other nutrients was not carried out.

4.2. Sandy Silt Fill

Dark brown to brown sandy silt fill with some clay and trace gravel was encountered underlying the topsoil in Boreholes 1 and 2. The sandy silt fill was considered to be very loose to compact, with SPT N-values ranging from 2 to 24 blows per 0.30 m (average 13 blows per 0.30 m). The sandy silt fill was considered to be moist, with moisture contents ranging from approximately 13.1% to 21.3% (average 17.2%).

4.3. Silt Till

Brown silt till with some clay, trace sand and gravel was encountered underlying the sandy silt fill in both boreholes. The silt till was considered to be stiff to hard, with SPT N-values ranging from 11 to greater than 100 blows per 0.30 m (average 56 blows per 0.30 m). The silt till was considered to be moist, with moisture contents ranging from approximately 10.1% to 22.1% (average 16.1%).

4.4. Silty Sand Till

Grey (becoming a reddish colour in Borehole 2) silty sand till with trace clay was encountered underlying the silt till in both boreholes. The silty sand till was considered to be very dense, with an SPT N-value of greater than 100 blows per 0.30 m. The silty sand till was considered to be moist, with moisture contents ranging from approximately 10.1% to 10.6% (average 10.4%).

4.5. Groundwater

No accumulated groundwater was observed in the boreholes; however, occasional wet seams within the silt till soils were observed in the boreholes conducted for this investigation. The following table summarizes the borehole number, ground surface elevation, elevation of water accumulated in the borehole, cave elevation, approximate elevation of wet to saturated soils, and the bottom of borehole elevation for each borehole:

Borehole No.	Ground Surface Elevation (m)	Approximate Elevation of Accumulated Water in Open Borehole (m)	Approximate Borehole Cave Elevation (m)	Approximate Elevation of Wet to Saturated Soils (m)	Bottom of Borehole Elevation (m)
BH1	199.17	--	192.77	193.98 to 192.46	190.94
BH2	199.45	--	194.42	192.74	191.22

It should be noted that groundwater conditions (particularly perched water) are generally dependent on the amount of precipitation, control of surface water, as well as the time of year, and can fluctuate significantly in elevation and volume.

The relatively stiff and fine-grained silt till/silty sand till have the potential to create perched water conditions in the overlying soils. It should be noted that groundwater conditions (particularly perched water) are generally dependent on the amount of precipitation, control of surface water, as well as the time of year, and can fluctuate significantly in elevation and volume.

Recommendations with respect to dewatering conditions are provided in Section 5.8 of this report.

5.0 DISCUSSION AND RECOMMENDATIONS

This section of the report provides CMT Inc.'s interpretation of the factual geotechnical data obtained during the investigation and is intended for the guidance of the owner and design engineer. Where comments are made on construction, they are provided only to highlight those aspects which could affect the design of the project. Contractors bidding on or undertaking the work should make their own independent interpretation of the factual subsurface information provided as it affects their proposed construction means and methods, equipment selection, scheduling, pricing, and the like.

It is understood that the project will involve the construction of a new residential development (condominiums) with associated roadways, underground utilities and stormwater management facilities.

Utilizing the information gathered during the geotechnical investigation and assuming that the borehole information is representative of the subsoil conditions throughout the site, the following comments and recommendations are provided.

5.1. Serviceability and Ultimate Limit Pressure

Based on the information obtained from the boreholes, the following table provides a summary of the estimated geotechnical reaction at the Serviceability Limit State (SLS) and the factored geotechnical resistance at the Ultimate Limit State (ULS) at the various elevations, including soil type:

BH No.	Ground Surface Elevation (m)	SLS kPa (psf)	ULS kPa (psf)	Estimated Highest Founding Elevations (m)	Depth Below Existing Grade to Founding Elevation (m)	Soil Type
1	199.17	150 (3,000) 250 (5,000)	225 (4,500) 350 (7,000)	196.58 to 193.07 193.07 to 190.94 (termination)	2.59 6.10	Silt Till/Silty Sand Till
2	199.45	150 (3,000) 250 (5,000)	225 (4,500) 350 (7,000)	197.63 to 193.35 193.25 to 191.22 (termination)	1.82 6.10	Silt Till/Silty Sand Till

Based on the bearing capacities and elevations provided in the table above, suitable founding elevations for conventional foundations designed with an estimated bearing capacity of 150 kPa (3,000 psf) at SLS and 225 kPa (4,500 psf) at ULS range throughout the site, however, suitable founding soils are typically found below the existing topsoil, fill soils, as well as loose/soft upper native soils.

Should footings be designed to be constructed at elevations higher than the elevations indicated in the table above, then structural fill will be required in order to achieve the design grades for the proposed foundations. The serviceability limit pressure for granular structural fill placed and compacted in accordance with Section 5.4.4 of this report and constructed on approved competent native soil is estimated to be at least 150 kPa (3,000 psf) at SLS and 225 kPa (4,500 psf) at ULS. Alternatively, lean mix concrete fill could be utilized for this application.

Alternatively, footings could be stepped down to bear on approved undisturbed founding soils. Due to the presence of fill soils, it is imperative that the founding soils be assessed at the time of construction by qualified geotechnical personnel in order to confirm their founding suitability.

Footings founded on soil may be placed at a higher elevation relative to another footing provided that the slope between the outside face of the footings is separated by a minimum slope of 10 horizontal to 7 vertical (10H:7V) with an imaginary line projected from the underside of the footings.

It is recommended that structural foundation drawings be cross-referenced with site servicing drawings to ensure that service pipes do not conflict with building foundations (including the zone of influence down and away from the footings). With respect to the Serviceability Limit State (SLS), the total and differential footing settlements are not expected to exceed the generally acceptable limits of 25 mm (1") and 19 mm (3/4") respectively.

All exterior footings must be provided with a minimum of 1.4 m of soil cover or equivalent thermal insulation in order to provide protection against frost action.

It should be noted that occasional wet seams were observed within the native soils that exist at or below founding elevations and these soils may be in a wet state and may be too wet to provide suitable bearing for foundations without drainage or construction of a mud mat or granular drainage layer. It is imperative that the subgrade soils be inspected and approved by competent geotechnical personnel to ensure that the founding soils are suitable for bearing. Dewatering during construction may be required (see Section 5.8 of this report), along with the potential construction of a mud mat or granular drainage layer.

CMT Inc. would be pleased to review design drawings when they become available and provide further recommendations with respect to bearing and foundation elevations.

5.2. Seismic Site Classification

The site classification for seismic response in Table 4.1.8.4 of the 2012 Ontario Building Code relates to the average properties of the upper 30 m of strata. The information obtained in the geotechnical field investigation was gathered from the upper 8.23 m of strata. Based on the information gathered in the geotechnical field investigation, the site classification for seismic site response would be considered Site Class D (stiff soils) for structures founded on the native soils or structural fill at the recommended founding elevations provided in Section 5.1 of this report. The structural engineer responsible for the design of the structure should review the earthquake loads and effects.

5.3. Soil Design Parameters

The following table provides estimated soil design parameters for imported granular fill, as well as the existing native soils encountered on-site.

The estimated soil design parameters can be utilized for the design of perimeter shoring, foundations and retaining walls, as required.

Soil Type	Soil Density (kg/m ³)	Friction Angle (Degree)	Coefficient of Active Pressure (K _a)	Coefficient of Passive Pressure (K _p)	Coefficient of At-Rest Pressure (K ₀)	Coefficient of Friction (μ)	Cohesion (Undrained) (kPa)
Imported Granular 'A'/ Granular 'B' (OPSS 1010)	2,100	34	0.28	3.54	0.44	0.45	0
Sandy Silt Fill	1,750	28	0.36	2.77	0.53	0.35	0
Silt Till	1,850	30	0.33	3.00	0.50	0.38	10

Soil Type	Soil Density (kg/m ³)	Friction Angle (Degree)	Coefficient of Active Pressure	Coefficient of Passive Pressure	Coefficient of At-Rest Pressure	Coefficient of Friction (u)	Cohesion (Undrained) (kPa)
Silty Sand Till	1,800	32	0.31	3.25	0.47	0.41	0

5.4. Site Preparation

The site preparation for the proposed new residential development (condominiums) is anticipated to include the removal of any potential remaining structures associated with the existing golf course, removal of topsoil and vegetation, the subexcavation of all unsuitable fill and any native soils deemed not capable of supporting the design bearing capacity, removal or relocation of any existing services, followed by the placement of structural fill (as required) and site grading to achieve proposed grades.

5.4.1. Topsoil Stripping/Vegetation Removal

All existing topsoil, vegetation and trees (including tree root structures as well as any loose soils that are typically associated with root structures) must be removed from within the proposed building and driveway envelopes to expose approved competent subgrade soils. The topsoil may be used in landscaped areas where some settlement can be tolerated; otherwise, it should be properly disposed of off-site.

The volume of topsoil removed during the stripping process can be influenced by the equipment utilized for the stripping process as well as the moisture conditions at the time of stripping. If an excavator with a smooth bucket is utilized for stripping, there would generally be less potential for topsoil to become intermixed with the underlying, generally loose/soft subsoil and therefore less concern of over-excavation to remove all topsoil. If the topsoil is stripped with wheeled equipment or bulldozers, then there is an increased potential for the topsoil and subsoil to become intermixed, subsequently requiring additional excavation to remove all topsoil. This is further influenced by rutting which can occur during wet conditions.

5.4.2. Fill/Unsuitable Soil Removal

All existing fill, as well as any native soil that has inadequate bearing capacity or has been disturbed by construction processes and is deemed unsuitable to support foundations or slab-on-grades, must be subexcavated from within the proposed building envelopes to expose approved competent subgrade soils. It would also be sound construction practice to subexcavate all existing unsuitable fill from the paved roadway and driveway areas; however, this may not be cost-effective. At a

minimum, thorough inspection will be required at the time of construction to assess the existing fill to ensure there is no buried topsoil or other deleterious materials within the prepared subgrade. Remedial action will also be required to further consolidate the existing fill if it is decided to leave it in place. If the existing fill is left in place, provisions for the alterations to the design of the pavement structure should be included in the tender documents. Review of the subgrade and potential changes to the design of the pavement structure, as required, will be addressed at the time of construction.

Prior to reusing excavated material on-site as potential bulk fill in the driveways, thorough field inspection and approval by qualified geotechnical personnel would be required to ensure that existing fill materials do not comprise organics, topsoil or other deleterious materials.

5.4.3. Removal/Relocation of Existing Buried Piping and Structures

Any existing tile drains (field tiles or municipal tile drains) and/or irrigation pipes that may be located within the proposed building and driveway envelopes must be completely removed to a minimum distance of 15.0 m (50.0 ft) outside of the construction envelopes. All drains that are terminated must be completely sealed with concrete or grout at termination points to prevent the migration of soils into pipe voids which may result in potential settlement. Ideally, depending on flow direction, any existing tile drains (if present) should be redirected and reconnected outside of the building envelopes in order to maintain flow and prevent subsurface accumulation of water. It may be prudent (if feasible) to incorporate existing field tiles into the storm sewer system or a separate collection system, to assist in systematically draining the subsurface soils in the residential development. All existing trench backfill material associated with the drains must be subexcavated and the subsequent excavation must be backfilled with approved soils placed in accordance with Section 5.4.4 of this report. The location of existing field tiles is commonly identified by lines of buried topsoil within the subgrade soils and/or water boiling out of the ground following excavation. The field tiles are historically installed at 15.0 m (50.0 ft) intervals, however, this can vary from site to site. Irrigation pipes are usually installed close to existing golf greens.

Any existing foundation walls, footings, slab-on-grade and other construction materials as well as all associated backfill material must be removed from areas of demolished structures. It is recommended to advance test pits at the start of site grading operations in the areas of any previous structures to determine the extent and quality of the backfill material. Any excavations conducted to remove existing structures must be inspected and backfilled according to the procedures outlined in Section 5.4.4 of this report. It is recommended that imported sand and gravel (OPSS 1010 Granular 'B' or an approved alternative) be placed as structural fill to backfill any demolition areas.

5.4.4. Site Grading

Following removal of any existing below-grade structures, the topsoil, vegetation, as well as the subexcavation of the relatively loose fill and any native soils deemed unsuitable of supporting the design bearing capacity, the exposed subgrade must be proof-rolled, and any soft or unstable areas must be subexcavated and replaced with approved fill materials.

Any fill materials required to achieve the design grades should be placed according to the following procedures:

- Should the native subgrade soils at the design founding elevation in the proposed building envelope(s) comprise of wet or saturated soils, then a granular drainage layer constructed in accordance with Section 9.14.4 of the current Ontario Building Code (OBC) may be required. Alternatively, a lean mix concrete mud mat may be placed overlying the subgrade soils to provide a stable base;
- Prior to placement of any structural fill or bulk fill, the subgrade for the proposed building envelope and driveways must be prepared large enough to accommodate a 1:1 slope commencing a distance of 1.0 m beyond the outside edge of the proposed foundation and pavement edge (where feasible) to the approved competent founding soils;
- Soils approved for use as structural fill must be placed in loose lifts not exceeding 0.3 m (12") in depth for granular soils (recommended fill material) and 0.2 m (8") in depth for silts and clays (not recommended for this application), or the capacity of the compactor (whichever is less);
- Imported granular fill materials (OPSS 1010 Type III Granular 'B' recommended for this application) can be compacted utilizing adequate heavy vibratory smooth drum compaction equipment;
- Fine-grained silt and clay soils (not recommended) must be compacted utilizing adequate heavy padfoot vibratory compaction equipment;
- Approved fill materials must be at suitable moisture contents to achieve the specified compaction. Any wet to saturated soils encountered in the boreholes would generally be considered difficult for use as structural fill as they would require extensive air-drying in order to achieve the specified density. Soil moisture will also be dependent on weather conditions at the time of construction. Granular soils may require the addition of water in order to achieve the specified compaction;

- Approved structural fill materials that will support structures (including foundations, interior slab-on-grades, sidewalks and large expansive exterior slabs) must be compacted to 100% standard Proctor maximum dry density (SPMDD). The native silt till/sandy silt till are not recommended for use as structural fill as they can be subject to excess void space and potential settlement if not properly placed and compacted;
- Approved bulk fill (foundation wall backfill, bulk fill under slab-on-grades that will not support footings or heavy point loading, bulk fill for driveways) must be compacted to a minimum 98% SPMDD. It would be expected that the native silt till/sandy silt till would be suitable for use as bulk fill; however, depending on the time of year and weather conditions when construction takes place, soils excavated at depth may require air-drying in order to achieve the specified density;
- Granular 'B' subbase and Granular 'A' base materials for the paved parking areas must be compacted to 100% SPMDD.

Based on the subsurface conditions observed in the boreholes, wet soils may be encountered, depending on the depth of excavation. As such, for soils excavated from within the zone of saturation, significant air-drying along with working of the soils may be required in order to achieve the specified compaction. Utilizing the existing soils during site grading may be more achievable if work is completed during the generally drier summer months. It should be noted, however, that due to the nature of some of the soils, during hot dry weather, the addition of water might be required in order to achieve the specified compaction. Reuse of excavated soils on-site will be subject to approval from qualified geotechnical personnel.

It should also be noted that the native soils encountered in the boreholes were observed to become very dense/hard with depth (SPT N-values in excess of 50 blows per 0.30 m) and may prove difficult to excavate with conventional excavating equipment. It is imperative that should the very dense/hard soils be utilized as fill, that the material must be broken down (pulverized) to minimize void space and reduce the potential for settlement. Problems associated with compacting very dense/hard soils include the potential for long-term settlement due to excessive void space caused by the generally blocky structure of the excavated soils. As such, the very dense, blocky material must **not** be used as structural fill. The contractor must have equipment on-site that can effectively break down (pulverize) the very dense excavated soil into workable sizes (as required). Backfilling utilizing this material must be performed in thin lifts with considerable compactive effort applied, thereby reducing the void space and minimizing long-term settlement. This process could be difficult and time-consuming.

5.5. Foundation Subgrade Preparation

The native soils encountered in the boreholes are sensitive to change in moisture content and can become loose/soft if the soils are subjected to additional water or precipitation, as well as severe drying conditions. The native subgrade soils could also be easily disturbed if traveled on during construction. Once they become disturbed, they are no longer considered adequate for the support of shallow foundations.

To ensure and protect the integrity of the founding soils during construction operations, the following is recommended:

- During construction, the subgrade should be sloped to a sump (as required) located outside the building footprints (if feasible) in the excavation to promote surface drainage of rainwater or seepage and the collected water should be pumped out of the excavations. It is critical that all water be controlled (not allowed to pond) and that the subgrade and foundation preparations commence in dry conditions;
- Should the native subgrade soils at the design founding elevation in the proposed building envelopes comprise saturated soils, then a granular drainage layer, constructed in accordance with Section 9.14.4 of the current Ontario Building Code (OBC) may be required;
- Construction equipment travel and foot traffic on the founding soils should be minimized;
- If construction is to be undertaken during subzero weather conditions, the founding native soils and any potential fill materials must be maintained above freezing;
- Prior to placing concrete for the foundations, the founding soils must be cleaned of all disturbed, soft or caved materials;
- The foundation formwork and concrete should be installed as soon as practical following the excavation, inspection and approval of the founding soils. The longer that the excavated soils remain open to weather conditions and groundwater seepage, the greater the potential for construction problems to occur;
- If it is expected that the founding soils will be left open to exposure for an extended period of time, it is recommended that a 75 mm concrete mud slab be poured in order to protect the structural integrity of the founding soils.

5.6. Slab-on-Grade/Modulus of Subgrade Reaction

Prior to the placement of the granular base for any slab-on-grade construction, the subgrade soils should be proof-rolled. Any soft or weak zones, as well as the unsuitable fill in the subgrade, should be subexcavated and backfilled with approved fill materials (see Sections 5.4.4 and 5.10 of this report).

The following table provides the estimated modulus of subgrade reaction (k) for imported granular fill, as well as the native soils encountered on-site:

Soil Type	Estimated Modulus of Subgrade Reaction (k)
Imported Sand and Gravel (OPSS 1010)	81,000 kN/m ³ (300 lb/in ³)
Silt Till	48,000 kN/m ³ (175 lb/in ³)
Silty Sand Till	61,000 kN/m ³ (225 lb/in ³)

In dry conditions, floor slabs can be founded on a minimum thickness of 150 mm (6") of Granular 'A' (OPSS 1010) and compacted to 100% SPMDD. Alternatively, (particularly in wet conditions), 150 mm (6") of 19 mm clear crushed stone (OPSS 1004) should be used instead of Granular 'A'. Utilizing clear crushed stone for the slab-on-grade base can assist in providing a moisture barrier by reducing the potential for capillary rise of moisture from the subgrade soils. Compactive effort is required to consolidate the clear stone. The 19 mm clear crushed stone should meet the physical property and gradation requirements of OPSS 1004.

It is recommended that areas of extensive exterior slab-on-grade (sidewalks and accessibility ramps) be constructed with a Granular 'B' subbase (450 mm) and a Granular 'A' base (150 mm), as well as incorporating subdrains, to promote rapid drainage and reduce the effects of frost heaving. This is particularly critical at barrier-free access points. Alternatively, structural frost slabs could be designed and constructed, or sufficient thermal insulation could be provided, at all door entrances and areas of barrier-free access.

5.7. Excavations

All excavations must be carried out in accordance with Ontario Regulation 213/91 (Reg 213/91) of the Occupational Health and Safety Act and Regulations for Construction Projects.

Type 2 Soils - In general, the native silt till/silty sand till soils encountered in the boreholes, in a drained state (not saturated), would be classified as Type 2 soils under Reg 213/91. Type 2 soils must be sloped from within 1.2 m of the bottom of the excavation at a minimum gradient of 1 horizontal to 1 vertical. Soils underlain by Type 3 or Type 4 soils that are exposed in the excavation must be treated accordingly as Type 3

or Type 4 soils (see below). Soils in a saturated condition (if encountered) must be treated as Type 4 soils, addressed below.

Type 3 Soils - In general, any existing fill materials (backfill of existing foundations and services) in a drained state (not saturated), would be classified as Type 3 soils under Reg 213/91. The Type 3 soils must be sloped from the bottom of the excavation at a minimum gradient of 1 horizontal to 1 vertical. Soils underlain by Type 4 soils that are exposed in the excavation must be treated accordingly as Type 4 soils (see below). All saturated soils encountered must be treated as Type 4 soils, as described below.

Type 4 Soils - In general, any wet to saturated soils would be classified as Type 4 soils under Reg 213/91. Type 4 soils must be sloped from the bottom of the excavation at a minimum gradient of 3 horizontal to 1 vertical.

If it is not practical to excavate according to the above requirements, then a trench support system (designed in accordance with the Ontario Health and Safety Act Regulations) may be utilized. When using a temporary trench support system consisting of trench boxes to reduce the lateral extent of the excavations, it should be noted that the support system is intended primarily to protect workers as opposed to controlling lateral soil movement. Any voids between the excavation walls and the support system should be immediately filled to reduce the potential for loss of ground and to provide support to existing adjacent utilities and roadways, and it is recommended that the excavation be carried out in short sections, with the support system installed immediately upon excavation completion.

5.8. Construction Dewatering Considerations

Although no accumulated groundwater was encountered in the boreholes upon completion of drilling, some wet seams within the silt till soils were observed within the boreholes. It should also be noted that the relatively fine-grained silt till/silty sand till soil typically encountered in the lower zone of the boreholes have the potential to create perched water conditions. As such, it is critical that provisions for site dewatering be part of the site development and construction process.

Seepage control requirements and groundwater conditions during construction are generally dependent on the amount of precipitation, control of surface water, the time of year, the area of work on the site, and the depth of the excavations, and can fluctuate significantly in elevation and volume. As required, seepage should generally be adequately controlled using conventional construction dewatering techniques such as pumping from properly filtered sump pits. However, if heavy seepage occurs, it may be necessary to increase the number of pumps during construction. Some seepage and sloughing should be expected from the wet seams within the till.

Dewatering should be performed in accordance with OPSS 517 and the control of water must be in accordance with OPSS 518. It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. Collected water should discharge a sufficient distance away from the excavation to prevent re-entry. Sediment control measures must be installed at the discharge point of the dewatering system to avoid any potential adverse impacts on the environment.

5.9. Service Pipe Bedding

The native soils encountered in the geotechnical investigation are generally considered suitable for indirect support of the site service pipes. Should instability due to saturated soil conditions be encountered, it may be necessary to increase the thickness of the granular base and utilize 19 mm clear stone to create an adequate supporting base for the service pipes and/or manholes. Pipe embedment, cover and backfill for both flexible and rigid pipes should be in accordance with all current and applicable OPSD, OPSS and OBC standards and guidelines and as follows:

Flexible Pipes – The pipe bedding should be shaped to receive the bottom of the pipe. If necessary, pipe culvert frost treatment should be undertaken in accordance with OPSD-803.031. The trench excavations should be symmetrical with respect to the centre-line of the pipe. The granular material placed under the haunches of the pipe must be compacted to 100% SPMDD prior to the continued placement and compaction of the embedment material. The homogeneous granular material used for embedment should be placed and compacted uniformly around the pipe. Should wet conditions be encountered at the base of the trench, then the pipe bedding should consist of 19 mm clear stone (meeting OPS Specifications) wrapped completely in a geotextile fabric such as Terrafix 270 or equivalent. The general contractor is responsible to protect service piping from damage by heavy equipment.

Rigid Pipes - In general, the pipe installation recommendations for rigid pipes are the same as those for flexible pipes, except that the minimum bedding depth below a rigid pipe should be $0.15D$ (where D is the pipe diameter). In no case should this dimension be less than 150 mm or greater than 300 mm.

5.10. Perimeter Building Drainage, Foundation Wall Backfill and Trench Backfill

In order to assist in maintaining a dry building with respect to surface water seepage, it is recommended that exterior grades around the building be sloped down and away at a 2% gradient or more, for a distance of at least 1.5 m to 2.0 m (depending on property line setbacks). Any surface discharge rainwater leaders must be constructed with solid piping that discharges with positive drainage at least 1.5 m away from the building foundations and/or beyond sidewalks to a drainage swale or appropriate storm drainage system.

The founding elevations for the proposed structures were not available at the time of preparation of this report. CMT Inc. can provide further recommendations for building drainage once the design drawings are completed and the founding elevations have been confirmed.

It should be noted that based on the observations in the boreholes, there is potential for perched water conditions. The construction of foundations, slabs-on-grade, and deep structures such as sump pits within or below zones of saturation will require design of site-specific waterproofing and dewatering systems constructed in accordance with the 2012 OBC. It is recommended that a good quality sump pump be utilized, and that the system be equipped with a battery backup in the event of power failure, (keeping in mind that a battery backup system does not typically have a long run time).

If the new developments will have basements, an exterior perimeter drainage system comprising perforated drainage pipe with a factory installed filter sock, bedded in 19 mm clear crushed stone (OPSS 1004) and wrapped in a geotextile filter fabric such as Terrafix 270R (or equivalent) must be installed at an elevation that is below the proposed slab-on-grade elevation and provided with positive drainage into a sump pit. The portion of the piping that connects the exterior drainage tile system into the sump pit must comprise solid piping to prevent exterior water from being introduced into the interior subslab stone. It may be prudent to install perforated drainage pipe on the interior as well to provide an outlet for any water that may collect in the subslab stone (particularly during the construction phase of the project). It is also recommended that a capped cleanout port(s) be extended up to the ground surface elevation to provide future access (if required). The rainwater leaders must not be connected to the perimeter weeping tile system. Foundation wall and slab-on-grade dampproofing and/or waterproofing must conform to current OBC regulations. If required, it would be recommended that a waterproofing supplier/specialist be consulted to recommend an appropriate product and installation requirements that would be suited to this site.

Depending on the groundwater conditions at the design founding elevations, it may be necessary to install a granular drainage layer to provide a suitable base for the foundations. This will depend on the bearing capacity required for the founding strata. If required, the granular drainage layer must conform to the requirements listed in Section 9.14.4 of the OBC 2012.

In order to reduce the effects of surficial frost heave in areas that will be hard surfaced, it is recommended that the exterior foundation backfill consist of free-draining granular material such as approved on-site sand or sand and gravel or imported Granular 'B' Type I or Type III (OPSS 1010), with a maximum aggregate size not exceeding 100 mm, and that it extend a minimum lateral distance of 600 mm out from the foundation walls and/or beyond perimeter sidewalks and entranceway slabs. It is critical that particles greater than 100 mm in diameter are not in contact with the foundation wall to prevent point loading and overstressing. The backfill material used against the foundation walls must be placed so that the allowable lateral capacities of the foundation walls are not exceeded. Where only one side of a foundation wall will be backfilled, and the height of the wall is such

that lateral support is required, or where the concrete strength has not been achieved, the wall must be braced or laterally supported prior to backfilling. In situations where both sides of the wall are backfilled, the backfill should be placed in equal lifts, not exceeding 200 mm differential on each side during backfill operations and the backfill should be compacted to a minimum of 100% SPMDD.

The native silt till/silty sand till soils, as well as approved fill materials (non-organic), are generally considered suitable for reuse as trench backfill and bulk fill in the driveways; however, any wet soils encountered may require air-drying in order to achieve the specified compaction. Air-drying cannot typically be achieved during winter construction; therefore, depending on the time of year that construction takes place, it may be more feasible to utilize an imported granular fill for this project.

Backfilling operations should be carried out with the following minimum requirements:

- Adequate heavy smooth drum or padfoot vibratory compaction equipment should be used for the compaction and to break down any large blocky pieces of soil;
- Loose lift thicknesses should not exceed 0.3 m (12") for granular soils or 0.2 m (8") for silt soils or the capacity of the compactor (whichever is less);
- The soils must be at suitable moisture contents to achieve compaction to a minimum 95% SPMDD in non-structural bulk fill areas. Service trenches excavated within the zone of influence of footings for structures must be compacted to a minimum of 100% SPMDD;
- It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure that compaction requirements are achieved;
- Service trench backfill materials may consist of approved excavated soils with no particles greater than 100 mm and no topsoil or other deleterious materials;
- If construction operations are undertaken in the winter, strict consideration should be given to the condition of the backfill material to make certain that frozen material is not used.

5.11. Pavement Design/Drainage

All existing topsoil, vegetation (including tree root structures as well as any loose soils that are typically associated with root structures), fill, and any soils containing organics or other deleterious materials, must be stripped/subexcavated from within the driveways and any other paved areas. It is recommended to either subexcavate any existing loose subgrade materials or provide further consolidation with vibratory compaction equipment

in order to prepare a proper, stable subgrade. Prior to placement of the new granular base, the subgrade must be proof-rolled, and any soft or unstable areas should be subexcavated and replaced with suitable drier materials. The subgrade should be graded smooth (free of depressions) and properly crowned to ensure positive drainage, with a minimum grade of 3% toward the drainage outlet or curb line. When service pipes are installed, pipe bedding and backfilling should be undertaken as indicated in Sections 5.9 and 5.10 of this report.

Rapid drainage of the pavement structure is critical to ensure long-term performance and to help minimize frost heave. The requirement for subdrains will be dependent on the composition of the prepared roadway subgrade soils. Should the subgrade soils be comprised of fine-grained, frost-susceptible soils, then it is recommended to install subdrains, provided gravity drainage to a suitable outlet can be provided. It is recommended to install minimum 100 mm diameter perforated subdrains to collect and redirect water beneath the pavement surface. Subdrains should be designed and installed in accordance with OPSS 405 and OPSD 216.021. If Granular 'A' bedding (OPSS 1010) is utilized, the subdrains should be equipped with a factory installed filter sock. If 19 mm clear stone (OPSS 1004) is utilized as bedding for the subdrain, then the bedding must be wrapped completely with geotextile filter fabric such as Terrafix 270R (or equivalent) and a factory installed filter sock is not required. Installation of rigid subdrains allows for better grade control and less potential for damage during installation; however, it would be expected that there would be higher cost implications associated with the installation of rigid subdrains over flexible subdrains. Positive drainage through grade control of subdrains is critical, as improperly installed subdrains can turn drainage systems into reservoirs, which can fuel frost action. The subdrains will hasten the removal of water, thereby reducing the risk and effects of frost heaving and load transfer in saturated conditions. It is suggested that, at a minimum, subdrains be installed through all low areas of the parking lot and ideally along the edge of the pavement as well to prevent water from entering the subbase. The subdrains should be installed in a 0.3 m (1.0 ft) by 0.3 m (1.0 ft) trench in the subgrade and bedded approximately 50 mm (2") above the bottom of the trench. The subgrade must be prepared with positive drainage to the subdrains and the subdrains must be installed with positive drainage into a catch basin structure or other suitable outlet.

Should the subgrade comprise free-draining granular soils (minimum 1.0 m thick with positive drainage at the interface with any less permeable soils), then the installation of subdrains may not be required.

The native subgrade soils are sensitive to change in moisture content and can become loose or soft if the soils are subject to inclement weather and seepage or severe drying. Furthermore, the subgrade soils could be easily disturbed if traveled on during construction. As such, where this material will be exposed, it is recommended that the granular subbase be placed immediately upon completion of the subgrade preparation to protect the integrity of the subgrade soils.

It is expected that the roadways will be subject to light traffic (personal vehicles) as well as heavy traffic (delivery trucks, maintenance and emergency vehicles). Based on the anticipated loading, the following pavement design is provided:

Material	Recommended Thickness For New Pavement	
	Light Duty	Heavy Duty
Asphaltic Concrete	HL3 - 40 mm (1.5") HL4 or HL8 - 50 mm (2.0")	HL3 - 50 mm (2.0") HL4 or HL8 - 60 mm (2.5")
Granular 'A' Base (OPSS 1010)	150 mm (6.0")	150 mm (6.0")
Granular 'B' Subbase (OPSS 1010)	400 mm (16.0")	450 mm (18.0")

Should wet to saturated conditions be encountered during construction, site assessments may be required at the time of construction to determine what options can be undertaken to construct a stable roadway base. These options may include subexcavation and increasing the thickness of the Granular 'B' subbase, the use of reinforcing geotextile and/or geogrid, or a combination of all. As such, it is recommended that provisions for subexcavation and disposal of wet soils, importing and placing additional Granular 'B' (OPSS 1010), as well as supply and placement of a reinforcing geotextile (Terrafix 270R or equivalent) and geogrid (Tensar BX1200 or equivalent) should be included in the tender documents.

The granular base and subbase materials must conform to the physical property and gradation requirements of OPSS 1010 and must be compacted to 100% SPMDD. Asphaltic concrete should be supplied, placed and compacted to a minimum 92.0% Marshall maximum relative density, in accordance with OPSS 1150 and OPSS 310.

Construction joints in the surface asphalt must be offset a minimum of 150 mm to 300 mm (6" to 12") from construction joints in the binder asphalt so that longitudinal joints do not coincide.

Frost tapers must be constructed at any changes from light traffic to heavy traffic areas within the roadways, parking areas and driveways. If it is anticipated that heavy equipment (such as loader and dump trucks) will be utilized for snow removal, it would be recommended that the heavy traffic pavement structure be utilized throughout the all roadways.

Where new asphalt is joined into existing asphalt, it is recommended that the existing asphalt be sawcut in a straight line prior to being milled to a depth of 40 mm and a width of 150 mm as per OPSD 509.010. It is recommended that a tackcoat in conformance with OPSS 308 be applied to the edge and surface of all milled asphalt prior to placement of new asphalt.

The pavement should be designed to ensure that water will not pond on the pavement surface. If the surface asphalt is not placed within a reasonable time following placement of the binder asphalt, it is recommended that the catch basin lids are set at a lower elevation or apertures provided to allow surface water to drain into the catch basins and not accumulate around the catch basins. The strength of the pavement structure relies on all of the components to be in place in order to provide the design strength; therefore, it is strongly recommended that the surface asphalt be placed shortly after placement of the binder asphalt so as to avoid undue stress on the binder asphalt by not having the complete pavement structure in place.

It should be noted that, currently, asphalt mixes tend to be more flexible and, as such, there is a tendency for damage to occur from vehicles turning their steering wheels or applying excessive brake pressure. The damage can occur from both passenger vehicles as well as large vehicles. The condition is further intensified during hot weather. In high traffic/tight turning areas, it is recommended that rigid Portland cement pavement be considered.

5.12. Excess Soil Management

5.12.1. Chemical Testing was NOT Undertaken

Generally, if surplus soils are to be exported off-site, it will be necessary to perform chemical analysis of the soils. Chemical analysis was not undertaken as part of this geotechnical investigation. Should chemical analysis tests be required, the required tests vary and will be dependent on the disposal site utilized by the general contractor.

Most commonly, the soils are tested for the following:

- F1-F4, VOC's, BTEX as per O. Reg. 153/04 as amended by R511
- SVOC as per O. Reg. 153/04 as amended by R511
- Metals/Inorganics as per O. Reg. 153/04 amended by R511

The chemical analysis results are then compared to Ontario Regulation 153/04 - as amended by O.Reg. 511 – April 15, 2011 Standards = [Suite] – ON-511-T1/T2-SOIL-RPI.

5.12.2 TCLP Requirement

If soils are transported to a landfill facility, additional chemical testing in accordance with Ontario Regulation 347, Schedule 4, as amended to Ontario Regulation 558/00, dated March 2001, Toxicity Characteristic Leaching Procedure (TCLP) will be required.

When transporting soils off-site, the following is recommended:

- All chemical analyses and environmental assessment reports must be fully disclosed to the receiving site owners/authorities, whom must agree to receive the material;
- An environmental consultant must confirm the land use at the receiving site is compatible to receive the material;
- An environmental consultant must monitor the transportation and placement of the materials to ensure that the material is placed appropriately at the pre-approved site;
- The excess materials may not be transported to a site that has previously had a Record of Site Condition (RSC) filed, unless the material meets the criteria outlined in the RSC.

It should be noted that landfill sites will generally only accept laboratory test results that have been completed within 30 days of exporting. Therefore, it is recommended that provisions for chemical analysis be included in the tender documents. It should also be noted that the laboratory testing generally takes five (5) working days to process with a regular turnaround time.

5.13. Stormwater Infiltration

As part of the geotechnical investigation completed at this site, gradation analyses were performed on representative samples of the native soils. The following table provides the sample location (borehole number), sample depth, corresponding estimated coefficient of permeability (k), as well as soil type:

Borehole No.	Depth (m)	Estimated Coefficient of Permeability (k) cm/s	Soil Type
1	2.29 – 2.90	$<1.0 \times 10^{-6}$	Silt, some clay, trace sand and gravel (ML)
2	6.70 – 7.62	4.0×10^{-4}	Silty Sand, trace clay (ML)

5.14. Radon

According to information provided by Health Canada, radon is a radioactive gas that is naturally formed through the breakdown of uranium in soil, rock and water. When radon escapes the earth in the outdoors, it mixes with fresh air, resulting in concentrations that are too low to be of concern. However, when radon enters an enclosed space, such as a building, high concentration of radon can accumulate and become a health concern. Health Canada indicates that most buildings and homes have some level of radon in them. Unfortunately, it is not possible to predict before construction whether or not a new building will have high radon levels as radon can only be detected by radon measurement devices, which would be installed in a building, post construction. Section 9.13.4.1 Soil Gas Control of the current 2012 Ontario Building Code (OBC) states that *"Where methane or radon gases are known to be a problem, construction shall comply with the requirements for soil gas control in MMAH Supplementary Standard SB-9, Requirements for Soil Gas Control"*.

6.0 SLOPE STABILITY ASSESSMENT

It is understood that the project will involve the construction of a new residential development (condominiums) with associated roadways, underground utilities and stormwater management facilities. The Grey Sauble Conservation Authority has identified slope hazards along the west side of the subject property near the rear of the proposed Lots 4, 6 and 8 adjacent to an unnamed tributary to Pete's Creek. Since the proposed construction would be located at the top of the regulated slope, a slope stability assessment was undertaken as part of this geotechnical investigation.

Utilizing the information obtained during the geotechnical investigation and assuming that the borehole information is representative of the subsoil conditions throughout the site, the following comments and recommendations are provided.

6.1. Existing Slope Assessment

In order to assess the current stability of the existing slopes, a slope stability analysis was completed. One (1) slope cross-section located through the west facing slope (referenced as Cross-Section A-A) was reviewed. The cross-section location is shown on Drawing 2. Based on measurements obtained from a topographic survey provided by GSS Engineering Consultants Ltd., the steepness of the slope was estimated. The top (crest) and bottom (toe) of the slope was determined by estimating where the slope steepness is less than 6.7H:1.0V. The west slope at the cross-section location (A-A) generally has an overall steepness of approximately 3.97H:1.0V.

CMT Inc. staff conducted an on-site visual inspection of the existing slope conditions. In general, the west slope (A-A) was well-vegetated with a significant amount of large trees throughout the slope and low-lying vegetation over the remainder of the area. There were no observed signs of slope instability such as seepage, slumps, erosion features or tension cracks. The toe of the slope is located within the associated floodplain limit based on drawings from the GSCA. Based on Table 8.1 - Slope Stability Rating Chart from *Geotechnical Principles for Stable Slopes, 1997* produced by Terraprobe for the Ontario Ministry of Natural Resources (MNR) (same as Table 4.2 – Slope Stability Rating Chart from *Technical Guide – River and Stream Systems: Erosion Hazard Limit, 2002* by the MNR), the west slope was determined to have a total rating value of 33 and is considered to have a slight potential for slope instability (see Appendix C).

6.1.1. Slope Stability Assessment

The stability of the slope was assessed using Bishop's simplified method. With this method, the factor of safety of a slope is determined by comparing the moment of the weight of a soil wedge about the centre of a slip circle, with the resisting moment provided by the shear stresses along the slip surface.

Based on the information collected in the boreholes, the following table shows the estimated soil parameters that were used for the slope stability analysis:

Soil Type	Unit Weight (kN/m ³)	Friction Angle	Cohesion (kPa)
Sandy Silt Fill	17.0	28°	0
Silt Till	18.0	30°	10
Silty Sand Till	17.5	32°	0

The above parameters are estimated based on the information obtained from the boreholes and are assumed to be consistent throughout the site.

A Factor of Safety of 1.0 is considered to represent a potential failure condition. As per Table 4.3 of *Technical Guide – River and Stream Systems: Erosion Hazard Limit, 2002* by the MNRF, the land use of the site would be classified as "Active" (habitable or occupied structures near slope). A Factor of Safety of 1.5 is considered to be adequate for this site with respect to shallow and deep-seated (global) failure surfaces.

The slope stability analysis was completed utilizing the SLIDE software package developed by Rocscience. Based on the analysis completed, a minimum Safety Factor of 3.292 against deep seated slope failure was determined for the existing slope and loading conditions at cross-section A-A. Based on the results of the analysis, the existing west slope (A-A) is considered to be stable in its current configuration and has an estimated stable slope angle of approximately 27°.

It should be noted that no slip surfaces with a factor of safety (FOS) less than 1.5 have been identified in the cross-sectional analysis of the existing slope. In some cases, slip surfaces which are surficial in nature are encountered. The slope stability analysis software does not take the presence of topsoil and vegetation into consideration, which is why surficial slip surfaces with lower factors of safety may exist in a cross-sectional analysis. As long as the slope remains well-vegetated, the existing slopes should be safe from surficial failures.

The results of the slope stability analysis including the safety factors achieved for the existing slope are provided in Drawing 4 (Cross-Section A-A). The existing top of **stable** slope location is provided in Drawing 3, 4 and 5.

6.2. Flooding/Erosion Hazards

The slope was also assessed with respect to the *GSCA Policies for the Administration of the Development, Interference with Wetlands and Alteration to Shorelines & Watercourses Regulation (Ontario Regulation 151/06) Section 8*. Based on drawings provided by the GSCA with respect to regulated floodplain setback area, the floodplain setback of 30.0 m does fall within the property boundaries, however the proposed development is to be outside of the floodplain setback area. As such, there should be no concern to flooding hazards for the proposed construction. However, the floodplain does contact the toe of the slopes, and as such, there is a concern due to erosion of the base of the slopes resulting in potential failure.

The erosion allowance of the existing watercourse was determined from the Ministry of Natural Resources Technical Guide for River and Stream Systems: Erosion Hazard Limit. From Table 3 of the above-mentioned Technical Guide, the recommended erosion over a period of 100 years on the slope would be about 5.0 to 8.0 m. There was observed to be evidence of minor erosion along the existing creek bank, however the native soils appeared to consist of stiff/hard, cohesive clays or clayey silts. Since the existing erosion was considered to be minor along the existing creek bank, an estimate of 5.0 m of erosion over 100 years, or an erosion rate of 0.05 m/year was used for this assessment and was applied to the cross-sections for analysis.

Based on the analysis completed on the existing slope and applying 5.0 m of erosion at the toe of the slope, as well as the approximate loading of the proposed development, a minimum Safety Factor of 2.348 was determined against deep seated slope failure for the existing slope at cross-section A-A. Based on the results of the analyses, the west slope (A-A) should remain stable over the next 100 years. The 100-year stable slope angle for cross-section A-A is estimated to be 34°.

The results of the 100-year slope stability analysis including the safety factors achieved for the slope is provided in Drawing 5 (Cross-Section A-A). The estimated 100-year **stable** slope location of the existing slope is provided in Drawing 3.

As per Sections 8.2.2 to 8.2.11 of the GSCA Policies, a minimum setback of 6.0 m from the riverine erosion hazard to the proposed construction is required for emergency erosion control access. This setback is shown on Drawing 3.

With respect to Section 8.2.2 of the *GSCA Policies for the Administration of Development, Interference with Wetlands and Alternation to Shorelines & Watercourses Regulation (Ontario Regulation 151/06)*, it is the opinion of CMT Inc. that:

- Due to the nature and size of the proposed development site, there is not sufficient space outside of the regulated area to construct the proposed development.
- The proposed development location is not anticipated to impact the existing or future slope stability, with factors of safety greater than 1.5 as shown in Sections 6.1 and 6.2 of this report.
- The potential for surficial erosion to aggravate or create an erosion hazard should be decreased due to the control of water to appropriate outlets (addressed by drainage plan). The control of surface water will decrease the amount of water entering/affecting the existing slope conditions.
- Access for preventative actions or maintenance (6.0 m erosion control access) should not be affected and should be addressed in the site design drawings.

From a geotechnical perspective, there should not be a concern for the proposed development to meet the above requirements; however, proposed site plans, drainage plans and grading plans for the proposed development should be submitted to the GSCA to address the additional requirements of the above-mentioned policy. Grading and drainage plans should address the control of surface water over soft and hard landscaped areas, while pavement and rooftop drainage should be directed to swales, catch basins, storm sewers and/or on-site infiltration galleries. It should be understood that the GSCA will make the final determination on the approval of the proposed development.

CMT Inc. would be pleased to review design drawings when they become available and make any additional comments or recommendations for the proposed development with respect to the *GSCA Policies for the Administration of Ontario Regulation 151/06*.

7.0 SITE INSPECTION

Qualified geotechnical personnel should supervise excavation inspections as well as compaction testing for structural filling, site grading and site servicing. This will ensure that footings are founded in the proper strata and that proper material and techniques are used and the specified compaction is achieved. CMT Engineering Inc. would be pleased to review the design drawings and provide an inspection and testing program for the construction of the proposed development.

8.0 LIMITATIONS OF THE INVESTIGATION

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete, or if the proposed construction should differ from that mentioned in this report.

It is important to emphasize that a soil investigation is, in fact, a random sampling of a site and the comments are based on the results obtained at the test locations only. It is therefore assumed that these results are representative of the subsoil conditions across the site. Should any conditions at the site be encountered which differ from those found at the test locations, we request that we be notified immediately in order to permit a reassessment of our recommendations.

It should be noted that this report specifically addresses geotechnical aspects of the project and does not include any investigations or assessments relating to potential subsurface contamination. As such, there should be no assumptions or conclusions derived from this report with respect to potential soil or water contamination. Soil or water contamination is generally caused by the presence of xenobiotic (human-made) chemicals or other alteration processes in the natural soil and groundwater environment. If necessary, the investigation, assessment and rehabilitation of soil and water contaminants should be undertaken by qualified environmental specialists.

The samples obtained during the geotechnical investigation will be stored for a period of three months, after which time they will be disposed of unless alternative arrangements are made.

This report is intended solely for the client named. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the reliability of such third parties. The factual data, interpretation, and recommendations in this report pertain to a specific project as described in this report and are not applicable to any other project or site location. If the project is modified in concept, location or elevation, deviates from the assumptions stated herein, CMT Inc. should be given an opportunity to confirm that the recommendations are still valid. The subject geotechnical exploration and this report address only the geotechnical aspects of the proposed project; potential environmental impacts or related issues are beyond the defined scope of this work and have not been addressed.

We trust that this report meets with your present requirements. Should you have any questions, please do not hesitate to contact our office.

Prepared by:



Brandon R Figg, C.Tech.
Senior Soil Technician

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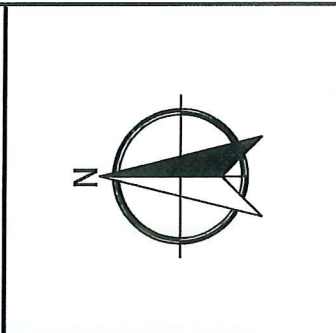
Reviewed by:

Nathan Chortos, P.Eng.
Senior Geotechnical Engineer

NOTES:

1. BASE MAP PROVIDED BY GOOGLE MAPS

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ENGINEERING

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Slope Stability Investigation
Meaford Golf Course
408 Ridge Road,
Meaford, Ontario

DRAWING TITLE:

SITE LOCATION MAP

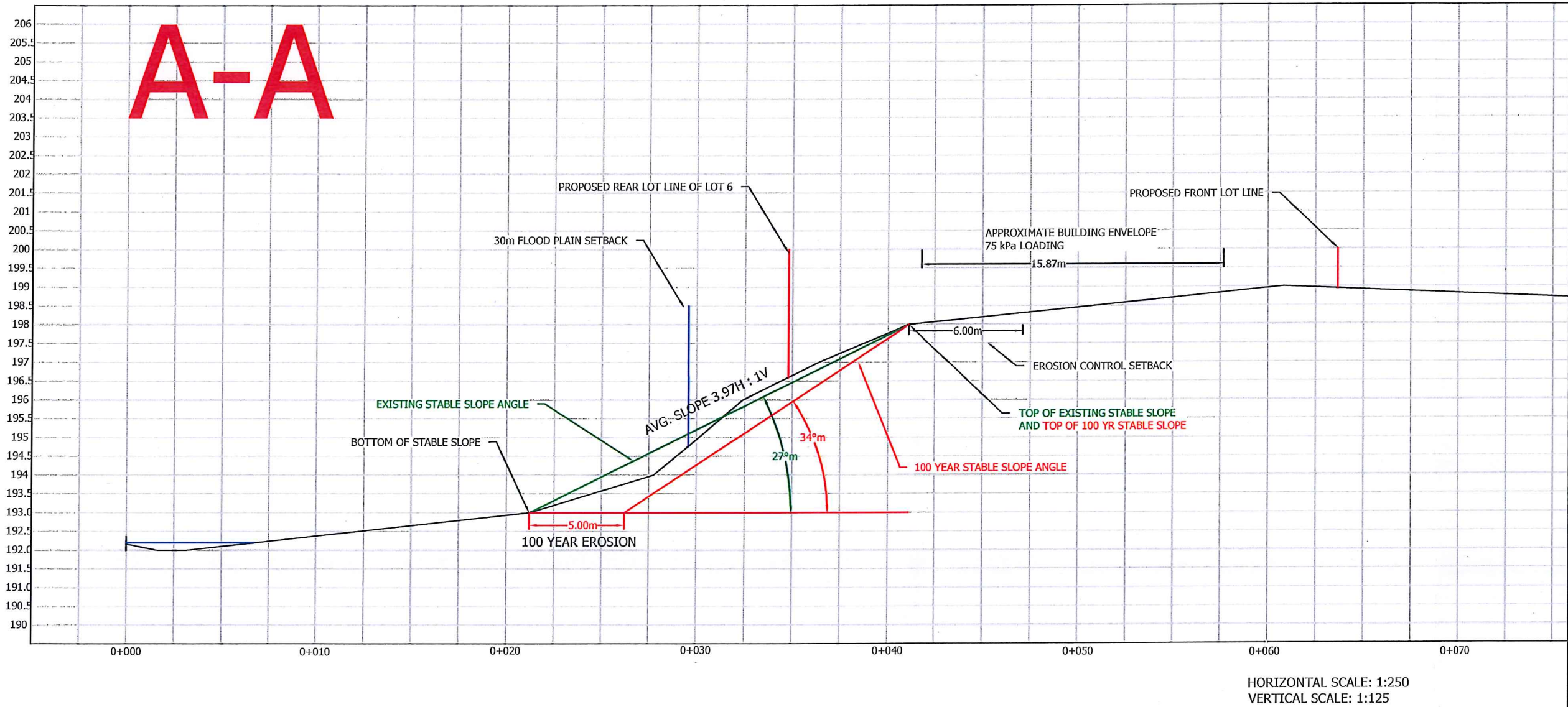
19-623	January 20, 2020
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N.T.S.	1
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NOTES:

1. SLOPE CROSS-SECTION AS PER TOPOGRAPHIC SURVEY COMPLETED BY GSS ENGINEERING CONSULTANTS LTD.



NO.	DESCRIPTION	DATE

REVISIONS

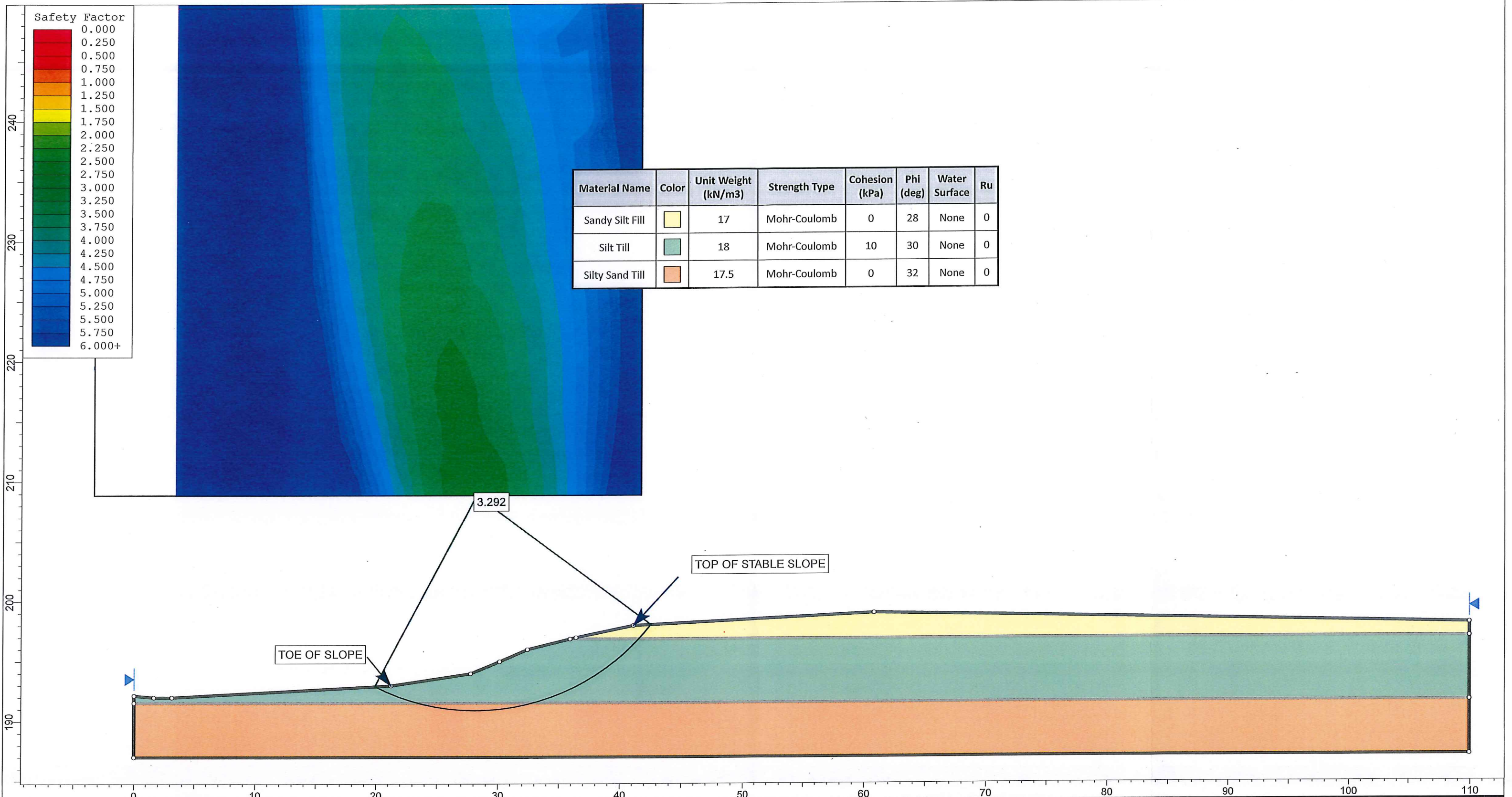


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PROJECT:
Geotechnical Investigation/
Slope Stability Study
408 Ridge Road,
Meaford, Ontario

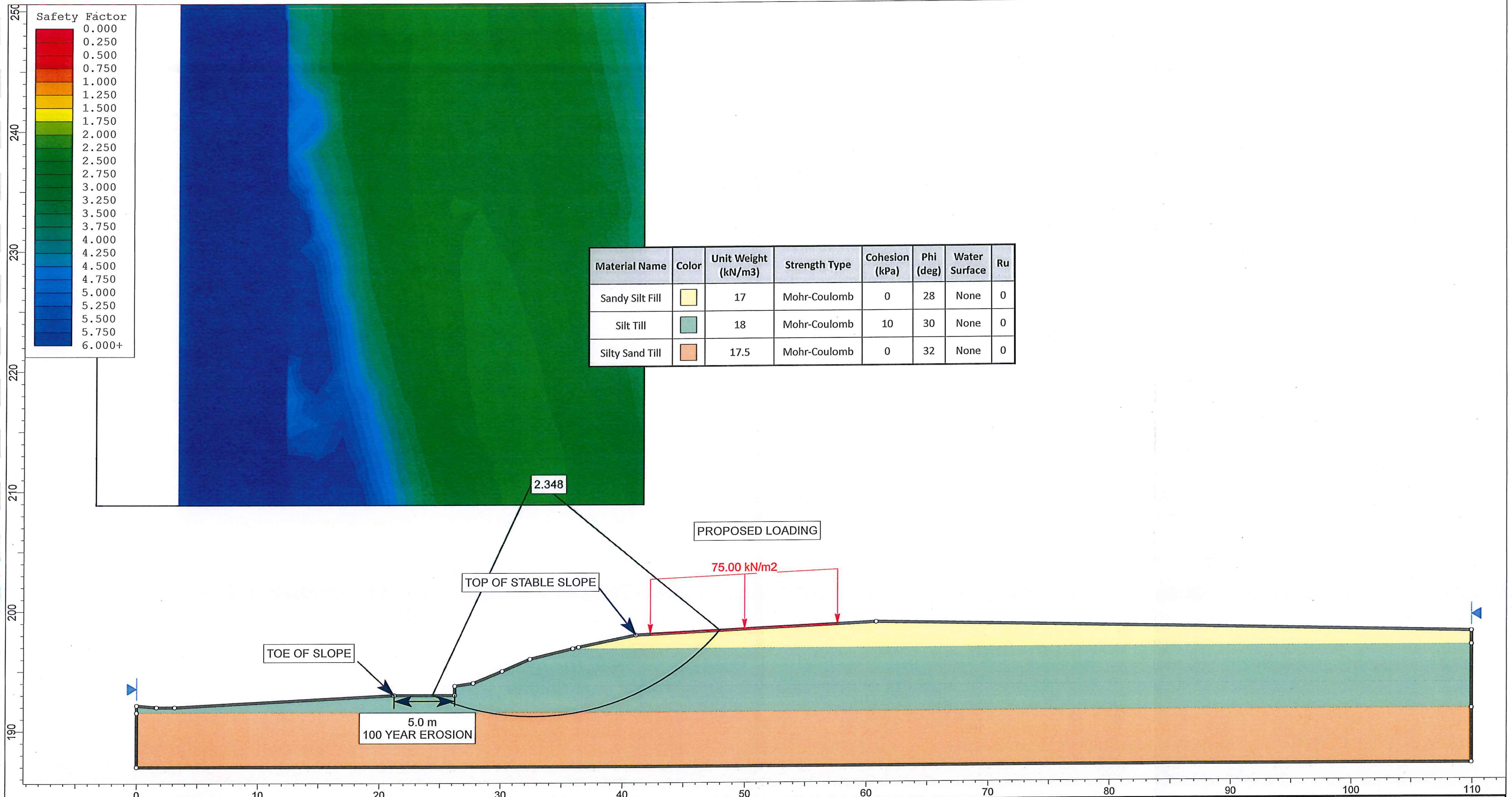
DRAWING TITLE:
SLOPE CROSS SECTION
A-A

PROJECT NO.: 19-623	DATE: February 5, 2020
SCALE: NOTED	DRAWING NO.: 3





Project			
DRAWING 4 - SLOPE CROSS SECTION A-A SHOWING MINIMUM FACTORS OF SAFETY ON EXISTING SLOPE			
Location			
408 Ridge Road, Meaford, ON			
Drawn By		Scale	Company
BRF		1:300	CMT ENGINEERING INC
Date		Job Number	
February 5, 2020		19-623	



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Ru
Sandy Silt Fill		17	Mohr-Coulomb	0	28	None	0
Silt Till		18	Mohr-Coulomb	10	30	None	0
Silty Sand Till		17.5	Mohr-Coulomb	0	32	None	0



Project				DRAWING 5 - SLOPE CROSS SECTION A-A			
Location				SHOWING MINIMUM FACTORS OF SAFETY ON SLOPE WITH 100 YEAR EROSION AND LOADING			
408 Ridge Road, Meaford, ON							
Drawn By		BRF		Scale		Company	
				1:300		CMT ENGINEERING INC	
Date		February 5, 2020		Job Number		19-623	

APPENDIX A
BOREHOLE LOGS

BOREHOLE 1

Page 1 of 1

Date Drilled: January 9, 2020
Rig: Geoprobe 7822DT
Contractor: CMT Drilling Inc.
Drilling Method: SPT/MC5

Elevation: 199.17 m
Logged by: B.R.F.

Project No.: 19-623
Project: Geotechnical Investigation and
Slope Stability Analysis
Location: 408 Ridge Road, Meaford, ON

Depth (ft/m)	Sample Type	Recovery (%)	Sample Number	Symbols	SOIL DESCRIPTION	Well Installation	Moisture Content % Wp [---X---] Wl	Pocket Penetrometer kPa 100 200 300 400 SPT (N) Blows/0.3 m 20 40 60 80
0					Ground Surface (m) 199.17			
0					0.00			
1	SS	1			TOPSOIL Black, loose, silty organic topsoil, moist (250mm)			3
2								
3	SS	2			SANDY SILT FILL Dark brown to brown, loose to compact, sandy silt fill, some clay, trace gravel, moist		16.6	13
4								
5								
6	SS	3					13.1	24
7								
8								
9	SS	4			SILT TILL Brown, stiff, silt till, some clay, trace sand and gravel, moist		18.1	11
10								
11								
12	SS	5			becoming very stiff		18.2	20
13								
14								
15	MC5	6					19.1	
16								
17								
18								
19	SS	7			becoming hard		17.6	78
20								
21								
22	MC5	8					22.1	
23								
24								
25	SS	9			occasional wet seams		20.6	50/75 mm
26								
27								
28	MC5	10					10.1	
29								
30								
31	SS	11			SILTY SAND TILL Grey, very dense, silty sand till, trace clay, moist		10.6	50/75 mm
32								
33								
					End of Borehole			
					Caving of the boreholes observed at approximately 6.40m (El. 192.77m) below ground surface. Groundwater was not encountered upon completion of the borehole.			

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BOREHOLE 2

Page 1 of 1

Date Drilled: January 9, 2020
Rig: Geoprobe 7822DT
Contractor: CMT Drilling Inc.
Drilling Method: SPT/MC5

Elevation: 199.45 m
Logged by: B.R.F.

Project No.: 19-623
Project: Geotechnical Investigation and
Slope Stability Analysis
Location: 408 Ridge Road, Meaford, ON

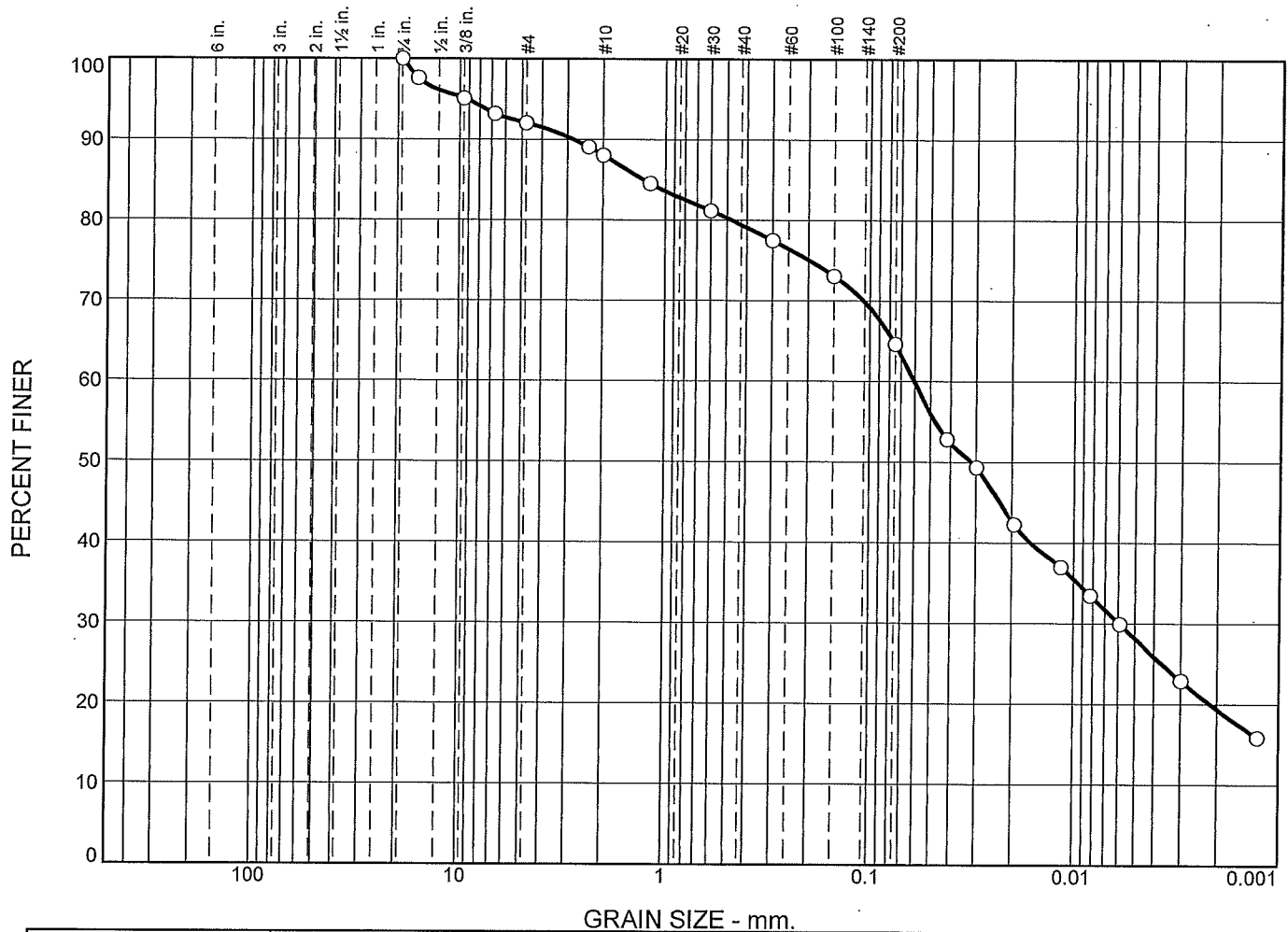
Depth (ft/m)	Sample Type	Recovery (%)	Sample Number	Symbols	SOIL DESCRIPTION	Well Installation	Moisture Content % Wp [---X---] WI 10 20 30 40 50	Pocket Penetrometer kPa 100 200 300 400 SPT (N) Blows/0.3 m 20 40 60 80
0					Ground Surface (m) 199.45			
1	SS		1		TOPSOIL Black, loose, silty organic topsoil, moist (50mm)			2
2								
3	SS		2		SILTY SAND FILL Dark brown, very loose to loose, silty sand fill, some clay, trace gravel, moist		21.3	8
4								
5					197.93			
6	SS		3		SILT TILL Brown, stiff, silt till, some clay, trace sand, moist with occasional wet seams		16.1	11
7								
8	SS		4				17.6	14
9								
10					196.40			
11	SS		5		becoming very stiff		15.8	24
12								
13							18.8	
14	MC5		6					
15					194.88			
16	SS		7		becoming hard		16.8	41
17								
18							16.3	
19	MC5		8					
20							15.6	80
21	SS		9					
22								
23	MC5		10				11.1	
24					192.13			
25					SILTY SAND TILL Grey, very dense, silty sand till, trace clay, moist			
26	SS		11				10.1	50/125 mm
27					191.22			
28					End of Borehole 8.23			
29								
30					Caving of the boreholes observed at approximately 5.03m (El. 194.42m) below ground surface. Groundwater was not encountered upon completion of the borehole.			
31								
32								
33								

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APPENDIX B
GRAIN SIZE ANALYSES

Particle Size Distribution Report



	% Cobbles	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	8.0	4.0	8.6	14.8	45.2	19.4

SOIL DATA					
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	BH1	2	0.76-1.37m	sandy silt, some clay, trace gravel	CL-ML
				Sampled by BF of CMT Engineering Inc., January 9, 2020	
				Tested by JM of CMT Engineering Inc., January 15, 2020	

CMT Engineering Inc.

St. Clements, ON

Client: Meaford Golf Course Inc.

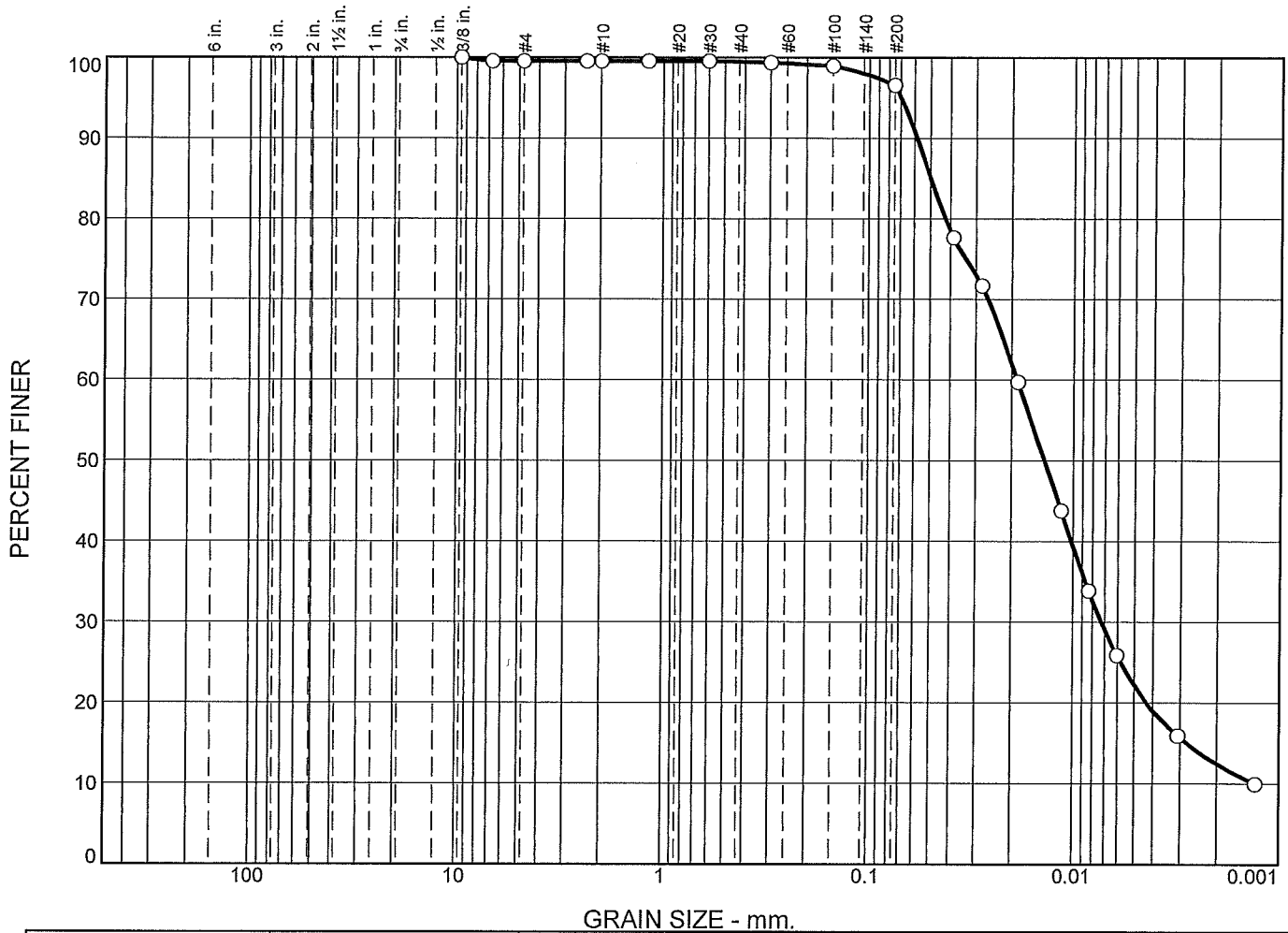
Project: Residential Development, Area "B"

Hilton Head Heights Condominiums - 408 Ridge Road, Meaford

Project No.: 19-623

Figure 1

Particle Size Distribution Report



	% Cobbles	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.5	0.0	0.0	2.9	84.1	12.5

SOIL DATA					
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	BH1	4	2.29-2.90m	silt, some clay, trace sand and gravel	ML
				Sampled by BF of CMT Engineering Inc., January 9, 2020	
				Tested by JM of CMT Engineering Inc., January 15, 2020	

CMT Engineering Inc.

St. Clements, ON

Client: Meaford Golf Course Inc.

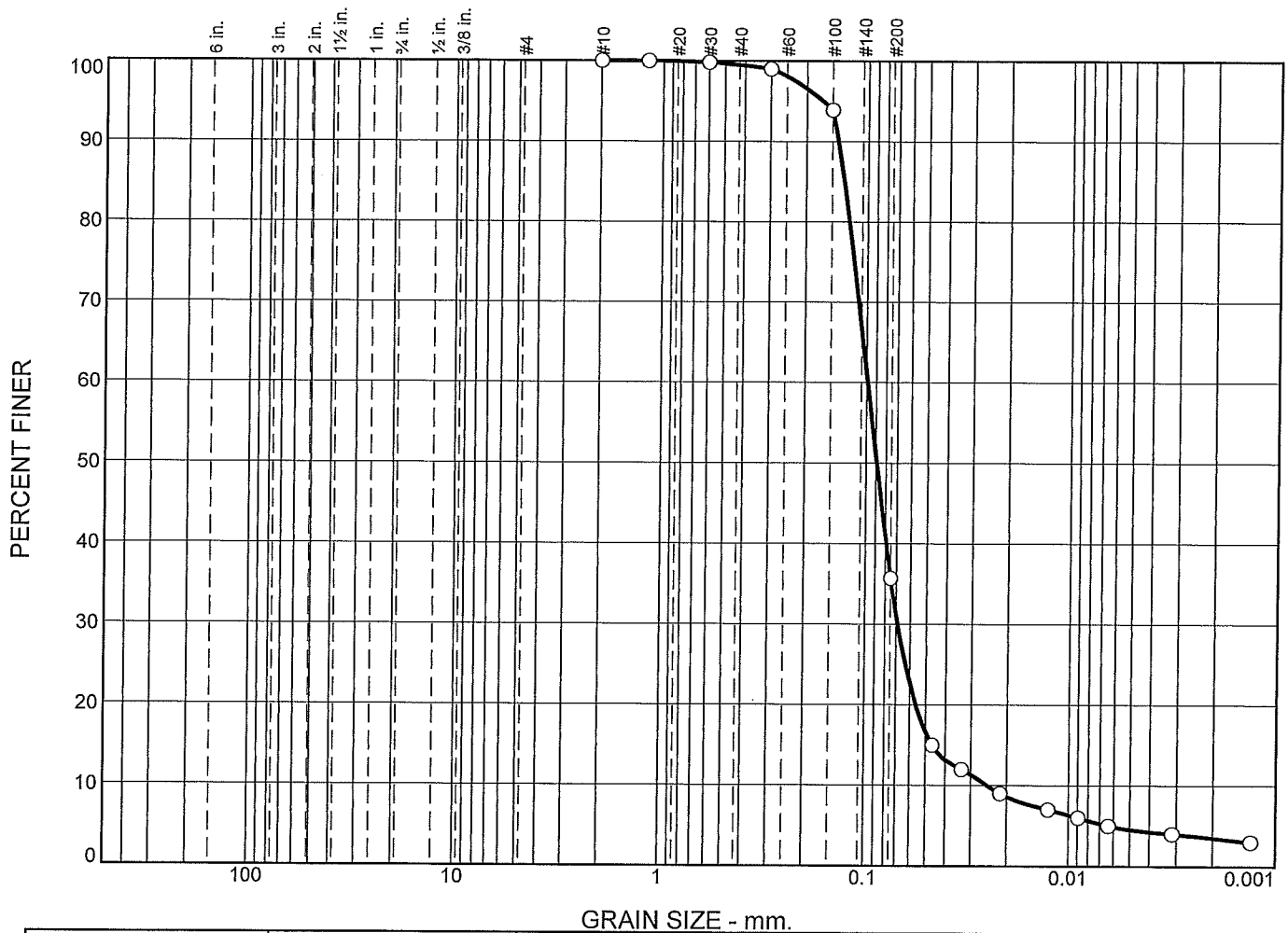
Project: Residential Development, Area "B"

Hilton Head Heights Condominiums - 408 Ridge Road, Meaford

Project No.: 19-623

Figure 2

Particle Size Distribution Report



	% Cobbles	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.0	0.0	0.5	63.8	32.2	3.5

SOIL DATA					
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	BH2	10	6.7-7.62m	silty sand, trace clay	SM
				Sampled by BF of CMT Engineering Inc., January 9, 2020	
				Tested by JM of CMT Engineering Inc., January 15, 2020	

CMT Engineering Inc.

St. Clements, ON

Client: Meaford Golf Course Inc.

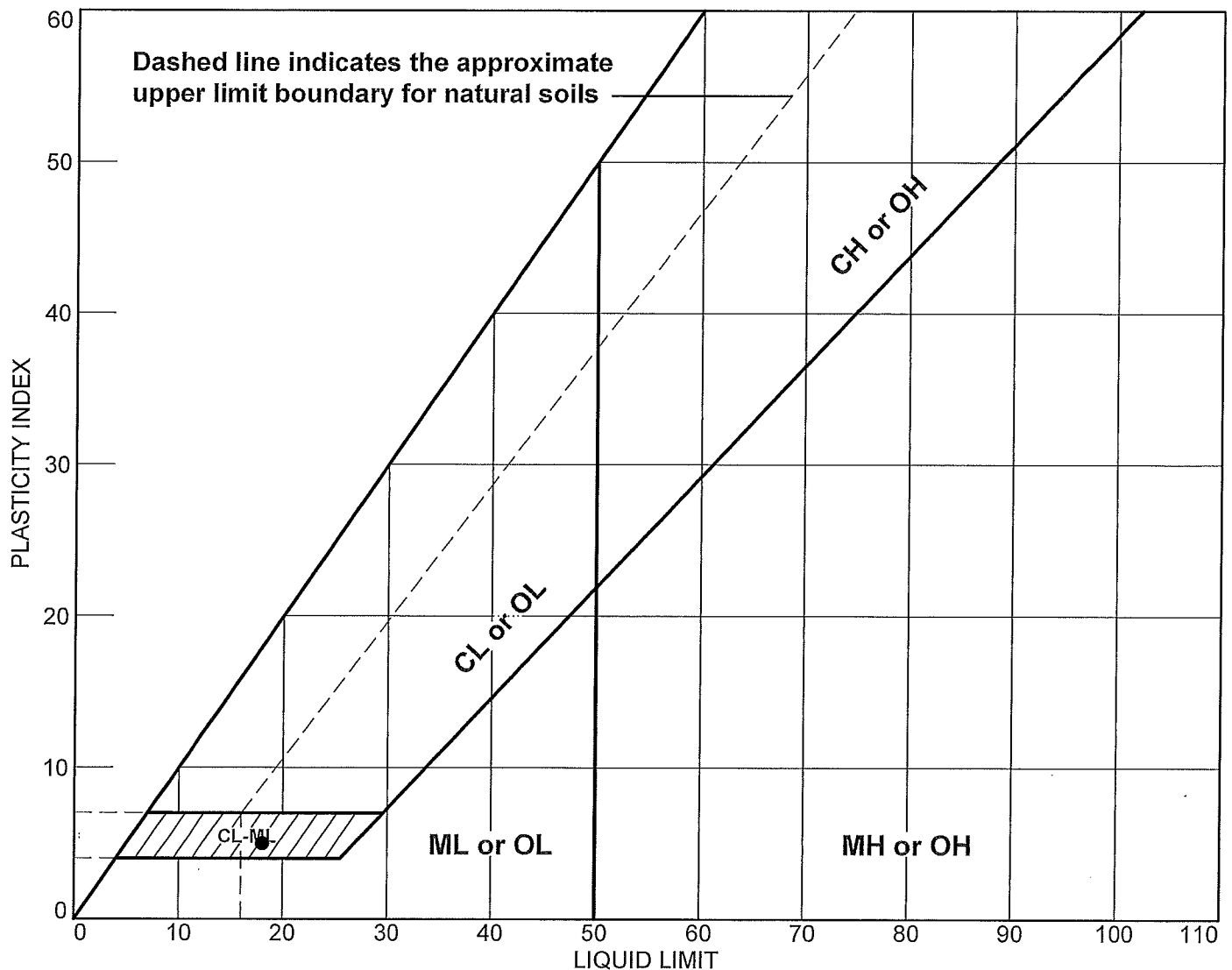
Project: Residential Development, Area "B"

Hilton Head Heights Condominiums - 408 Ridge Road, Meaford

Project No.: 19-623

Figure 3

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	BH1	2	0.76-1.37m		13	18	5	CL-ML

CMT Engineering Inc.

St. Clements, ON

Client: Meaford Golf Course Inc.

Project: Residential Development, Area "B"

Hilton Head Heights Condominiums - 408 Ridge Road, Meaford

Project No.: 19-623

Figure 4

APPENDIX C

SLOPE STABILITY RATING CHARTS

TABLE 4.2 - SLOPE STABILITY RATING CHART (SOUTH SLOPE)

Site Location	408 Ridge Road, Meaford, ON		Project No.	19-623	
Property Owner	Meaford Golf Course Inc.		Inspection Date	Jan. 9, 2020	
Inspected By	TS/BRF		Weather	Overcast, -10°C	

1. SLOPE INCLINATION		
degrees	horiz:vert	
a) 18 or less	3:1 or flatter	0
b) 18 - 26	2:1 to more than 3:1	6
c) more than 26	steeper than 2:1	16
2. SOIL STRATIGRAPHY		
a) shale, limestone, granite (bedrock)		0
b) sand, gravel		6
c) glacial till		9
d) clay, silt		12
e) fill		16
f) leda clay		24
3. SEEPAGE FROM SLOPE FACE		
a) none or near bottom only		0
b) near mid-slope only		6
c) near crest only or from several levels		12
4. SLOPE HEIGHT		
a) 2.0 m or less		0
b) 2.1 m to 5.0 m		2
c) 5.1 m to 10.0 m		4
d) more than 10.0 m		8
5. VEGETATION COVER ON SLOPE FACE		
a) well-vegetated, heavy shrubs or forested with mature trees		0
b) light vegetation; mostly grass, weeds, occasional trees, shrubs		4
c) no vegetation, bare		8
6. TABLE LAND DRAINAGE		
a) table land flat, no apparent drainage over slope		0
b) minor drainage over slope, no active erosion		2
c) drainage over slope, active erosion, gullies		4
7. PROXIMITY OF WATERCOURSE TO SLOPE TOE		
a) 15 metres or more from slope toe		0
b) less than 15 metres from slope toe		6
8. PREVIOUS LANDSLIDE ACTIVITY		
a) no		0
b) yes		6
SLOPE STABILITY RATING VALUES INVESTIGATION RATING SUMMARY		TOTAL 33
SUMMARY OF RATING VALUES AND RESULTING INVESTIGATION REQUIREMENTS		
1. Low potential	< 24	- site inspection only, confirmation report letter
2. Slight potential	25-35	- site inspection and surveying, preliminary study, detailed report
3. Moderate potential	>35	- boreholes, piezometers, lab tests, surveying, detailed report
NOTES:		
a) Choose only one from each category; compare total rating value with above requirements.		
b) If there is a water body (stream, creek, river, pond, bay, lake) at the slope toe, the potential for toe erosion and undercutting should be evaluated in detail and protection provided if required.		

Reference: Technical Guide - River and Stream Systems: Erosion Hazard Limit, Ontario Ministry of Natural Resources, 2002.