Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION
BANCROFT DRIVE NEW SUBDIVISION
SUDBURY, ONTARIO

Prepared for:	Michael McDowell Holdings 2403 Navanod Road Sudbury, Ontario P3A 1A4
Attention:	Michael McDowell

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Distribution

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1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) was retained by Michael McDowell Holdings Inc. to carry out a geotechnical investigation to assess the subgrade soils for the proposed subdivision. The subject property is located at off Bancroft Drive, Sudbury, Ontario (see Figure 1).

Based on the results of the geotechnical investigation, engineering recommendations are presented for the following items:

- Frost depth;
- Appropriate types of foundations;
- Bearing capacity of the sub-strata;
- Excavation procedures;
- Trench stability;
- Bedding and compaction requirements;
- Suitability of on site soil to reuse as backfill;
- Foundation factors for earthquake forces;
- Geotechnical Construction Implications;
- Dewatering and drainage requirements;
- Considerations for constructibility
- Asphalt pavement recommendations.

2.0 SITE AND BRIEF PROJECT DESCRIPTION

The development of the Bancroft Drive Subdivision will consist of the development of five (5) residential properties accessed though a cul-de-sac.

The current site consist of a relatively low lying area covered with trees and wild grasses. The current access roadway off of Bancroft Drive provides access to the City of Greater Sudbury lift station located at the end of this road. Currently, the overall drainage is in a south direction.

The subject property is bound by the following:

North	-	Residential lots and Bancroft Drive;
West	-	Undeveloped wooded area;
South	-	City of Sudbury lift station and Canadian Pacific Railway;
East	-	Undeveloped wooded area

The single residential dwellings will consist of one or two storey wood frame buildings without basements. The residential lots would be serviced by the City of Greater Sudbury potable water and sanitary sewers.

The proposed lot grading plan was prepared by S. A. Kirchhefer Limited dated March 2022 entitled:

Residential Development
Michael McDowell Holdings Inc.
Grading Plan
Sudbury Area

3.0 GEOTECHNICAL INVESTIGATION

The geotechnical investigation was carried out between March 3rd and 4th, 2022. The field investigation consisted of advancing the following five (5) exploratory boreholes (see Figure 2 for the borehole locations):

1. **BH-1** located approximately at the intersection of the proposed roadway and Bancroft Drive
2. **BH-2** located within the building footprint on lot 1
3. **BH-3** located between lots 2 and 3
4. **BH-4** located within the development cul-de-sac
5. **BH-5** located between lots 4 and 5

Prior to conducing the exploratory borehole investigation, the underground services locates were provided by all members of Ontario One.

The location and the geodetic elevations of the boreholes were determined in the field by Bortolussi Surveying based (see Figure 2) based on the proposed site plan provided by S. A. Kirchhefer Limited.

The drilling work was carried out by Landcore Drilling utilizing a track mounted drill rig, equipped with hollow stem augers and conventional soil sampling equipment. The operation was monitored by a Terraprobe Engineer in Training (EIT) who logged the borings and examined the samples as they were obtained. All samples obtained from these boreholes were sealed into plastic jars, and transported to the Terraprobe laboratory for detailed inspection and testing. All of the borehole samples were examined (tactile) in detail by the project engineer and classified according to visual and index properties.

The Standard Penetration Test (SPT) was used to obtain samples of the strata penetrated in the exploratory boreholes, using the Split-Barrel Method technique as outlined in ASTM D1586. The soil samples were taken with a conventional 50 mm diameter split barrel sampler at 0.75 m intervals within the upper 3.0 metres and 1.5 metres intervals thereafter. The conventional interval sampling procedure used for this investigation does not recover continuous samples of soil at any borehole locations. There is consequently some interpolation of the borehole layering between samples and indications of changes in stratigraphy as shown on the borehole logs are therefore approximate.

One (1) piezometer was installed in borehole BH-3 to monitor the groundwater elevation. The groundwater elevation in the piezometer was measured on March 4, 2022 and is noted on the borehole log in Appendix A.

4.0 SUBSURFACE CONDITIONS

Details of the subsurface conditions encountered at the site are summarized below. The subsurface soil and groundwater conditions encountered in the boreholes are presented on the attached Log of Borehole sheets in Appendix A.

It should be noted that the subsurface conditions are confirmed at the borehole locations only. The stratigraphic boundaries indicated on the Log of Borehole sheets are inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil or rock type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions have been confirmed in a series of widely spaced boreholes, and will vary between and beyond the borehole locations. The following discussion has been simplified in terms of the major soil and rock strata for the purposes of geotechnical design. It may not be possible to drill a sufficient number of boreholes and sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling.

All of the soil samples, that were retrieved from this geotechnical investigation, were tested in our soils laboratory to determine the water contents. In addition, grain size analysis were conducted on selected soil samples. The results of this soil testing is presented in Appendix B.

4.1 Soil Stratigraphy

BH-1 The surface strata consisted of a black to brown, soft to firm Clayey SILT, trace sand stratum that was moist and approximately 2.29 metres thick. The Clayey SILT stratum was underlain by a grey to brown, firm SILT & CLAY, trace sand stratum that was wet and approximately 2.28 metres thick. The SILT & CLAY stratum was underlain by a grey firm Clayey SILT, trace sand stratum that was saturated and extended to the full depth of the borehole of 6.70 metres.

BH-2 The surface strata consisted of a brown, firm Clayey SILT, trace sand stratum that was moist to wet and approximately 1.52 metres thick. The Clayey SILT stratum was underlain by a brown to grey, soft to firm SILT & CLAY, trace sand stratum that was wet to saturated and approximately 3.98 metres thick. The SILT & CLAY stratum was underlain by a grey firm Clayey SILT, trace sand stratum that was saturated and extended to the full depth of the borehole of 6.70 metres.

BH-3 The surface strata consisted of a brown to grey, firm Clayey SILT, trace sand stratum that was moist and approximately 0.76 metres thick. The Clayey SILT stratum was underlain by a brown to grey, firm to stiff SILT & CLAY, trace sand stratum that was wet and approximately 3.05 metres thick.

The SILT & CLAY stratum was underlain by a brown compact Clayey SILT, trace sand, trace gravel stratum that was moist and extended to the full depth of the borehole of 4.27 metres. Auger was refusal was encountered at a depth of 4.27 metres on interpreted bedrock or boulders.

BH-4 The surface strata consisted of a dark brown, soft to firm Clayey SILT, trace sand stratum that was moist to wet and approximately 2.29 metres thick. The Clayey SILT stratum was underlain by a brown, firm SILT & CLAY, trace sand stratum that was wet and approximately 2.28 metres thick. The SILT & CLAY stratum was underlain by a grey, firm Clayey SILT, trace sand stratum that was moist and extended to the full depth of the borehole of 5.03 metres. Split spoon was refusal was encountered at a depth of 5.03 metres on interpreted bedrock or boulders.

BH-5 The surface strata consisted of a brown to grey, soft to stiff Clayey SILT, trace sand stratum that was moist to wet and approximately 1.52 metres thick. The Clayey SILT stratum was underlain by a brown to grey, firm to stiff SILT & CLAY, trace sand stratum that was wet and approximately 3.05 metres thick. The SILT & CLAY stratum was underlain by a grey, firm to stiff Clayey SILT, trace sand stratum that was wet to saturated and extended to the full depth of the borehole of 6.70 metres.

The following testing was conducted on representative soil samples:

1. Moisture contents.
2. Soil Gradations (hydrometers and sieve analysis)
3. Atterberg Limits
4. Consolidation analysis

Atterberg Limits tests were conducted on the following soil samples:

Sample 2 from BH-1 within the brown clayey SILT, trace sand stratum at a depth of approximately 0.90 metres. The results indicate a slightly plastic soil with slight or low compressibility which plots above the A-line and is classified as a CL (Inorganic clays of low to medium plasticity).

Sample 7 from BH-2 within the grey clayey SILT, trace sand stratum at a depth of approximately 6.40 metres. The results indicate a slightly plastic soil with moderate or intermediate compressibility which plots above the A-line and is classified as a CL (Inorganic clays of low to medium plasticity).

Sample 3 from BH-3 within the grey to brown SILT & CLAY, trace sand stratum at a depth of approximately 1.80 metres. The results indicate a medium plastic soil with moderate or intermediate compressibility which plots above the A-line and is classified as a CL (Inorganic clays of low to medium plasticity).

Sample 4 from BH-4 within the brown SILT & CLAY, trace sand stratum at a depth of approximately 2.50 metres. The results indicate a highly plastic soil with high compressibility which plots above the A-line and is classified as a CH (Inorganic clays of high plasticity).

Sample 5 from BH-5 within the brown to grey CLAY & SILT, trace sand stratum at a depth of approximately 3.35 metres. The results indicate a highly plastic soil with high compressibility which plots above the A-line and is classified as a CH (Inorganic clays of high plasticity).

The following table presents the soil stratigraphy encountered at each borehole location:

Borehole Soil Stratigraphy

Borehole (Elev.) (m)	Depth (m)	Subgrade Description	SPT Values 'N'	Water Content
BH - 1 (262.52)	0.00 - 2.29	1 - Clayey SILT, trace sand, black to brown, moist, soft to firm	4 - 7	19 - 24
	2.29 - 4.57	2 - SILT & CLAY, trace sand, grey to brown, wet, firm	5 - 6	32 - 37
	4.57 - 6.70	3 - Clayey SILT, trace sand, grey, saturated, firm	0 - 1	43 - 50
BH - 2 (262.26)	0.00 - 1.52	1 - Clayey SILT, trace sand, brown, moist to wet, firm	6	22 - 38
	1.52 - 5.50	2 - SILT & CLAY, trace sand, brown to grey, wet, soft to firm	5 - 6	31 - 44
	5.50 - 6.70	3 - Clayey SILT, trace sand, grey, saturated, firm	4	35
BH - 3 (261.95)	0.00 - 0.76	1 - Clayey SILT, trace sand, brown to grey, moist, firm		24
	0.76 - 3.81	2 - SILT & CLAY, trace sand, brown to grey, wet, firm to stiff	3 - 11	32 - 53
	3.81 - 4.27	3 - Clayey SILT, trace sand, brown, moist, compact	53	20
	4.27	4 - Auger refusal on interpreted bedrock or boulders		
BH - 4 (261.70)	0.00 - 2.29	1 - Clayey SILT, trace sand, brown, moist to wet, soft to firm	5 - 6	20 - 33
	2.29 - 4.57	2 - SILT & CLAY, trace sand, brown, wet, firm		
	4.57 - 5.03	3 - Clayey SILT, trace sand, grey, moist, firm	6 - 8	37 - 38
	5.03	4 - Auger refusal on interpreted bedrock or boulders	14	25
BH - 5 (260.81)	0.00 - 1.52	1 - Clayey SILT, trace sand, brown to grey, moist to wet, soft to stiff	11	23 - 29
	1.52 - 4.57	2 - SILT & CLAY, trace sand, brown to grey, wet, firm to stiff	3 - 5	36 - 41
	4.57 - 6.70	3 - Clayey SILT, trace sand, grey, wet, firm to stiff	1 - 2	34 - 47

4.1.1 Undrained Shear Strength

Field vane measurements were recorded from depths located between 3.66 to 7.01 metres below the existing grade in Boreholes BH-1, BH-2, BH-3 and BH-5.

The following table presents the corrected undrained shear strength of the underlying soils.

BH	Depth (m)	Elevation (m)	Soil Description	Peak Cu* (kPa)	Remoulded Cu (kPa)	Sensitivity	
BH-1	5.49 7.01	257.03 255.51	Clayey Silt Clayey Silt	41.10 35.96	12.84 7.71	3.20 4.67	Low sensitivity Medium sensitivity
BH-2	3.96 7.01	258.30 255.25	Silt and Clay Clayey Silt	22.63 30.82	4.53 7.71	5.00 4.00	Medium sensitivity Medium sensitivity
BH-3	3.66	258.29	Silt and Clay	35.85	8.96	4.00	Medium sensitivity
BH-5	3.96 5.49 7.01	256.85 255.32 253.80	Silt and Clay Clayey Silt Clayey Silt	95.03 51.37 30.82	24.89 10.27 7.71	3.82 5.00 4.00	Low sensitivity Medium sensitivity Medium sensitivity

*The measured undrained shear strength values obtained in the field were corrected using the PI from the atterberg tests.

The results indicate that the undrained shear strength of the grey to brown CLAY & SILT at a depth of 0.76 to 4.57 metres below the existing ground in boreholes 2, 3 and 5 ranged between 22.63 kPa to 95.03 kPa indicating a soft to stiff material. The remoulded shear strength indicated that the soils exhibit low to medium sensitivity to disturbance.

The results indicate that the undrained shear strength of the grey clayey SILT located at depths of 5.49 to 7.01 metres below the existing ground in boreholes 1, 2 and 5 varied from about 30.82 kPa to 51.37 kPa indicating a firm to stiff material. The remoulded shear strength indicated that the soils exhibit low to medium sensitivity to disturbance.

4.1.2 Probable Bedrock or Borehole Subgrade Elevation

The following table presents the recorded depths of the probable bedrock or large boulder based on auger refusal in certain boreholes.

Probable Bedrock or Boulder Subgrade Elevation

Rock Probe Location	Surface Elevation (m)	Depth to Probable Bedrock* (m)	Probable Bedrock* Subgrade Elevation (m)
BH - 3	261.95	4.27	257.68
BH - 4	261.70	5.03	256.67

* the depth to the probable bedrock or large boulders.

The auger refusal information from indicates that the underlying probable bedrock varies between 4.27 metres (BH - 3, probable bedrock elevation 257.68 metres) to 5.03 metres (BH - 4, probable bedrock elevation 256.67 m) below the existing grades. The data suggest that the underlying probable bedrock subgrade may generally slope in a eastward direction. However, it is anticipated that the bedrock depth may be erratic.

4.2 Groundwater

The representative soil samples retrieved from the boreholes were noted to be in a moist to wet/saturated condition. Based on the measured moisture contents of the soil samples and the measured water level in the piezometer, the estimated groundwater table will generally be located at approximately 260.53 m (BH-3) within the CLAY and SILT, trace sand stratum. Groundwater elevations that were measured in the piezometer on March 4th, 2022 are noted on the borehole logs in Appendix A.

It should be noted that the groundwater table is expected to fluctuate seasonally with higher levels expected during the spring and fall seasons.

4.3 Consolidation Testing

One (1) Shelby Tube was retrieved within the SILT & CLAY deposits in BH 2 as follows:

Borehole	Sample	Depth Below Grade	Elevation
2	6	4.57 to 5.03 m	257.69 to 257.23 m

One-dimensional consolidation tests conforming to ASTM D2435 were conducted on the relatively undisturbed sample that was retrieved from the borehole. The results of the consolidation testing are included in Appendix B.

5.0 GEOTECHNICAL DISCUSSION

The following discussions and recommendations are based on the factual data obtained from the investigation, and are presented for guidance of the design professionals only. The comments pertain to a specific project and location. This report is provided on the basis of these terms of reference and on the assumption that the preliminary design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards and guidelines of practice. If there are any changes to the site development features relevant to the interpretation made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Terraprobe should be retained to review the implications of these changes with respect to the contents of this report. Comments about construction are presented only to bring attention to aspects which might impact the design.

5.1 Foundation Discussion

The current soils investigation indicates that the underlying subgrade soils for this new building will consist of deposits of clayey SILTS to SILT & CLAYS. Auger refusal was encountered in both BH-3 and BH-4 on probable bedrock which extended up to 5.03 metres below the existing grade in borehole.

Based on the proposed current lot grading plan, it is anticipated that the local grades will be raised from 1.54 metres (Lot 1, BH-2) to 2.54 metres (Lot 4, BH-5). Based on the relatively weak consistency of the SILT & CLAYS, it is anticipated that the new building loads and backfill material will induce settlements which will result in unacceptable differential settlements at the bedrock interfaces.

Any existing peat and/or vegetation materials are not suitable for support of foundations. These materials would need to be sub-excavated and discarded from the site or utilised for landscaping purposes.

Based on the lot grading plan provided by S. A. Kirchhefer Limited, the following design elevations were provided:

1. Final Grade ranging between 263.30 to 264.05 metres
2. Footing elevation (1.80 m depth from FG) ranging between 261.50 to 262.25 metres

In order to proceed with the development of this project, the existing subgrade soils will need to be pre-loaded to consolidate the underlying soft soils found across the site in addition to the surcharge that will be incorporated to replace the peat soils.

All foundations should be designed to bear on an engineered fill placed on the undisturbed subgrade soils (Clayey SILT). The recommended minimum amount of engineered fill that should be placed on the consolidated undisturbed subgrade soils should be in the order of 600 mm.

The following table presents the maximum allowable bearing pressure recommended for the design of conventional strip footings placed on the engineered fill:

Bearing Capacity

	SLS (kPa)*	ULS (kPa)
House - Footings placed on engineered fill	80	120

5.1.1 Foundations General Notes

All footings should conform with the minimum requirements of the latest version of the Ontario Building Code (OBC, Part 9 and other relevant sections as required).

The above allowable bearing capacity reflects an increase of the local grades by an average of up to 2.07 metres for the residential properties based on the the current lot grading plan provided by S. A. Kirchhefer Limited. The above noted bearing capacities are based on a minimum strip footing width of 610 mm wide and spread footings in the range of 1.00 metre square. Foundations installed in accordance with the above recommendations (*Geotechnical reaction at SLS for of 25 mm of settlement) would be expected to experience differential settlements in the order of 19 mm (3/4 inch).

If the grading plan and required surcharge loads for this development differs, then Terraprobe must review the changes to assess the suitability and requirements for the engineered fill as recommended in this report as it relates to the bearing capacity.

In all cases, foundations should be placed on an engineered fill placed over the undisturbed pre-loaded subgrade soils which have been cleaned of all deleterious materials such as topsoil, loosened materials, and debris as well as any standing water prior to pouring concrete. Rainwater or seepage entering the excavations should be pumped away (not allowed to pond), and any disturbed material should be removed from the base of the excavation.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided.

5.1.2 Basement Concrete Slab-On-Grade

Concrete floor slabs should be placed on a minimum of 150 mm of Granular A (OPSS.MUNI 1010) compacted to a minimum 100% Standard Proctor Maximum Dry Density (SPMDD). The granular base should be placed on the engineered fill pad.

5.1.3 Basement Drainage

To assist in maintaining the building dry from surface water seepage, it is recommended that exterior grades around the structure be sloped away at a 2% gradient or more, for a distance of at least 2.0 metres. Roof drains should discharge a minimum of 1.50 metres away from the structure to a drainage swale or appropriate drainage outlet.

Since the houses will have basements, exterior perimeter foundation drains are required. The foundation drains should consist of a minimum 100 mm diameter fabric wrapped perforated pipe surrounded by a 19 mm diameter clearstone gravel (OPSS.MUNI 1004) with a minimum cover of 150 mm (OBC section 9.14.3, Division B, pg B9-60). The perimeter weeping tile would drain into a sump pit located in the basement area. The perimeter foundation drains should discharge towards the rear section of the house to a swale or suitable drainage outlet. The perimeter foundation drain installation and outlet considerations must conform to the Ontario Building Code and plumbing code requirements.

The exterior foundation backfill shall extend a minimum lateral distance of 600 mm out from the foundation wall and shall consist of a free-draining granular material, such as a Granular B Type I (OPSS.MUNI 1010) or suitable alternative drainage cellular media. All granular materials are to be placed in maximum 200 mm thick lifts and compacted to a minimum of 98% of it's SPMDD. Where the granular material is to be placed below any settlement sensitive structures, such as a sidewalks, driveways, etc., it is to be compacted to a minimum of 100% of it's SPMDD. 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. For the attached garage with no basement and a slab on grade, the backfill materials (placed inside and on the exterior of the foundation walls) should be placed so that the lateral capacity of the foundation wall is not exceeded. Ideally, the difference in elevation of the backfill material from one side of the wall to the other should not exceed 300 mm. All sub-surface walls with an occupied space (basement) should be damp proofed above the groundwater table.

5.2 Frost Protection

Based on regional frost depths (based on the freezing index) for the City of Greater Sudbury area, all exterior residential dwelling foundations in heated and unheated areas bearing on subgrade soils that are frost susceptible or engineered fills placed on subgrade soils that are frost susceptible must be provided with a minimum of 1.80 metres of earth cover for frost protection or alternative equivalent insulation.

For this project, where the frost protection cover is not met, it shall be provided in the form of a combination of earth cover and rigid insulation boards (or equivalent spray foam). For conventional foundation walls and strip footings, Terraprobe recommends the following insulation detail to provide the required frost protection for the single residential dwelling foundation system:

A- The insulation should consist of a minimum 50 mm thick rigid board insulation sheets (R10 minimum);

B- The horizontal insulation must consist of 50 mm rigid board insulation sheets placed on top of the footing base;

C- The horizontal insulation sheets must extend out horizontally beyond the foundation wall a minimum of 1.22 metres;

D- The horizontal insulation sheets placed on top of the footings should be sloped away from the building at a minimum gradient of 2% to promote positive drainage;

E- The vertical insulation must consist of 50 mm thick rigid board insulation sheets placed vertically along the foundation wall placed under the finish grade (covered a minimum of 25 mm) and rest on top of the horizontal sheet. Vertical insulation is not required for poured foundation walls based on the smooth surfaces.

F- As an alternative to placing vertical rigid board insulation sheets, the client may elect to install a waterproof membrane to prevent the potential of ad-freezing of concrete masonry blocks.

These insulation recommendations are site specific and are not be used for any other structures except for the site in which it was intended. All insulation is to be installed as outlined above as well as in accordance with the manufactures recommendations.

5.3 Engineered Fill - Foundation

The borehole data indicates that the upper undisturbed subgrade soils predominantly consist of Clayey SILT trace sand. The subgrade soils are sensitive to change in moisture content and can become soft if the soils are subject to additional water or precipitation. As well, they could be easily disturbed if travelled on during construction. As such, it is recommended that a the engineered fill pad be placed over the undisturbed subgrade soils immediately after verification of the soil capability by Terraprobe.

Once the subgrade has been exposed, it is recommended that it be proof rolled and inspected for obvious soft or loose, unstable areas. The excavated foundation base must be evaluated by a qualified Terraprobe geotechnical engineer or technician trained in this type of inspection to ensure that the founding subgrade soils exposed at the excavation base is consistent with the design bearing pressure intended by the geotechnical engineer. Should unstable areas be found, Terraprobe can provide appropriate advice for addressing local weak areas at that time, such as re-compaction and/or sub-excavation.

The engineered fill that is required for this project shall consist of a Granular B Type II (OPSS.MUNI 1010). The Granular B Type II material is to be placed in maximum 200 mm thick lifts and compacted to 100% of its Standard Proctor Maximum Dry Density (SPMDD). The engineered fill is to extend horizontally a minimum of 1.50 metres beyond the footing edges and slope down at 1 horizontal to 1 vertical. The engineered fill would be placed over the approved exposed undisturbed subgrade soils subgrade. Depending on the state of the exposed subgrade soils, a non-woven geotextile (minimum tear resistance of 267 N) or a combination of a geogrid and geotextile (stress-strain ration in the range of 2%) may be required as a separation medium. This would be reviewed on site by a by a Terraprobe geotechnical engineer.

Full time supervision of the placement and compaction of the engineered fill is required to for each lift of engineered fill. For a Granular B Type II, witnessing the proof rolling on a full time basis would be utilized to verify and approve the compactive effort.

5.4 Pre-loading

The placement of the engineered fill (including the roadway areas) and conventional spread footings over the undisturbed compressible subgrade soils (silt and clay) will cause settlements of the underlying soft soils. An undisturbed soil sample retrieved from a shelby tube at a depth of 4.57to 5.03 metres (BH 2, SA-6) below the existing grade had the following initial characteristics:

Water Content	$w =$	40.1%
Porosity	$n =$	51.8 %
Initial Void Ratio	$e_o =$	1.105
Specific Gravity	$G_s =$	2.701

The results of the consolidation conducted on a sample 19.04 mm high and 63.44 mm in diameter were plotted in a void ratio vs log pressure curve (see Appendix B). Based on the data, the coefficient of consolidation and coefficient of volume compressibility were derived at various pressures.

Settlements and time frames were calculated based on the following assumptions:

1. The compressible soil layers (silt and clay) are at least 9.00 metres thick
2. The pre-consolidated pressure of the compressible layers is 90 kPa
3. Based on the current lot grading plan, an average 2.50 metre ± surcharge (grade raise, roadway and lots)) would consist of a Granular B Type II (OPSS.MUNI 1010) fill with a unit weight of 23 kN/m^3.
4. The conventional strip footings placed on the engineered fill will be restricted to a net allowable bearing of 80 kPa.

Prior to the placement of the pre-load, settlement plates would be established to monitor the settlements of the subgrade. The monitoring would consist of installing settlement plates (pipes attached to settlement plates placed on the existing ground surface and buried within the pre-load fill, see Appendix D) which would be surveyed on a regular basis to establish the elevation of the pre-load. The frequency of the measurements would be set initially on a week basis for the first month and bi-monthly thereafter.

- Prior to installing the settlement plates, the excavated floor would be surveyed to establish the bench mark elevations.

- The settlement plates would be placed on the undisturbed subgrade soils and the fill placed around then ensuring they would not be damaged.

For this project, we would recommend a minimum of four (4) settlement plates as follows (see Figure 3):

- one settlement plate on Lot 1
- one settlement plate between Lot 4 and 5
- one settlement plate between Lot 2 and 3
- one settlement plate in the cul-de-sac

Once the settlements have reached the calculated values, the pre-load from the site would be removed and construction of the building could be started.

The following matrix presents the recommended pre-load heights to achieve the soil improvement based on the laboratory data and expected settlements.

Pre-load Material	Density kN/m^3	Expected Settlement (mm)	Case	Pre-load Height (m)	Time Frame Months
Granular B Type II	23	160	single drain	3.50	18

The pre-load fill footprint should cover the entire subdivision plus an additional 1.50 metre beyond the building footprint on the outer limits and extend outward within an area defined by a 1 to 2 line downward from the top of the pre-load fill.

The pre-load height includes a permanent load (surcharge) of approximately 2.50 metres of fill (removal of upper soil stratum to placed 600 mm of Granular B Type II + grade increase). In order to reduce the possibility of a localised bearing failure, the total pre-load height should be restricted to 4.0 metres or less. For the single drain case and a pre-load height of 3.50 metres, the anticipated consolidation of the underlying soils would be in the range of 18 months.

It is noted that theses settlement calculations and expected time frames are estimate based on limited soil information gathered during this soil investigation. The expected settlements time frame may be greater than calculated. The monitoring of the settlement will indicate if adjustment are required to the pre-load to achieve the expected settlements as time passes by.

It is also noted that the entire subdivision development would need to be pre-loaded. Therefore, the client is advised, that any future developments should be included in order to minimize the risk of affecting the first phase of the development.

In addition, we would recommend that a preconstruction survey of all neighbouring properties should be undertaken prior to the placement of any pre-loading. The preconstruction survey will serve to protect the client from building damage claims unrelated to the construction activities in the development of this subdivision. Terraprobe can provide the services for this preconstruction survey.

5.5 General Backfill

Any topsoil or fill soil materials encountered at the site should not be reused as backfill in settlement sensitive areas, such as beneath the floor slab areas, pavements and trench backfill areas These materials may be stockpiled and reused for landscaping purposes or removed from the site if it is environmentally feasible (soils are not contaminated). Any fill materials that is required to be discarded from the site will need to follow the latest version of the MECP (Ministry of Environment, Conservation and Parks) excess soil regulation O.Reg. 406/19: On-Site and Excess Soil Management and OPSS.MUNI 180.

If contaminated soils are encountered during excavations and construction, Terraprobe can provide appropriate measures to disposed of such soils as per the latest MOE "Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the *Environmental Protection Act*" (currently dated April 15, 2011). This document sets out the prescribed contaminants and the applicable site condition standards for those contaminants for the purposes of Part XV.1 of the *Environmental Protection Act*.

All backfill materials should consist of free draining material such as a Granular B Type II (OPSS.MUNI 1010) which can be readily compacted. In settlement sensitive areas, such as beneath pavements and trenches, the backfill should be placed in maximum lifts 300 mm or less and compacted to a minimum of 100% of its SPMDD. It is recommended that inspection and testing be carried out during construction to confirm the backfill quality, thickness and to ensure adequate compaction.

Should construction be conducted during the winter season, it is imperative to ensure that frozen material is not utilized as backfill.

5.6 Pipe Bedding

The buried services should be placed on conventional Class 'B' granular bedding as per the latest version of the City of Greater Sudbury GSSD-1227.010 specifications for sewer pipes & water mains for good ground conditions. The granular bedding could be placed over the undisturbed soil subgrade, an engineered fill, rock shatter or an exposed bedrock subgrade In the case of a soil trench, where disturbance of the trench base has occurred, such as due to groundwater seepage, or construction traffic, the disturbed soils should be sub-excavated and replaced with suitably compacted granular fill.

5.7 Trench Backfill

Trench backfill above the springline of the pipe should conform to the latest version of the City of Greater Sudbury GSSD-1227.010 specifications. Backfilling of narrow trenches can be accomplished by reusing the excavated soils (provided they are not too wet) above the springline of the pipe to the underside of the roadway subbase materials provided the moisture content is maintained within 2% of optimum moisture content.

If the subgrade soils prove difficulty to compact with vibratory compaction equipment, it is recommended that a free draining material such as Granular B Type I or Type II (OPSS.MUNI 1010) be used. All fill should be placed in 150 mm lifts and compacted to a minimum of 95% of its Standard Proctor Maximum Dry Density (SPMDD).

5.8 2015 National Building Code Seismic Hazard Calculation

Under Ontario Regulation 88/19, the ministry amended Ontario's Building Code (O. Reg 332/12) to further harmonize Ontario's Building Code with the 2015 National Codes. These changes will help reduce red tape for businesses and remove barriers to interprovincial trade throughout the country. The amendments are based on code change proposals the ministry consulted in 2016 and 2017. The majority of the amendments came into effect on January 1, 2020, which includes structural sufficiency of buildings to withstand external forces and improve resilience.

Seismic hazard is defined in the 2012 Ontario Building Code (OBC 2012) by uniform hazard spectra (UHS) at spectral coordinates of 0.2 s, 0.5 s, 1.0 s and 2.0 s and a probability of accedence of 2% in 50 years. The OBC method uses a site classification system defined by the average soil/bedrock properties (e.g. shear wave velocity (v_s), Standard Penetration Test (SPT) resistance, and undrained shear strength (s_u)) in the top 30 meters of the site stratigraphy below the foundation level, as set out in Table 4.1.8.4A of the Ontario Building Code (2012). There are 6 site classes from A to F, decreasing in ground stiffness from A, hard rock, to E, soft soil; with site class F used to denote problematic soils (e.g. sites underlain by thick peat deposits and/or liquefiable soils). The site class is then used to obtain peak ground acceleration (PGA), peak ground velocity (PGV) site coefficients Fa and Fv, respectively, used to modify the UHS to account for the effects of site-specific soil conditions.

Based on the above noted information, it is recommended that the site designation for seismic analysis be 'Site Class E', as per Table 4.1.8.4.A of the Ontario Building Code (2012). Consideration may be given to conducting a site specific Multichannel Analysis of Surface Waves (MASW) at this site to determine the average shear wave velocity in the top 30 metres of the site stratigraphy. An improved seismic site designation (Site Class A or B) may be possible.

The values of the site coefficient for design spectral acceleration at period T, F(T), and of similar coefficients F(PGA) and F(PGV) shall conform to Tables 4.1.8.4.B. to 4.1.8.4.I. using linear interpolation for intermediate values of PGA.

See the site specific 2015 National Building Code Seismic Hazard Calculation in Appendix C.

5.9 Pavement Design

5.9.1 Subgrade Preparation

The pavement subgrade for the parking lot is expected to consist of undisturbed subgrade soils materials consisting of Clayey SILTS. The proper base and subbase fill materials become very important in addressing the proper load distribution to provide a durable pavement structure. In particular, the silt content of the sub-grade material also plays a key role in the design of the pavement structure.

The laboratory gradations conducted on selected soil samples indicate that the undisturbed subgrade soil material contain high amount of silt (contents ranged between 51% to 73% in the upper 2.00 metres of soil) which are considered susceptible to frost action and will manifest itself by frost heaves and frost boils, inducing cracks in the asphalt surface. It is imperative that proper surface and subsurface drainage of the pavement structure is achieved with proper grades, catch basins and subdrains.

The undisturbed subgrade soils are sensitive to change in moisture content and can become soft if the soils are subject to additional water or precipitation. As well, they could be easily disturbed if travelled on during construction. As such, it is recommended that the engineered fill be placed immediately upon excavation to protect the integrity of the soil. Typically, the first layer of engineered fill will consist of a minimum of 300 mm of material to mitigate the disturbance of the subgrade soil prior by vibratory compaction**.**

For this project Terraprobe recommends removing the required subgrade soils to permit the construction of the recommended asphalt pavement structure. Prior to placing the granular subbase and base courses, the exposed subgrade must be inspected by Terraprobe to confirm the soil conditions encountered. Should unstable areas be found, Terraprobe can provide appropriate advice for addressing local weak areas at that time, such as re-compaction and/or sub-excavation and stabilization with a non-woven geotextile (minimum tear resistance of 267 N) and/or granular materials.

The most severe loading condition on the subgrade usually occurs during construction. As such, construction equipment should not travel over the subgrade until a minimum of 300 mm of engineered fill is placed.

5.9.2 Pavement Structure

The following are the minimum design requirements for flexible pavement in local residential roadways which can be used for this site based on a properly prepared subgrade:

Pavement Design Requirements
Clayey SILT Subgrade

Pavement Layer	Compaction Requirements	Truck and light vehicle Pavement Thickness Design
Surface Course Asphaltic Concrete HL-3 (OPSS.MUNI 1150)	as per OPSS 310 min 92.0 % MRD	50 mm
Base Course Asphaltic Concrete HL-8 (OPSS.MUNI 1150)	as per OPSS 310 min 92.0 % MRD	50 mm
Base Course: Granular A (OPSS.MUNI 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm
Subbase Course: Granular B Type II (OPSS.MUNI 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	300 mm
Subgrade Fill: Granular B Type II (OPSS.MUNI 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	as required
150 mm diameter fabric wrapped sub-drains		

The granular materials should be placed in lifts 150 mm thick or less and be compacted to a minimum of 100% of its SPMDD for granular base and granular sub-base. Asphalt materials should be rolled and compacted to OPSS.MUNI 310 specifications. The granular and asphalt pavement materials and their placement should conform to:

OPSS.MUNI Forms 310, 313, 501, 1003, 1010, 1101 and 1101 1150

In-situ density testing to monitor the effectiveness of the compaction equipment in achieving the required densities is required for certification.

5.9.3 Pavement Drainage

The above pavement thickness design is based on a drained pavement subgrade by drainage ditches.

Control of surface water is also a factor in achieving good pavement life. Grading adjacent pavement areas should be designed so that water is not allowed to pond adjacent to the outside edges of the pavement. The surface of the pavement should be free of depressions and sloped at a minimum grade of 2% to drain towards the drainage ditches.

6.0 STATEMENT OF LIMITATIONS AND RISK

6.1 Procedures

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The preliminary geotechnical engineering discussions and recommendations that have been presented are based on the factual data obtained from this investigation.

The exploratory borehole investigation was carried out by Landcore Drilling. The investigation work was monitored by a Terraprobe EIT whom logged the boreholes and examined the soil samples from the different soil stratums. Soil samples were sealed into plastic jars and transported to the Terraprobe soil laboratory for further testing and classification. There is consequently some interpolation of the borehole soil strata layering and indications of changes in stratigraphy as described are therefore approximate.

As noted, the undisturbed subgrade soils generally contain a significant amount of fine grained soils (silt and clay particles) and will become weakened when subject to traffic when wet. If site works are carried out during periods of wet weather, then it can be expected that the subgrade will be disturbed unless an adequate granular working surface is provided to protect the integrity of the subgrade soils. The disturbance caused by the traffic can result in the removal of disturbed soil and use of fill materials for site restoration or underfloor fill that is not intrinsic to the project requirements.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing program implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to existing between sampling points may differ from those that actually exist.

6.2 Excavations

Where workmen must enter excavations carried deeper than 1.20 metres, the trench excavations should be suitably sloped and/or braced in accordance with the latest version of the Occupational Health and Safety Act and Regulations for Construction Projects (Part III - Excavations, Section 226). Alternatively, the excavation walls may be supported by bracing or close shoring or a trench box.

The Occupational Health and Safety Act recognizes four (4) broad classifications of soils, which are summarized as follows:

TYPE 1 SOIL

a. is hard, very dense, and only able to be penetrated with difficulty by a small sharp object;
b. has a low natural moisture content and a high degree of internal strength;
c. has no signs of water seepage; and
d. can be excavated only by mechanical equipment.

TYPE 2 SOIL

a. is very stiff, dense and can be penetrated with moderate difficulty by a small sharp object;
b. has a low to medium natural moisture content and a medium degree of internal strength; and
c. has a damp appearance after it is excavated.

TYPE 3 SOIL

a. is stiff to firm and compact to loose in consistency or is previously excavated soil;
b. exhibits signs of surface cracking;
c. exhibits signs of water seepage;
d. if it is dry, may run easily into a well-defined conical pile; and
e. has a low degree of internal strength.

TYPE 4 SOIL

a. is soft to very soft and very loose in consistency, very sensitive and upon disturbance is significantly reduced in natural strength;
b. runs easily or flows, unless completely supported before excavating procedures;
c. has almost no internal strength
d. is wet or muddy; and
e. exerts substantial fluid pressure on its supporting system.

The undisturbed subgrade soil materials found on this site would be classified as Type 3 soils above the groundwater table and Type 4 soils below the groundwater table under these guidelines.

Based on Type 3 soils; the excavations will need to be sloped at a minimum gradient of 1 horizontal to 1 vertical from the bottom of the excavation.

Based on Type 4 soils; the excavations will need to be sloped at a minimum gradient of 3 horizontal to 1 vertical from the bottom of the excavation

6.3 Anticipated Groundwater Management

It is anticipated that any potential perched groundwater within the upper granular fill matrix should be able to be controlled with conventional sump pumps.

Generally, groundwater inflow within silt to sand sized particles can be controlled to a depth of up to approximately 600 mm below the water table by installing strategically placed sumps and pumping the collected water out of the excavations. Deeper excavations in this type of material will require more positive control, such as through well points and/or interlocking steel sheet piles. It is noted that excavations carried below the water table in cohesionless soil (silt, sand, sand and gravel) will experience loosening and sloughing of the base and sides, unless the ground water level is lowered first.

It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not undermine any adjacent structures. The contractor should submit their proposal to the prime consultant for review and approval prior to construction. A permit to take water may be required from the Ministry of the Environment. It is the responsibility of the contractor to make this application as required and any other applications from other Ministries or authorities as required (DFO, Conservation authorities, etc.).

All collected water is to discharge a sufficient distance away from the excavation to prevent re-entry. Sediment control measures, such as a silt fence should be installed at the discharge point of the dewatering system. The utmost care should be taken to avoid any potential adverse impacts on the environment.

It should be noted that the water table is expected to fluctuate seasonally with higher levels expected during the spring and fall seasons.

6.4 Horizontal Earth Pressure

If required, walls or bracings subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

$$\mathbf{P = K\,[\gamma\,(h-h_w) + \gamma' h_w + q] + \gamma_w h_w}$$

where:

- P = the horizontal pressure at depth, h (m)
- K = the earth pressure coefficient,
- h_w = the depth below the ground water level (m)
- γ = the bulk unit weight of soil, (kN/m^3)
- γ' = the submerged unit weight of the exterior soil, (γ - 9.8 kN/m^3)
- q = the complete surcharge loading (kPa)

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall, this equation can be simplified to:

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

This equation assumes that free-draining granular backfill is used and positive drainage is provided to ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure.

Resistance to sliding of earth retaining structures is developed by friction between the base of the footing and (N) and the frictional resistance of the soil (tan φ) expressed as R = N tan φ. This is an ultimate resistance value and does not contain a factor of safety.

Passive earth pressure resistance is generally not considered as a resisting force against sliding for conventional retaining structure design because a structure must deflect significantly to develop the full passive resistance.

The average values for use in the design of structure subjected to unbalanced earth pressures at this site are tabulated as follows:

Parameter	Definition	Units
φ	internal angle of friction	degrees
γ	bulk unit weight of soil	kN/ m^3
K_a	active earth pressure coefficient (Rankin)	dimensionless
K_o	at-rest earth pressure coefficient (Rankin)	dimensionless
K_p	passive earth pressure coefficient (Rankin)	dimensionless

The following soil material properties can be used for design purposes for this project:

	Silt and Clay	Gran A	Gran B Type I	Gran B Type II
Effective Angle of Internal Friction(Phi),degrees, unfactored	28	38	34	40
Cohesion (kPa)	5	0	0	0
Unit Weight (Gamma), kN/m^3	18	22	21	23
Active Earth Pressure, Coefficient, K_a	0.36	0.23	0.28	0.22
Passive Earth Pressure, Coefficient, K_p	2.77	4.2	3.54	4.6
At rest Earth Pressure, Coefficient, K_o	0.53	0.38	0.44	0.35

The values of the earth pressure coefficients noted above are for a horizontal grade behind the wall. The earth pressure coefficients for an inclined grade (retained soil) will vary based on its inclination.

Where permanent drainage for earth retaining walls is not install, hydrostatic pressure acting on the walls must be included in the above calculation; the unit weight of water, γ_w = 9.81 kN/m^3. For sloping backfill, the Canadian Highway Bridge Design Code, section C 6.9 should be consulted for the design recommendations.

The surcharge effect from compaction equipment during construction must be taken into account. Where lighter compaction equipment and smaller lifts are used the surcharge effect will be minimized. This should be reviewed in detail by a structural engineer. Permanent earth retaining wall designs are to be carried out in accordance with the latest edition of the Canadian Foundation Engineering Manual and/or the Canadian Bridge Design Code.

6.5 Quality Control

For this project, the foundations for the new building will be founded on engineered fill placed over exposed Clayey SILT subgrade. The foundation excavation and installation (engineered fill placement) must be monitored and evaluated by Terraprobe to ensure that the founding bearing area achieved is consistent with the design bearing capacity intended by the geotechnical engineer. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.2, Division B, of the latest version Ontario Building Code (2012). If Terraprobe is not retained to carry out foundation evaluations during construction, then Terraprobe accepts no responsibility for the performance or non-performance of the foundations, even if they are ostensibly constructed in accordance with the design recommendations contained in this report.

The requirements for fill placement on this project have been stipulated relative to Standard Proctor Maximum Dry Density as determined by ASTM D698. Terraprobe operates a CCIL (Canadian Council of Independent Laboratories) certified aggregates laboratory. In situ determinations of density during fill placement on site are recommended to demonstrate that the specified densities are achieved. Terraprobe is a CNSC licensed operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary, with our qualified technical staff. For a Granular B Type II or blast/crushed rock fill, witnessing the proof rolling on a full time basis would be utilised to verify and approve the compactive effort.

It has been assumed that concrete for the this structure will be specified in accordance with the requirements of CAN3 - CSA A23.1. Terraprobe maintains a CSA certified concrete laboratory and can provide concrete sampling and testing services for the project as necessary.

7.0 STATEMENT OF LIMITATIONS AND RISK

7.1 Procedures

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The geotechnical engineering discussions and recommendations that have been presented are based on the factual data obtained from this investigation.

The exploratory borehole investigation was carried out Landcore Drilling utilizing a track mounted drill rig, equipped with hollow stem augers and conventional soil sampling equipment. The operation was monitored by a Terraprobe Engineer in Training (EIT) who logged the borings and examined the samples as they were obtained. Selected soil samples were sealed into plastic jars and transported to the Terraprobe soil laboratory for further testing and classification. There is consequently some interpolation of the borehole soil strata layering and indications of changes in stratigraphy as described are therefore approximate.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing program implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to existing between sampling points may differ from those that actually exist.

It may not be possible to advance a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities.

7.2 Changes In Site And Scope

It must also be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. In particular, caution should be exercised in the consideration of contractual responsibilities as they relate to control of seepage, disturbance of soils, and frost protection. Groundwater conditions are particularly susceptible to change as a result of season variation and alterations in drainage conditions.

The engineering discussion and recommendations are based on the factual data obtained from this investigation completed at the site by Terraprobe and are intended for use by the owner and their retained designers in the design phase of the project. Since the project is still in the design stage, all aspects of the project relative to the subsurface conditions cannot be anticipated.

If there are changes to the project scope and development features the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructibility issues and quality control may not be relevant to the revised project or complete. Terraprobe must be retained to review the implications of changes with respect to the contents of this report and must be retained to review the design drawings and specifications prior to construction.

8.0 CLOSURE

This report was prepared for the express use of our client Michael McDowell Holdings Inc. and their retained design consultants. This report is copyright of Terraprobe and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe.

Michael McDowell Holdings Inc. and their retained design consultants are authorized users.

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

We trust that the foregoing is sufficient for your present requirements. If you have any questions or if we can be of further assistance, please do not hesitate to contact us.

Yours truly,
Terraprobe Inc.

LICENSED PROFESSIONAL ENGINEER
October 1, 2022
D.G. PAQUETTE
35380302
PROVINCE OF ONTARIO

Denis Paquette, P. Eng.
Principal, Sudbury Branch Manager

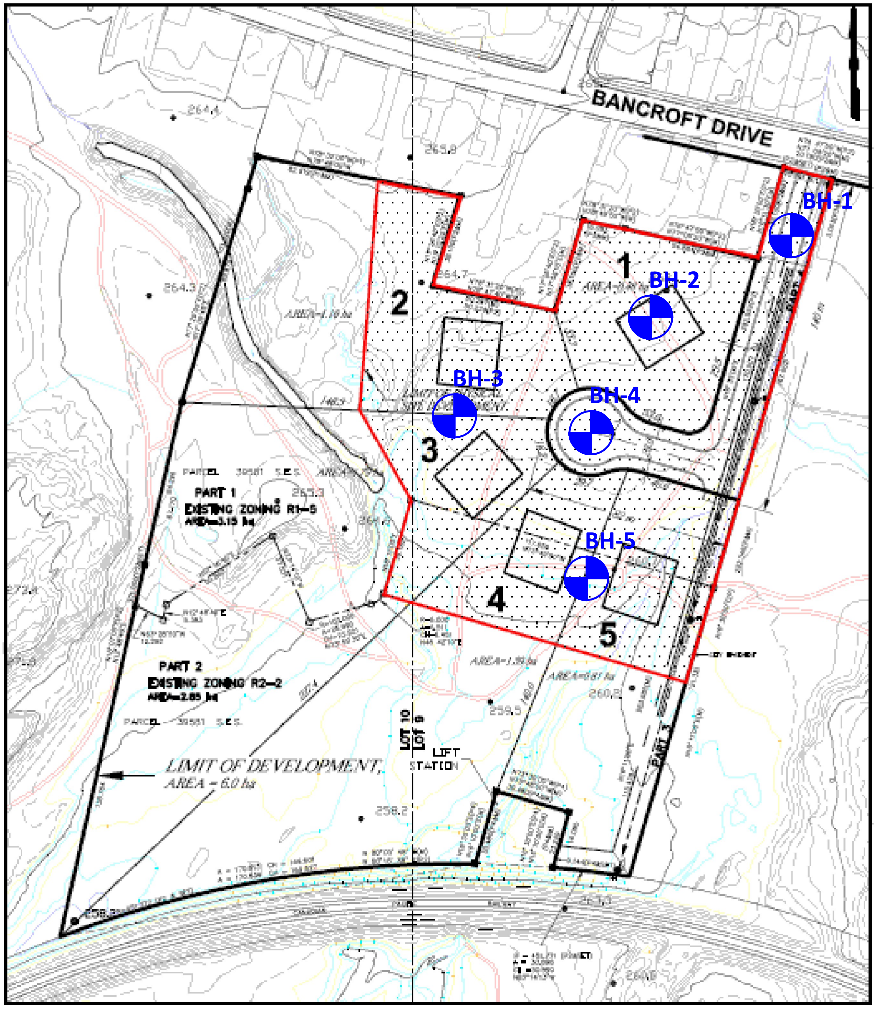
FIGURES

TERRAPROBE Inc.

Map provided By Google Earth

Drawing NTS

Terraprobe	Title/Name Location: Site Location Plan – Bancroft Drive Subdivision Bancroft Drive, Sudbury, Ontario	Date March 7, 2022	Project # 5–22–0030–01	Figure 1



SITE PLAN

N

LEGAL DESCRIPTION:
PLAN 53R-12893
PART OF LOTS 6 & 7
CONCESSION 5
PCL 29457 RP
TOWNSHIP OF LORNE
TOWN OF WALDEN

NOTES
1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE INDICATED.
2. DESIGN IS PRELIMINARY ONLY, AND NOT TO BE USED FOR CONSTRUCTION.

REVISIONS		
DATE	DETAILS	BY

REGISTERED PROFESSIONAL ENGINEER
S. A. KIRCHHEFER
PROVINCE OF ONTARIO

DATE: SEPTEMBER 15, 2021
DRAWN:
DESIGNED:
CHECKED:
ENGINEER:
APPROVED:

S. A. Kirchhefer Limited
Consulting Engineer and Planner
Sudbury, Ontario

PROPOSED
RESIDENTIAL DEVELOPMENT
MIKE McDOWELL HOLDINGS INC.
BANCROFT DRIVE, SUDBURY

SCALE: 1:2,000
CONTRACT NO.:
CAD/FILE
E:/DRAWS/MISC\2021
MIKE McDOWELL.DWG
FIGURE NO.:

Drawing provided by S.A. Kirchhefer Limited

Not to scale

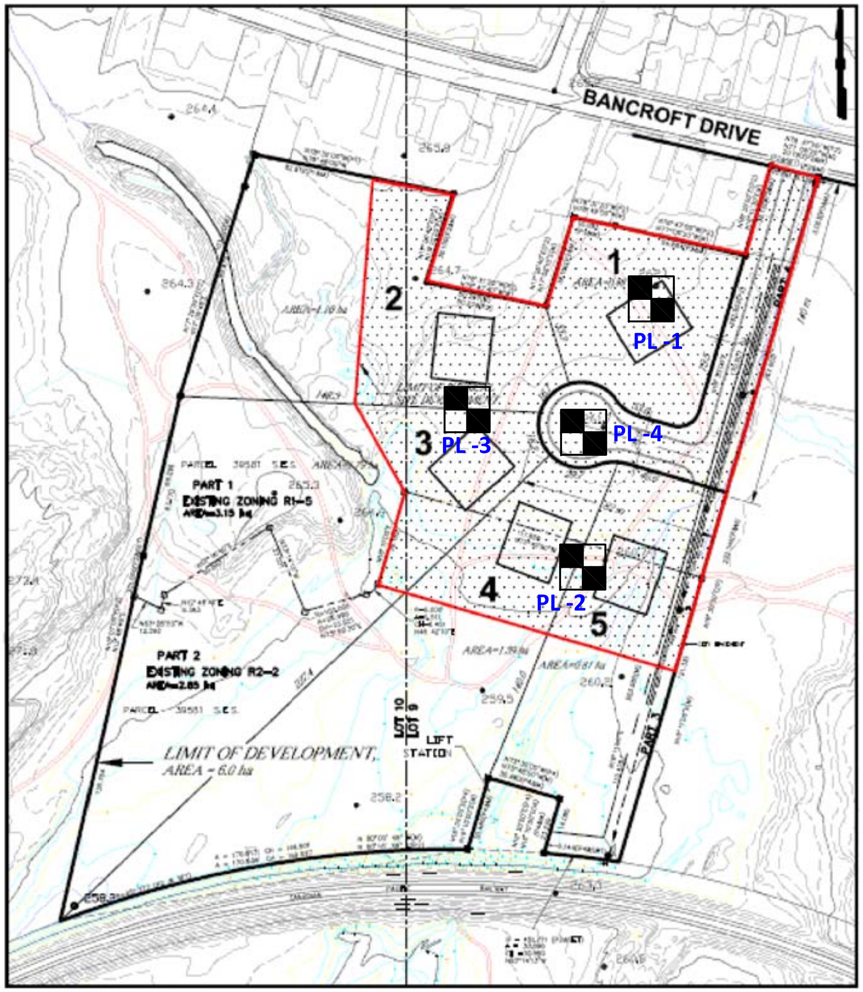
Terraprobe

Title/Name Location: Borehole Location plan – Bancroft Drive Subdivision
Bancroft Drive, Sudbury, Ontario

Date: March 7, 2022

Project No.: 5-22-0030-01

Figure: 2



SITE PLAN

LEGAL DESCRIPTION:
PLAN 53R-12893
PART OF LOTS 6 & 7
CONCESSION 5
PCL 29457 RP
TOWNSHIP OF LORNE
TOWN OF WALDEN

N

NOTES	REVISIONS							
1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE INDICATED. 2. DESIGN IS PRELIMINARY ONLY, AND NOT TO BE USED FOR CONSTRUCTION.	DATE	DETAILS	BY	S. A. KIRCHHEFER	DATE: SEPTEMBER 15, 2021	S. A. Kirchhefer Limited Consulting Engineer and Planner Sudbury, Ontario	PROPOSED RESIDENTIAL DEVELOPMENT	SCALE: 1:2,000
					DRAWN:		MIKE McDOWELL HOLDINGS INC.	CONTRACT NO.:
					DESIGNED:		BANCROFT DRIVE, SUDBURY	CAD FILE
					CHECKED:			FIGURE NO.:
					APPROVED:			

Drawing provided by S.A. Kirchhefer Limited

Not to scale

Terraprobe	Title/Name Location: Settlemnt Plate Location plan – Bancroft Drive Subdivision Bancroft Drive, Sudbury, Ontario	Date: March 7, 2022	Project No.: 5-22-0030-01	Figure: 3

APPENDIX A

Borehole Logs

Terraprobe Inc.

Terraprobe

ABBREVIATIONS AND TERMINOLOGY

SAMPLING METHODS

AS	auger sample
CORE	cored sample
DP	direct push
FV	field vane
GS	grab sample
SS	split spoon
ST	shelby tube
WS	wash sample

PENETRATION RESISTANCE

Standard Penetration Test (SPT) resistance ('N' values) is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a standard 50 mm (2 in.) diameter split spoon sampler for a distance of 0.3 m (12 in.).

Dynamic Cone Test (DCT) resistance is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a conical steel point of 50 mm (2 in.) diameter and with 60° sides on 'A' size drill rods for a distance of 0.3 m (12 in.)."

COHESIONLESS SOILS

Compactness	'N' value
very loose	< 4
loose	4 – 10
compact	10 – 30
dense	30 – 50
very dense	> 50

COHESIVE SOILS

Consistency	'N' value	Undrained Shear Strength (kPa)
very soft	< 2	< 12
soft	2 – 4	12 – 25
firm	4 – 8	25 – 50
stiff	8 – 15	50 – 100
very stiff	15 – 30	100 – 200
hard	> 30	> 200

COMPOSITION

Term (e.g)	% by weight
trace silt	< 10
some silt	10 – 20
silt*y*	20 – 35
sand *and* silt	> 35

TESTS AND SYMBOLS

MH	mechanical sieve and hydrometer analysis
w, w_c	water content
w_L, LL	liquid limit
w_P, PL	plastic limit
I_P, PI	plasticity index
k	coefficient of permeability
γ	soil unit weight, bulk
G_s	specific gravity
φ'	internal friction angle
c'	effective cohesion
c_u	undrained shear strength

	Unstabilized water level
	1st water level measurement
	2nd water level measurement
	Most recent water level measurement
3.0 +	Undrained shear strength from field vane (with sensitivity)
C_c	compression index
c_v	coefficient of consolidation
m_v	coefficient of compressibility
e	void ratio

FIELD MOISTURE DESCRIPTIONS

Damp	refers to a soil sample that does not exhibit any observable pore water from field/hand inspection.
Moist	refers to a soil sample that exhibits evidence of existing pore water (e.g. sample feels cool, cohesive soil is at plastic limit) but does not have visible pore water
Wet	refers to a soil sample that has visible pore water

Terraprobe

LOG OF BOREHOLE 1

PROJECT: Proposed New Subdivision
CLIENT: Michael McDowell Holdings Inc.
LOCATION: Bancroft Drive, Sudbury, ON

DATE: March 03, 2022
EQUIPMENT: CME 55 Track Mounted
ELEVATION DATUM: Geodetic FILE: 5-22-0030-01

DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUE	Water content / Shear strength	O.V.M. Reading (ppm)	STANDPIPE INSTALLATION OR REMARKS
262.52	100 mm of topsoil/organics						
	Frozen, Black, Moist	1	AS				
	Firm, Brown, Clayey SILT, trace sand	2	SS	7	wP16% wL26%		
	Soft	3	SS	4			
260.23 / 2.29	Firm, Grey to Brown, Wet, SILT & CLAY, trace sand	4	SS	6			Estimated Groundwater Level 2.29 metres
		5	SS	5			
257.95 / 4.57	Firm, Grey, Wet, Clayey SILT, trace sand	6	SS	1	41.10 / 12.84		
		7	SS	0	35.96 / 7.71		
255.82 / 6.70	End of Borehole						

PENETRATION RESISTANCE PLOT: 20 40 60 80
SHEAR STRENGTH kPa: FIELD VANE – + Q – ●; POCKET PEN – * U – ○; 20 40 60 80
PLASTIC LIMIT wP, NATURAL MOISTURE CONTENT w, LIQUID LIMIT wL
WATER CONTENT (%): 10 20 30

NOTES:

Terraprobe

LOG OF BOREHOLE 2

PROJECT: Proposed New Subdivision

CLIENT: Michael McDowell Holdings Inc.

LOCATION: Bancroft Drive, Sudbury, ON

DATE: March 03, 2022

EQUIPMENT: CME 55 Track Mounted

ELEVATION DATUM: Geodetic FILE: 5-22-0030-01

DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUE	STANDPIPE INSTALLATION OR REMARKS
262.26					
	Frozen Brown Moist Clayey SILT, trace sand Firm Wet	1	AS		
		2	SS	6	
260.74 1.52	Firm Brown Wet Firm Grey SILT & CLAY, trace sand	3	SS	5	Estimated Groundwater Level 1.52 metres
		4	SS	6	
		5	SS	6	
	Soft	6	ST		
256.76 5.50	Firm Grey Saturated Clayey SILT, trace sand	7	SS	4	
255.56 6.70	End of Borehole				

DEPTH SCALE IN METRES: 0 1 2 3 4 5 6 7 8 9

PENETRATION RESISTANCE PLOT: 20 40 60 80

SHEAR STRENGTH kPa: FIELD VANE – + Q – ● POCKET PEN – * U – O; 20 40 60 80

- 22.63 / 4.53 (at approx. 4 m)
- 30.82 / 7.71 (at approx. 7 m)

WATER CONTENT (%): PLASTIC LIMIT wP, NATURAL MOISTURE CONTENT w, LIQUID LIMIT wL; 10 20 30

- wP19%, wL31%, 34% (sample 7)

O.V.M. Reading (ppm)

NOTES:

SHEET 1 OF 1

LOG OF BOREHOLE 3

Terraprobe

PROJECT: Proposed New Subdivision
CLIENT: Michael McDowell Holdings Inc.
LOCATION: Bancroft Drive, Sudbury, ON

DATE: March 03, 2022
EQUIPMENT: CME 55 Track Mounted
ELEVATION DATUM: Geodetic FILE: 5-22-0030-01

DEPTH (m) 261.95	SOIL PROFILE – DESCRIPTION	STRATA PLOT	SAMPLES – NUMBER	TYPE	"N" VALUE	DEPTH SCALE IN METRES	PENETRATION RESISTANCE PLOT 20 40 60 80 / SHEAR STRENGTH kPa (FIELD VANE – + Q – ● POCKET PEN – * U – ○) 20 40 60 80	PLASTIC LIMIT wP / NATURAL MOISTURE CONTENT w / LIQUID LIMIT wL – WATER CONTENT (%) 10 20 30	O.V.M. Reading (ppm)	STANDPIPE INSTALLATION OR REMARKS
	Firm Brown to grey Moist Clayey SILT, trace sand		1	AS		0				
261.19 0.76	Stiff Brown to grey Wet Firm SILT & CLAY, trace sand Grey		2	SS	11	1				
			3	SS	6	2		wP 13% – wL 42%		Piezometer Groundwater Table 1.42 metres (ELV. 260.53 m) March 04, 2022
			4	SS	3	3	35.85 + 8.96			
258.14 3.81	Compact Brown Moist Clayey SILT, trace sand and gravel		5	SS	53	4				
257.68 4.27	Auger refusal on interperated bedrock or boulders End of Borehole					5 6 7 8 9				

NOTES: Split Spoon became clogged with frozen matterial on sample 2 resulting in little sample recovery from the spoon.

Terraprobe

LOG OF BOREHOLE 4

PROJECT: Proposed New Subdivision

CLIENT: Michael McDowell Holdings Inc.

LOCATION: Bancroft Drive, Sudbury, ON

DATE: March 03, 2022

EQUIPMENT: CME 55 Track Mounted

ELEVATION DATUM: Geodetic FILE: 5-22-0030-01

DEPTH (m) 261.70	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUE	STANDPIPE INSTALLATION OR REMARKS
	Soft to firm Dark Brown Moist	1	AS		
	Firm Brown	2	SS	6	
	Clayey SILT, trace sand				
	Wet	3	SS	5	Estimated Groundwater Level 1.52 metres
259.41 2.29	Firm Brown Wet	4	SS	8	
	SILT & CLAY, trace sand	5	SS	6	
257.13 4.57	Firm Grey Moist Clayey SILT, trace sand	6	SS	14	
256.67 5.03	Split Spoon refusal on interperated bedrock or boulders End of Borehole				

DEPTH SCALE IN METRES: 0–9

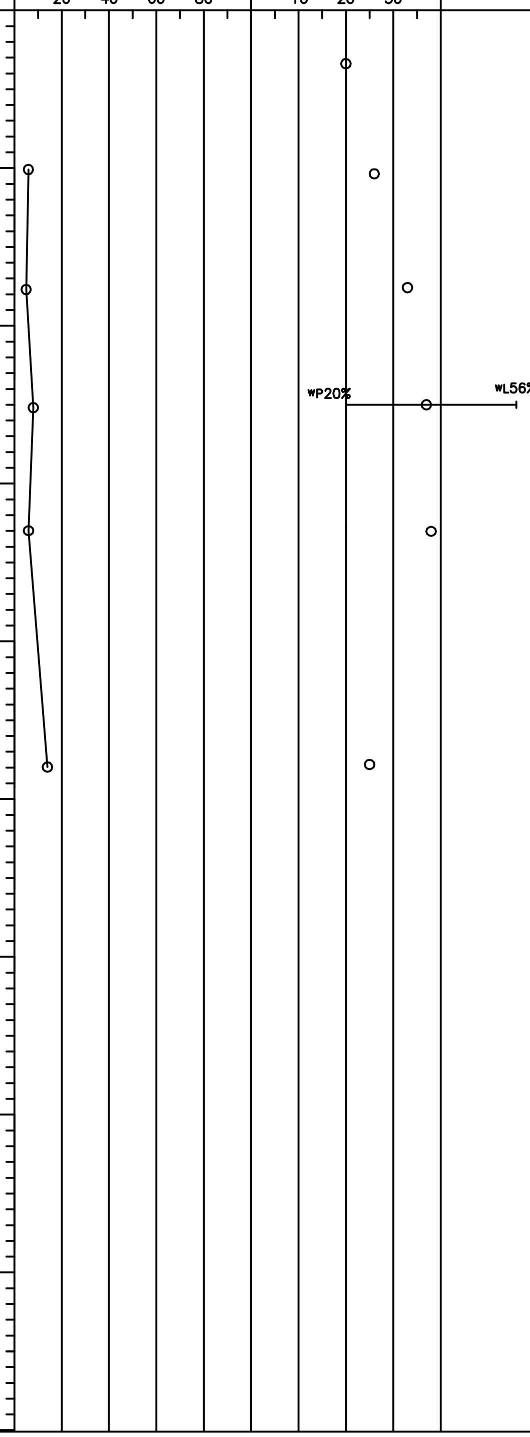
PENETRATION RESISTANCE PLOT 20 40 60 80

SHEAR STRENGTH kPa: FIELD VANE – + Q – ● ; POCKET PEN – * U – ○ ; 20 40 60 80

PLASTIC LIMIT wP – NATURAL MOISTURE CONTENT w – LIQUID LIMIT wL

WATER CONTENT (%) 10 20 30

O.V.M. Reading (ppm)



NOTES:

SHEET 1 OF 1

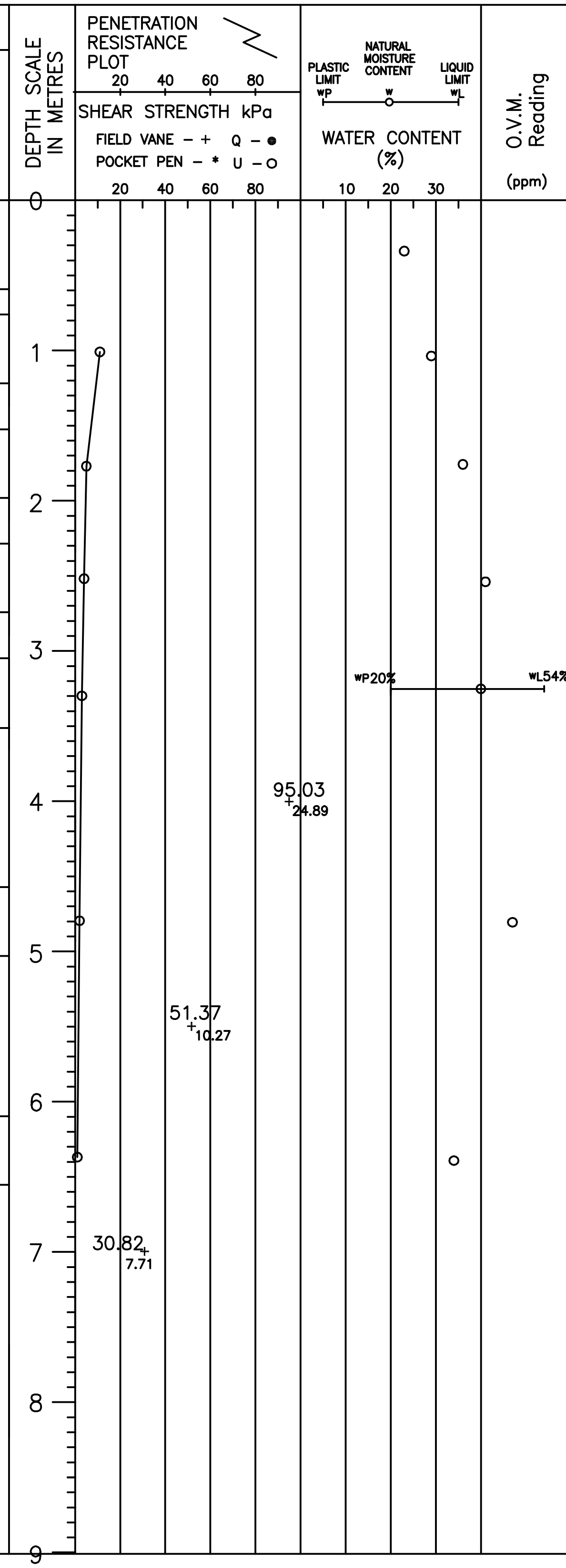
Terraprobe

LOG OF BOREHOLE 5

PROJECT: Proposed New Subdivision
CLIENT: Michael McDowell Holdings Inc.
LOCATION: Bancroft Drive, Sudbury, ON

DATE: March 04, 2022
EQUIPMENT: CME 55 Track Mounted
ELEVATION DATUM: Geodetic FILE: 5-22-0030-01

DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUE	STANDPIPE INSTALLATION OR REMARKS
260.81					
	Soft to Firm Brown Moist Clayey SILT, trace sand Stiff Brown to Grey Wet	1	AS		Estimated Groundwater Level 0.76 metres
		2	SS	11	
259.29 1.52	Firm Brown to Grey Wet SILT & CLAY, trace sand Stiff	3	SS	5	
		4	SS	4	
		5	SS	3	
256.24 4.57	Stiff Grey Wet Clayey SILT, trace sand Firm	6	SS	2	
		7	SS	1	
254.11 6.70	End of Borehole				



NOTES:

SHEET 1 OF 1

APPENDIX B

Soil Laboratory Results

TERRAPROBE INC.

Terraprobe

WATER CONTENT TEST FORM

PROJECT:	Proposed New Subdivision	FILE NO.:	5-22-0030-21
LOCATION:	Bancroft Drive, Sudbury, Ontario	LAB NO.:	8018
CLIENT:	Micheal McDowell Holdings Ltd.	SAMPLE DATE:	Mar. 3 &4, 2022
		SAMPLE BY:	J.C.
		TEST DATE:	Mar. 7, 2022
		TESTED BY:	T.E

BOREHOLE NUMBER		1	1	1	1	1	1	1
SAMPLE NUMBER		1	2	3	4	5	6	7
DEPTH OF SAMPLE (m)		0.00 - 0.76	0.76 - 1.06	1.52 - 2.02	2.29 - 2.89	3.05 - 3.65	4.57 - 5.17	6.10 - 6.70
WT. OF WET SOIL + TARE (g)	A	120.81	1187.40	105.59	80.39	94.15	83.88	100.03
WT. OF DRY SOIL + TARE (g)	B	106.47	1107.40	91.26	68.29	76.80	67.93	76.77
WEIGHT OF TARE (g)	C	30.64	726.90	30.61	30.17	30.45	30.54	30.48
WATER CONTENT (%)	A-B/B-C*100	**19%**	**21%**	**24%**	**32%**	**37%**	**43%**	**50%**

BOREHOLE NUMBER		2	2	2	2	2	2	2
SAMPLE NUMBER		1	2	3	4	5	6	7
DEPTH OF SAMPLE (m)		0.00 - 0.76	0.76 - 1.26	1.52 - 2.02	2.29 - 2.89	3.05 - 3.65	4.57 - 5.03	6.10 - 6.70
WT. OF WET SOIL + TARE (g)	A	111.84	97.13	79.47	76.63	100.04	Shelby Tube	1199.20
WT. OF DRY SOIL + TARE (g)	B	97.45	78.68	67.95	63.54	78.89		1077.40
WEIGHT OF TARE (g)	C	30.67	29.91	30.68	30.44	30.36		727.80
WATER CONTENT (%)	A-B/B-C*100	**22%**	**38%**	**31%**	**40%**	**44%**		**35%**

BOREHOLE NUMBER		3	3	3	3	3
SAMPLE NUMBER		1	2	3	4	5
DEPTH OF SAMPLE (m)		0.00-0.76	0.76 - 0.96	1.52 - 2.02	2.29 - 2.89	3.81 - 4.41
WT. OF WET SOIL + TARE (g)	A	95.18	84.40	1201.50	113.08	158.90
WT. OF DRY SOIL + TARE (g)	B	82.69	65.47	1092.30	92.87	137.73
WEIGHT OF TARE (g)	C	30.12	30.03	790.70	30.27	30.31
WATER CONTENT (%)	A-B/B-C*100	**24%**	**53%**	**36%**	**32%**	**20%**

BOREHOLE NUMBER		4	4	4	4	4	4
SAMPLE NUMBER		1	2	3	4	5	6
DEPTH OF SAMPLE (m)		0.00-0.76	0.76 - 1.36	1.52 - 2.12	2.29 - 2.89	3.05 - 3.65	4.57 - 5.17
WT. OF WET SOIL + TARE (g)	A	96.93	103.39	90.52	1326.60	86.98	86.56
WT. OF DRY SOIL + TARE (g)	B	85.83	88.57	75.69	1180.40	71.31	75.34
WEIGHT OF TARE (g)	C	30.58	30.76	30.53	787.40	30.45	30.90
WATER CONTENT (%)	A-B/B-C*100	**20%**	**26%**	**33%**	**37%**	**38%**	**25%**

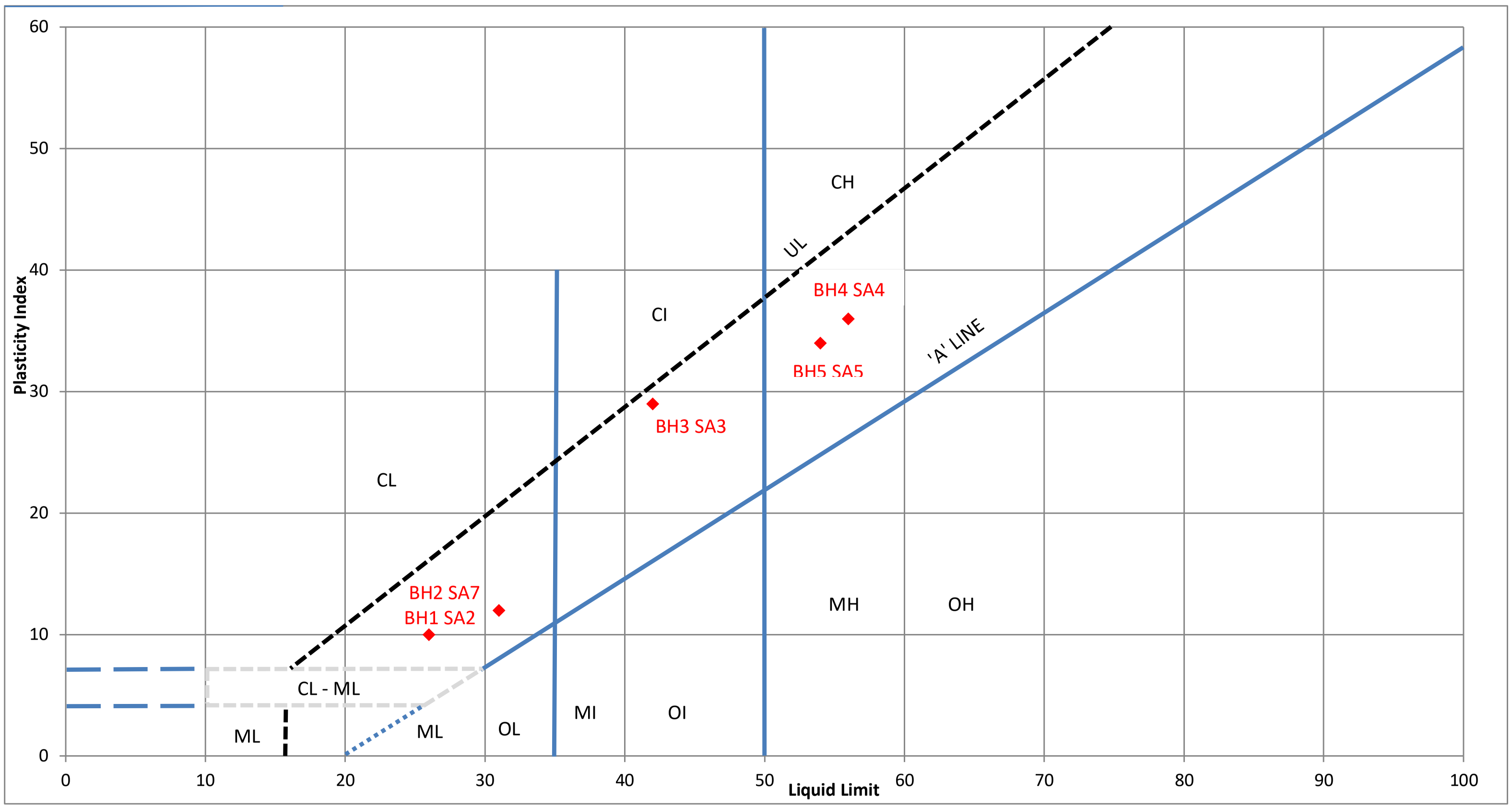
BOREHOLE NUMBER		5	5	5	5	5	5	5
SAMPLE NUMBER		1	2	3	4	5	6	7
DEPTH OF SAMPLE (m)		0.00-0.76	0.76 - 1.26	1.52 - 2.12	2.29 - 2.89	3.05 - 3.65	4.57 - 5.17	6.10 - 6.70
WT. OF WET SOIL + TARE (g)	A	100.26	88.05	83.96	98.74	1225.40	95.46	105.19
WT. OF DRY SOIL + TARE (g)	B	87.07	75.28	69.63	78.95	1082.20	74.78	86.43
WEIGHT OF TARE (g)	C	30.87	30.50	29.83	30.22	727.80	30.69	30.61
WATER CONTENT (%)	A-B/B-C*100	**23%**	**29%**	**36%**	**41%**	**40%**	**47%**	**34%**

COMMENT:

Terraprobe

Project: **Proposed New Subdivision**
Location: **Bancroft Drive, Sudbury, ON.**
Client: **Michael McDowell Holding Inc.**
Project No.: **5-22-0030-01**
Date **6-Sep-22**

BH #	1	2	3	4	5
Sample #	2	7	3	4	5
I_P (%)	10	12	29	36	34
W_L (%)	26	31	42	56	54
MC (%)	21	35	36	37	40
Soil	Clayey SILT	Clayey SILT	SILT & CLAY	SILT & CLAY	CLAY & SILT

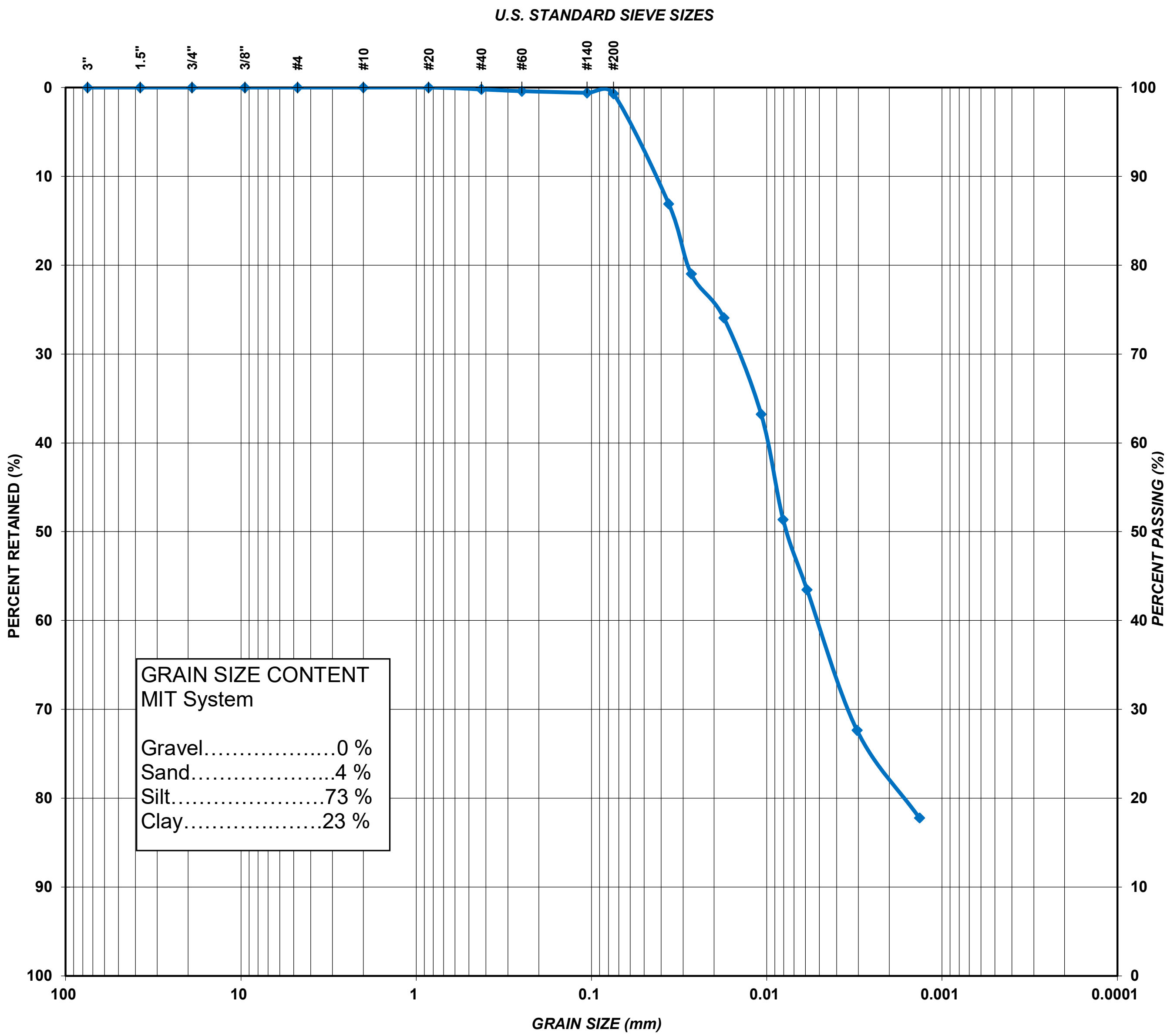


SIEVE AND HYDROMETER ANALYSIS TEST REPORT

Terraprobe

PROJECT:	**Proposed New Subdivision**	FILE NO.:	**5-22-0030-01**
LOCATION:	**Bancroft Drive, Sudbury, Ontario**	LAB NO.:	**8018**
CLIENT:	**Micheal McDowell Holdings Ltd.**	SAMPLE DATE:	**Mar. 3, 2022**
BOREHOLE NUMBER:	**1**	SAMPLED BY:	**J.C.**
SAMPLE NUMBER:	**2**		
SAMPLE DEPTH (m):	**0.76 - 1.06**		
SAMPLE DESCRIPTION:	**Clayey SILT, trace sand**		

GRAIN SIZE DISTRIBUTION



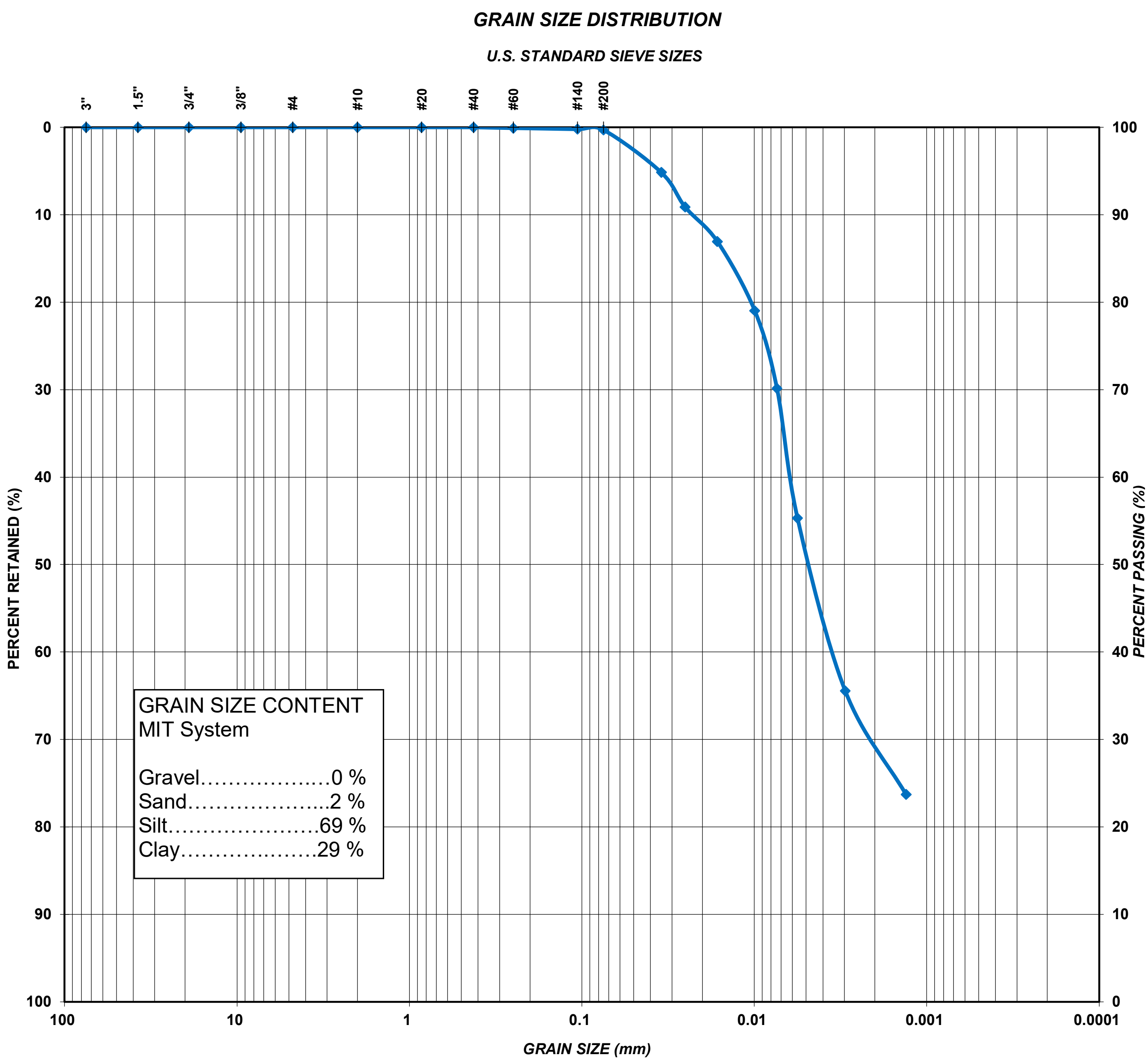
MIT SYSTEM	GRAVEL	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT	CLAY

UNIFIED SYSTEM	GRAVEL: COARSE	GRAVEL: FINE	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT AND CLAY

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

Terraprobe

PROJECT:	**Proposed New Subdivision**	FILE NO.:	**5-22-0030-01**
LOCATION:	**Bancroft Drive, Sudbury, Ontario**	LAB NO.:	**8018**
CLIENT:	**Micheal McDowell Holdings Ltd.**	SAMPLE DATE:	**Mar. 3, 2022**
BOREHOLE NUMBER:	**2**	SAMPLED BY:	**J.C.**
SAMPLE NUMBER:	**7**		
SAMPLE DEPTH (m):	**6.10 - 6.70**		
SAMPLE DESCRIPTION:	**Clayey SILT, trace sand**		



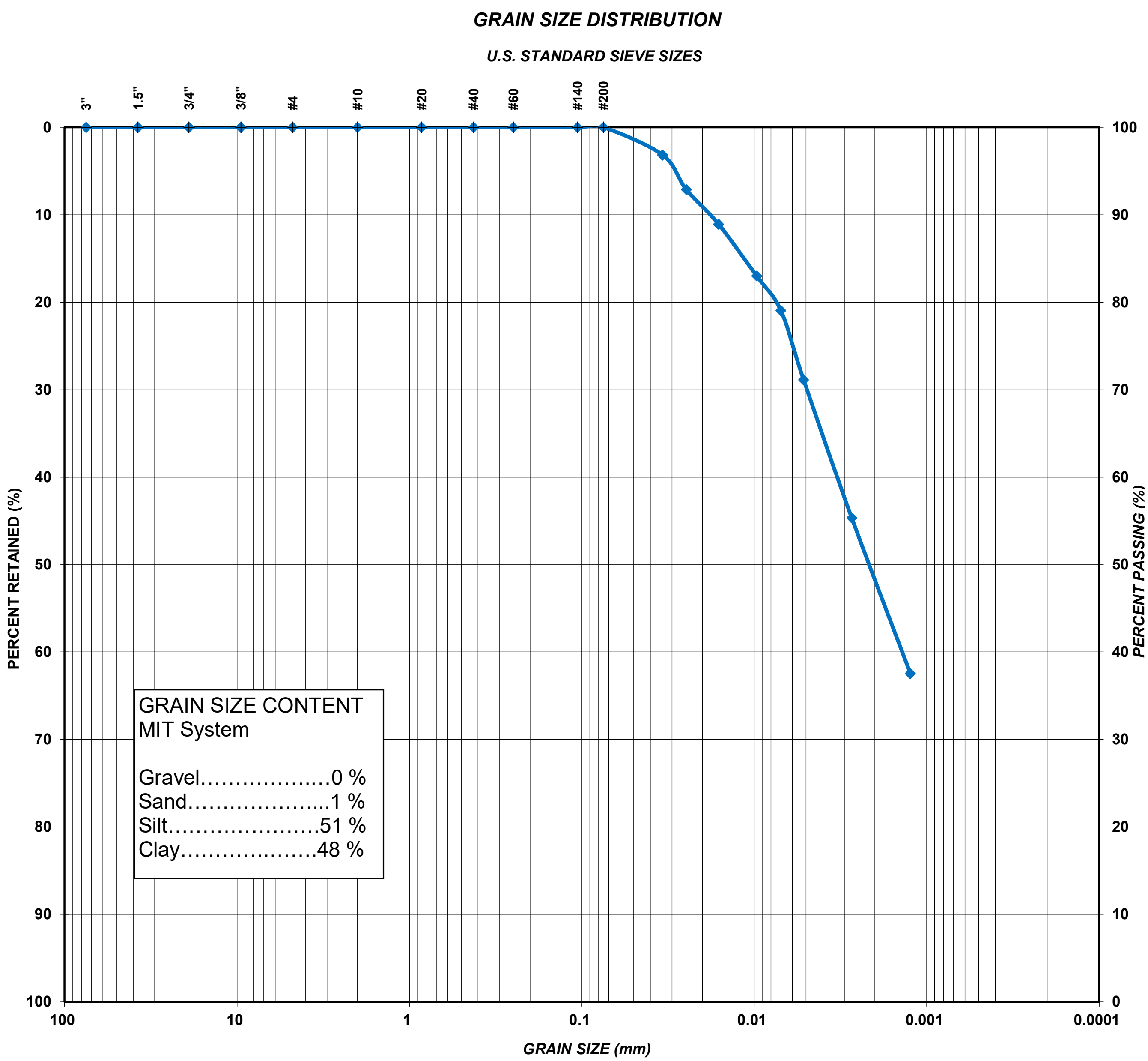
MIT SYSTEM	GRAVEL	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT	CLAY

UNIFIED SYSTEM	GRAVEL: COARSE	GRAVEL: FINE	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT AND CLAY

Terraprobe

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

PROJECT:	**Proposed New Subdivision**	FILE NO.:	**5-22-0030-01**
LOCATION:	**Bancroft Drive, Sudbury, Ontario**	LAB NO.:	**8018**
CLIENT:	**Micheal McDowell Holdings Ltd.**	SAMPLE DATE:	**Mar. 3, 2022**
BOREHOLE NUMBER:	**3**	SAMPLED BY:	**J.C.**
SAMPLE NUMBER:	**3**		
SAMPLE DEPTH (m):	**1.52 - 2.02**		
SAMPLE DESCRIPTION:	**SILT & CLAY, trace sand**		



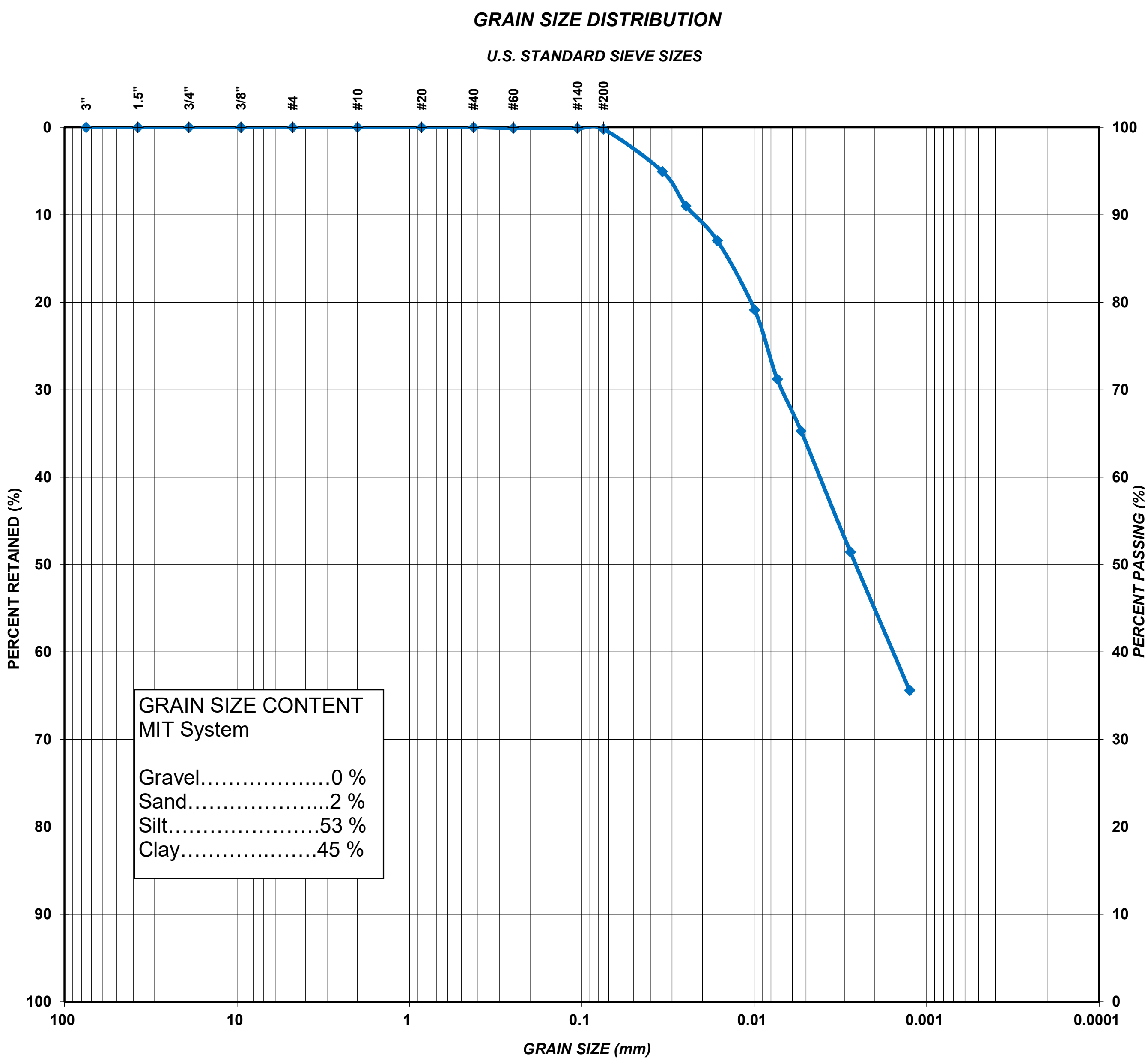
MIT SYSTEM	GRAVEL	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT	CLAY

UNIFIED SYSTEM	GRAVEL: COARSE	GRAVEL: FINE	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT AND CLAY

Terraprobe

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

PROJECT:	**Proposed New Subdivision**	FILE NO.:	**5-22-0030-01**
LOCATION:	**Bancroft Drive, Sudbury, Ontario**	LAB NO.:	**8018**
CLIENT:	**Micheal McDowell Holdings Ltd.**	SAMPLE DATE:	**Mar. 3, 2022**
BOREHOLE NUMBER:	**4**	SAMPLED BY:	**J.C.**
SAMPLE NUMBER:	**4**		
SAMPLE DEPTH (m):	**2.29 - 2.89**		
SAMPLE DESCRIPTION:	**SILT & CLAY, trace sand**		



MIT SYSTEM	GRAVEL	SAND (COARSE / MEDIUM / FINE)	SILT	CLAY

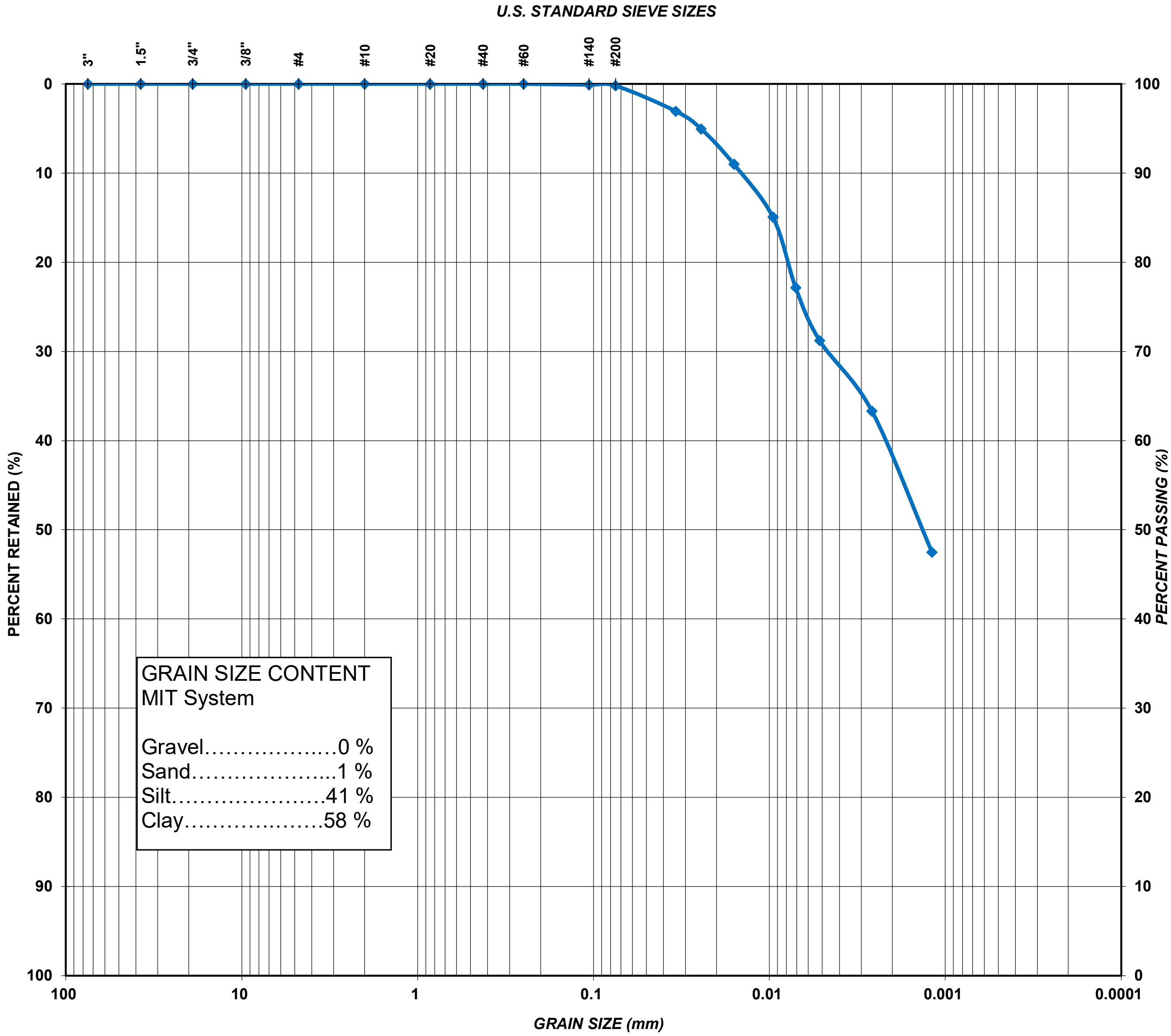
UNIFIED SYSTEM	GRAVEL (COARSE / FINE)	SAND (COARSE / MEDIUM / FINE)	SILT AND CLAY

SIEVE AND HYDROMETER ANALYSIS TEST REPORT

Terraprobe

PROJECT:	**Proposed New Subdivision**	FILE NO.:	**5-22-0030-01**
LOCATION:	**Bancroft Drive, Sudbury, Ontario**	LAB NO.:	**8018**
CLIENT:	**Micheal McDowell Holdings Ltd.**	SAMPLE DATE:	**Mar. 4, 2022**
BOREHOLE NUMBER:	**5**	SAMPLED BY:	**J.C.**
SAMPLE NUMBER:	**5**		
SAMPLE DEPTH (m):	**3.05 - 3.65**		
SAMPLE DESCRIPTION:	**CLAY & SILT, trace sand**		

GRAIN SIZE DISTRIBUTION

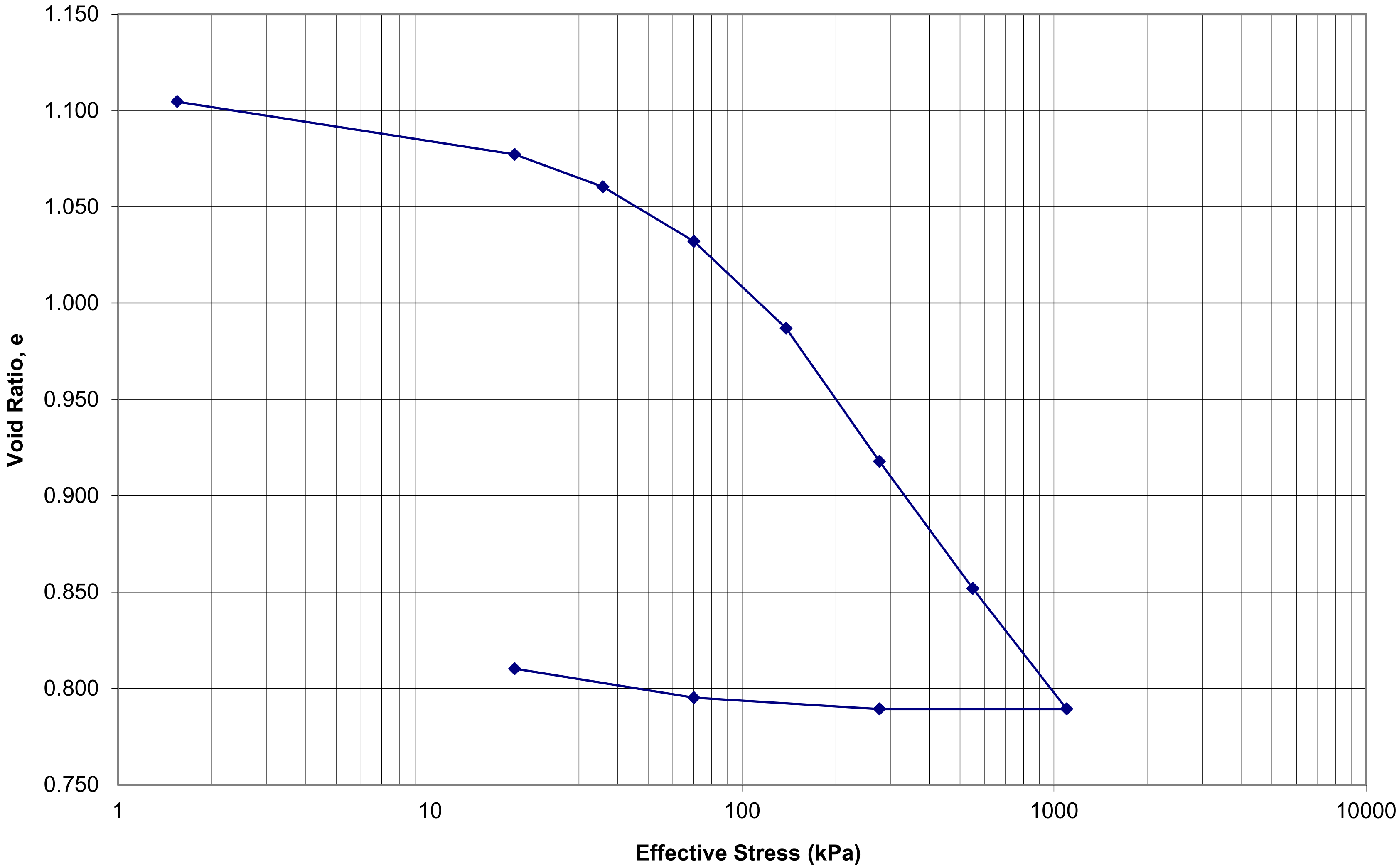


MIT SYSTEM	GRAVEL	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT	CLAY

UNIFIED SYSTEM	GRAVEL: COARSE	GRAVEL: FINE	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT AND CLAY

Terraprobe

Summary of Consolidation Test Results
5-22-0030-01



APPENDIX C

2015 National Building Code Seismic Hazard Calculation

Terraprobe Inc.

2015 National Building Code Seismic Hazard Calculation

INFORMATION: **Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836**
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 46.485N 80.912W **User File Reference:** Bancroft Drive Subdivision, Sudbury, On. 2022-02-12 13:09 UT

Requested by: Michael McDowell Holdings Inc., 5220030-01

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.085	0.051	0.032	0.010
Sa (0.1)	0.117	0.073	0.047	0.016
Sa (0.2)	0.113	0.073	0.048	0.017
Sa (0.3)	0.096	0.063	0.042	0.015
Sa (0.5)	0.078	0.051	0.034	0.012
Sa (1.0)	0.046	0.030	0.020	0.006
Sa (2.0)	0.024	0.015	0.009	0.002
Sa (5.0)	0.006	0.004	0.002	0.001
Sa (10.0)	0.003	0.002	0.001	0.000
PGA (g)	0.067	0.041	0.027	0.009
PGV (m/s)	0.063	0.039	0.024	0.007

Notes: Spectral (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s. Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

APPENDIX D

Settlement Plates Construction

Terraprobe Inc.

Designation: D 6598 – 07

Standard Guide for Installing and Operating Settlement Platforms for Monitoring Vertical Deformations[1]

This standard is issued under the fixed designation D 6598; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope*

1.1 This guide provides recommended designs and procedures for the fabrication, installation, operation, and reading of settlement platform to determine the magnitude and rate of foundation, fill settlements, or bothgenerally under a fill or embankment load. Two types of settlement platforms are described – those be monitored by elevation surveys from an external bench mark and those that include an internal reference system supported on unyielding soil or rock beneath the compressible layer(s) of interest.

1.2 *This guide does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this guide to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

1.3 *This guide offers an organized collection of information or a series of options and does not recommend a specific course of action. This document cannot replace education or experience and should be used in conjunction with professional judgement. Not all aspects of this guide may be applicable in all circumstances. This ASTM standard is not intended to represent or replace the standard of care by which the adequacy of a given professional service must be judged, nor should this document be applied without consideration of a project's many unique aspects. The word "standard" in the title of this document means only that the document has been approved through the ASTM consensus process.*

2. Referenced Documents

2.1 *ASTM Standards:* [2]

D 653 Terminology Relating to Soil, Rock, and Contained Fluids

D 3740 Practice for Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction

D 5092 Practice for Design and Installation of Ground Water Monitoring Wells

3. Terminology

3.1 *Definitions of Terms Specific to This Standard:*

3.1.1 *settlement platform*—a system consisting of a square base platform with an extendible riser pipe of known length which is used to monitor vertical deformations at the elevation of the base platform by survey measurements made of the top of the riser pipe.

3.1.2 *external and internal reference point system*—with an external system, the amount of settlement is determined by referencing the elevation of the settlement platform to an outside elevation benchmark; with an internal system, the amount of settlement is determined by measuring the relative displacement of two co-axial riser pipes moving relative to each other, the outer riser pipe being attached to the base platform and the inner riser pipe being fixed to an unyielding stratum.

3.1.3 *anchor*—an anchor system that provides an internal fixed reference point below the base of the settlement platform system.

3.1.4 *extendible riser*—a metal shaft or pipe which can be incrementally lengthened using sections of the same material and appropriate couplings as fill is placed and compacted to ensure that the top of the riser remains above the level of the surrounding ground surface. Depending on whether an external or internal reference point is being used, there may be one or two risers.

3.1.5 *isolation casing*—a casing of a larger diameter than the extendible risers is used in some installations to prevent down-drag of soil on the extendible riser that would otherwise be in contact with the soil from placing additional load on the platform and thereby leading to overestimates of deformations.

3.1.6 For definitions of other terms used in this guide see Terminology D 653.

4. Summary of Standard Guide

4.1 The standard guide presents recommended designs for settlement platforms along with procedures to install, operate and monitor them. The standard guide focuses on methods that permit (i) the effect of fill placement on underlying strata and

[1] This guide is under the jurisdiction of ASTM Committee D18 on Soil and Rock and is the direct responsibility of Subcommittee D18.23 on Field Instrumentation.

Current edition approved May 1, 2007. Published June 2007. Originally approved in 2000. Last previous edition approved in 2000 as D 6598–00.

[2] For referenced ASTM standards, visit the ASTM website, www.astm.org, or contact ASTM Customer Service at service@astm.org. For *Annual Book of ASTM Standards* volume information, refer to the standard's Document Summary page on the ASTM website.

***A Summary of Changes section appears at the end of this standard.**

(ii) the determination of the relative deformation within a fill. The guide addresses ways in which the instrument is protected from downdrag effects from the fill soils as well as measures to protect the instrument from damage by earth moving equipment. Standard survey procedures are used to determine the magnitude of deformations. Recommended procedures for reporting the details of an installation and the recorded deformations are presented.

5. Significance and Use

5.1 Earthen fills are often constructed as engineered structures, for example, dams, or to support engineered structures, for examples, roads or buildings. The weight of the fill may compress or deform the supporting soil or rock foundation resulting in settlement of the soil throughout the embankment. Temporary embankments or surcharge fills are constructed to increase the strength and/or reduce the compressibility of foundation soils prior to placement of the actual foundation or structure. The designers often monitor the settlement of the earth structure as a function of time to document the magnitude and rate of settlement, to evaluate the potential for future settlement, or to confirm the effectiveness of the surcharge and the schedule for its removal. The monitoring is performed using settlement platforms installed prior to or during the embankment construction. A platform provides an accessible survey point that settles with a selected soil horizon within or below the embankment. Careful design and installation of the settlement platform can isolate the survey point from extraneous sources of movement such as frost-induced heave, compression within the embankment, or volume changes caused by moisture gain or loss.

5.2 Various settlement platform designs have been developed by the agencies and practitioners that use them. This standard guide provides designs and procedures that can be referred to in design guidelines, specifications and reports.

5.3 This standard guide is not meant to restrict the use of other equally appropriate designs and procedures for the fabrication, installation, operation, and reading of settlement platforms to monitor deformations in earthen deposits during and after construction.

NOTE 1—Notwithstanding the statements on precision and bias contained in this guide, the precision of this guide is dependent on the competence of the personnel performing it and the suitability of the equipment and facilities used. Agencies that meet the criteria of Practice D 3740 are generally considered capable of competent and objective testing. Users of this guide are cautioned that compliance with Practice D 3740 does not itself ensure reliable testing. Reliable testing depends on many factors; Practice D 3740 provides a means of evaluating some of these factors.

6. Materials

6.1 A variety of materials are used in combination to provide a cost-effective, modular system. Given that the anticipated operational life of settlement platforms is typically relatively short, concerns about long term durability are generally negligible. Accordingly issues such as component weight, the ease with which the riser pipe can be extended and cost tend to dominate material selection decisions. The entire settlement platform system consists of 4 or 5 distinct components depending on the specific design. Typical alternative configurations are shown in Figs. 1-3. Key distinctions between these different configurations are summarized in Table 1. Additional considerations regarding materials for each of these components are provided below.

TABLE 1 Suitability and Use of Various Platform Configurations

Configuration	Fill Deformations	Foundation Deformations	External Reference	Internal Reference
Fig. 1	No[A]	Yes	Yes	No
Fig. 2	No[A]	Yes	No[B]	Yes
Fig. 3	No[A]	Yes	No[B]	Yes

[A] Fill settlements could be determined with this configuration if base platform placed at higher elevation.

[B] External reference (control) could be used with these configurations also.

6.2 *Base Platform*—a square base platform typically ranging between 0.3 to 1.0 m on side is placed at the elevation for which the vertical deformation is required. In some cases, a steel platform 5 to 15 mm thick is used. Alternatively, a platform 25 to 50 mm thick fabricated from plywood is sometimes used. This may be particularly desirable in short term applications where degradation of the wood is not a concern. Other materials such as concrete can be used for the base platform. In all cases, the thickness of the base platform should be selected giving consideration to the area of the platform to ensure that its rigidity is sufficient to avoid local bending.

6.3 *Riser Pipe*—a rigid metal shaft or an assembly of a rigid metal shaft and a rigid metal pipe, typically 25 to 50 mm in diameter, is used to reflect the vertical deformation of the platform at the ground surface. As layers of fill are placed, the riser pipes are extended by adding additional sections of pipe. Threaded couplings are typically used. These have the advantage that after the survey program is complete, some, if not all the riser pipe can be recovered before the installation is grouted to seal off any unwanted access for water to the subsurface. Use of PVC or other lightweight pipe materials is not recommended for reasons of survivability.

6.4 *Riser Pipe Isolation Casing*—an external pipe is sometimes used to isolate the riser pipe from the surrounding soil. This is done to prevent the effects of extraneous sources of movement such as frost-induced heave, skin-friction due to compression within the fill itself, or moisture induced volume changes. Given that this casing is only to isolate the riser pipe from these surrounding effects and does not constitute part of the deformation measuring system, PVC or other lightweight pipe materials are typically recommended. As with the riser pipe, the isolation casing can be extended as layers of fill are added. Isolation casing is typically only required if the fill or embankment height is greater than about 6 m or the plate is to be seated on a thin stiff layer overlying softer material where a punching failure might occur as a result of the down-drag load applied to the riser-pipe.

6.5 *Surface Protection Monument*—for settlement platforms that remain in place following completion of construction, installation of a surface protection monument to protect the riser pipe from tampering is advisable. Design of a protective casing system as described in Practice D 5092 is recommended.

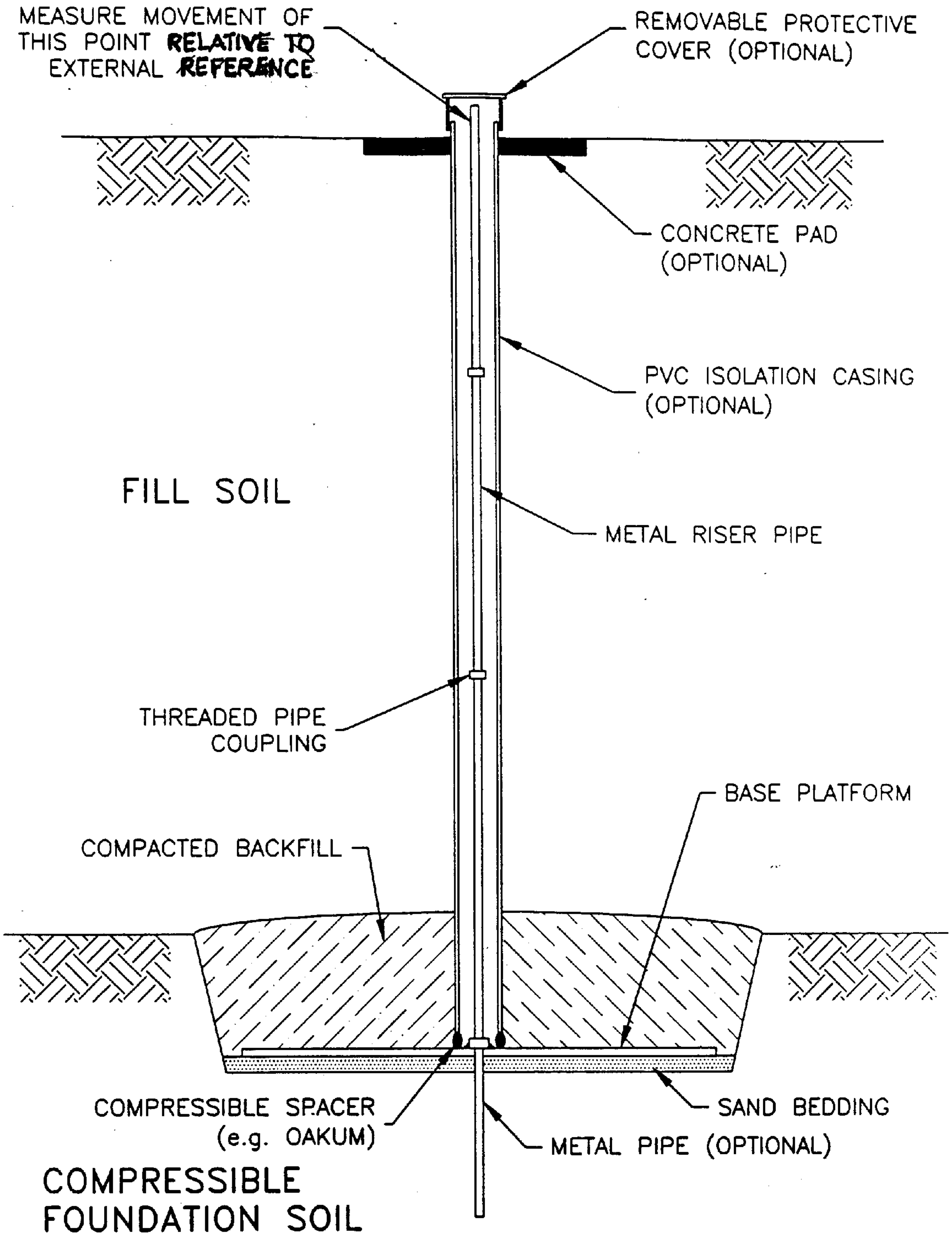


FIG. 1 Typical Installation for Externally Referenced Settlement Platform

6.6 For installations where an internal reference or benchmark supported on unyielding soil or rock beneath the compressible layer(s) of interest is used, rigid metal pipe similar to that described in 6.3 above is recommended. Alternatively, an anchor is used in conjunction with metal pipe to ensure a fixed base reference point. A typical anchor may consist of a number of metal prongs which are driven from an initially retracted position through slots in the conical drive point of an outer metal pipe using an inner metal riser pipe.

7. Procedure

7.1 A variety of instrument designs are possible depending on the specific application for which the settlement platform is to be used and whether an external or internal reference point

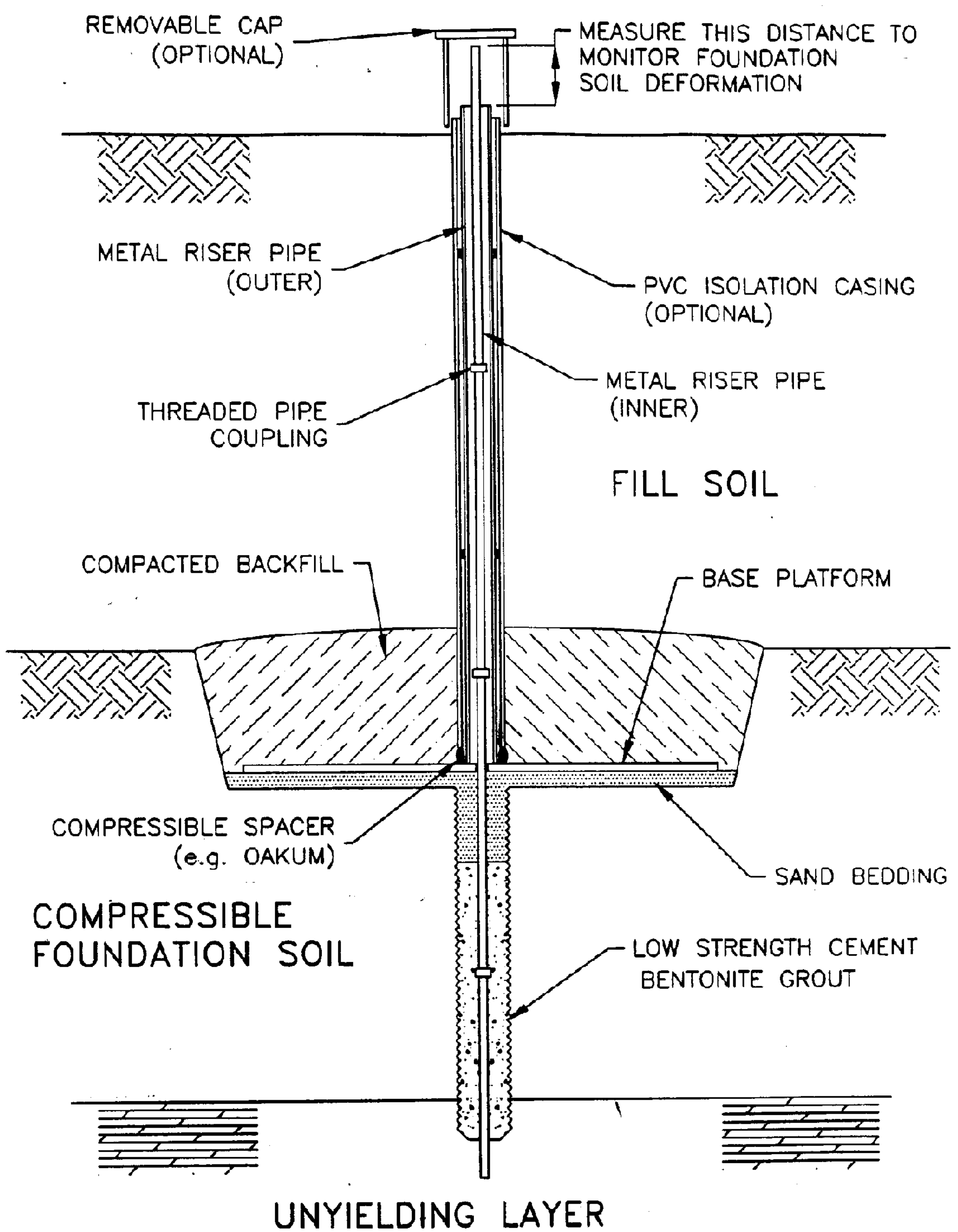


FIG. 2 Typical Installation for Internally Referenced Settlement Platform with Grouted Pipe

or bench mark is to be used. This standard test method describes a number of settlement platform systems intended to reflect these alternative configurations as well as a number of other features such as the use of riser pipe isolation casing.

7.2 Assuming that either the fill level is at the elevation that the base platform is to be installed or an excavation has been made to permit the level of interest to be accessed, installation of the base platform is preceded by the placement of a bedding layer. Typically, a free-draining clean sand is used (see Fig. 1 for example). If that an external reference point is used, the first section of riser pipe is connected to the base platform and the platform is positioned on the bedding sand and leveled manually. If that an internal reference system is used, the lower end of the shaft is first embedded in the unyielding soil stratum (see section 7.3). Backfill is then hand placed and compacted on top of the base platform to provide initial stability. The level of compaction required should be established for the specific project. Ensure that the riser pipe remains vertical during early filling. The zero reading or initial elevation of the top of the base plate is determined and recorded at this stage prior to the

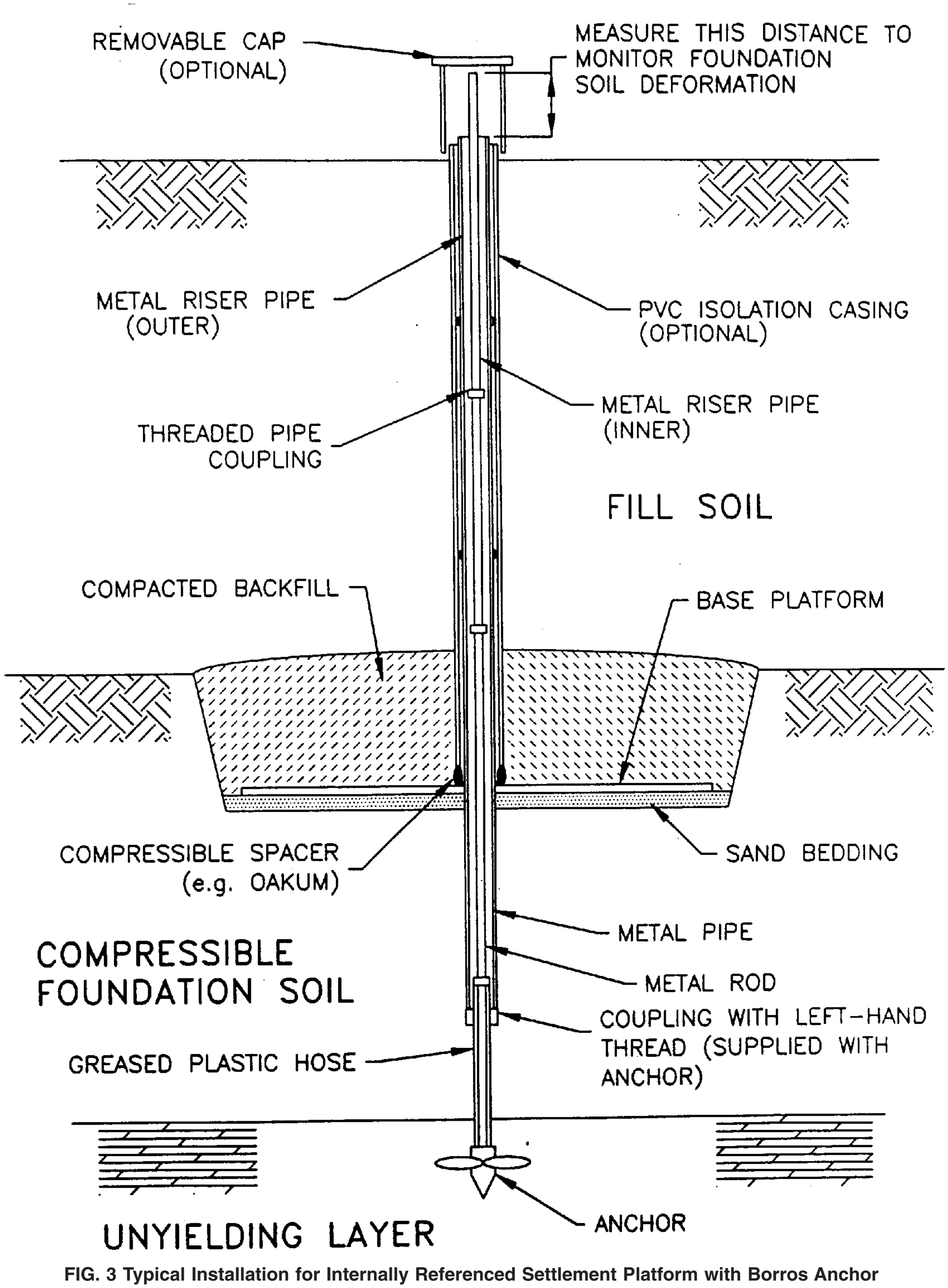


FIG. 3 Typical Installation for Internally Referenced Settlement Platform with Borros Anchor

placement of any fill layers with earth-moving equipment. Use of a measurement system capable of measuring deformations to an accuracy of 1% of the estimated total deformation is appropriate in most cases. For cases where total deformations are limited to the order of a few centimeters, accuracy is limited by the practicality of making the measurements. As filling progresses, additional sections of riser pipe are added to maintain the top of the riser pipe above the elevation of the fill. Sections of riser pipe between 1 and 2 m long are convenient for assembly as well as monitoring purposes. In cases where concerns exist about the influence of extraneous factors as noted in section 6.4 on the recorded deformations, riser pipe isolation casing are added as appropriate to ensure that the top of the isolation casing remains at least 25 mm below the top of the riser pipe and always above the top of the fill (see Fig. 1 for example).

7.3 If an internal reference point is used, then a system configuration such as shown in Figs. 2 and 3 should be used. A principal difference between these systems and the externally referenced system shown in Fig. 1 is the section of shaft or anchor that extends below the elevation of the base platform into the underlying unyielding layer. While externally referenced systems are used to indicate the relative vertical movement between the top of a riser pipe that is rigidly connected to the base platform and a remote survey point, internally referenced systems permit measurement of the relative vertical movement between the top of an outer riser pipe. The outer riser pipe is rigidly connected to the base platform and an inner riser pipe that is rigidly connected to an anchoring system founded in an underlying unyielding layer and passes through the center of the base platform. The anchor is installed by pre-drilling or hand-augering a hole into the competent stratum. The anchor is then placed in the hole and grouted in place using a cement-bentonite or similar material of sufficiently low strength to avoid supporting the platform. At contact between the grout and the platform avoid contact that influences the measured settlement. The unyielding layer should occur at shallow depths below the base plate to ensure economy of the internally referenced system relative to the cost of referencing surveys to an external point or bench mark. The choice between an externally or internally referenced system is based on comparative costs – the deeper the compressible layer, the more likely an externally referenced system is chosen.

7.4 Readings of the elevation of the top of the riser pipe are taken at time intervals frequent enough to permit critical deformations to be recorded. In addition, readings are taken immediately before and after any action such as the addition of extra sections of riser pipe. These measurements, as well as an independent measurement of the length of the section of pipe being added, permit appropriate corrections to be made to the recorded measurements. When extra sections of riser pipe are being added, ensure that only the coupling at the bottom of the section being added turns. Use two wrenches, one to hold the new section of pipe and the other to hold the coupling or the pipe immediately below it, depending on whether or not the coupling was in place during the last sequence of measurements.

7.5 Appropriate measures shall be implemented to maintain the alignment of the riser and the riser pipe isolation casing in a vertical position during the period that data is be collected. Construction equipment must be operated in a manner to ensure that the settlement platforms are not damaged or displaced laterally. Each assembly shall be clearly marked and flagged with ground stakes or protective barricades, if appropriate.

7.6 Although settlement platforms are generally used for relatively short-term applications, there may be some cases where long-term performance is a consideration. Issues such as their performance over extended periods of time as well as corrosion of the components should be appropriately considered. For cases where factors such as backfill materials and methods, down-drag, isolation casing alternatives or effects of the instrument on load distributions are likely to impact the precision of the recorded measurements, appropriate procedures are identified on a case by case basis.

7.7 The number and spacing of settlement platforms is project dependent and therefore no specific guidelines are presented in this standard.

8. Report

8.1 For settlement platform measurements, report the following information:

8.1.1 Settlement platform identification and initial elevation

8.1.2 Reference point type

8.1.3 Elevation of the reference point

8.1.4 Description of measuring device(s) used

9. Keywords

9.1 monitoring fill placement; field instrumentation; settlement platforms ; vertical settlement

SUMMARY OF CHANGES

In accordance with Committee D18 policy, this section identifies the location of changes to this standard since the last published edition (00) that may impact the use of this standard.

(1) Added Section 9, Keywords.