



## **FINAL REPORT**

### **FOUNDATION INVESTIGATION AND DESIGN REPORT** **New Storage Structure at McKerrow Patrol Yard, Hwy 17, Township of Baldwin, Sudbury Area**

**Agreement No. 5013-E-0008**  
**Assignment No. 7**  
**WO 2015-11006**

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May 22, 2015

# Ministry of Transportation

## Foundation Investigation and Design Report

Agreement No. 5013-E-0008

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WO 2015-11006

Geocres No. 411-332

### Type of Document:

FINAL

### Project Name:

New Storage Structure at McKerrow Patrol Yard, Hwy 17, Township of Baldwin, Sudbury Area

### Project Number:

ADM-00028245-H0

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# 1 FOUNDATION INVESTIGATION REPORT

## 1.1 Introduction

This report presents the results of a geotechnical investigation carried out by **exp** Services Inc. (**exp**) for the proposed new storage structure located at the McKerrow Patrol Yard, which is located on Hwy 17 in the Township of Baldwin, Sudbury area. The work was undertaken under Agreement # 5013-E-0008, Assignment No. 7 (WO 2015-11006). The terms of reference (TOR) were as presented in the Ministry of Transportation (MTO) letter received on March 02, 2015. The location of the new storage structure with dimensions of 24.4 m x 42.7 m is specified in the TOR (site map PLAN H-332-17-1).

The purpose of the investigation is to establish the existing subsurface conditions at the proposed location of the patrol yard structure within the construction limits. The site specific geotechnical investigation consisted of field investigation including visual inspection, drilling, soil sampling, and laboratory testing. Factual results of the geotechnical investigation and laboratory testing are included in this report. The report has been prepared specifically and solely for the projects described in the report. A hydrogeological assessment at the site was not in a scope of this investigation.

## 1.2 Site Description and Geological Setting

### 1.2.1 Site Description

The McKerrow Patrol Yard is located on Hwy 17 in the Township of Baldwin, Sudbury area, approximately 3 km east of the Hwy 6 and Hwy 17 junction (see Key Map on Drawing 1, Appendix B). The terrain at the structure site is relatively flat as shown on photographs included in Appendix A.

In the proposed structure area, there are a sand dome, salt sheds, and sand/gravel stockpiles. The existing sand dome of about 31 m diameter is located approximately 55 m northeast of the existing MTO benchmark BM (Elev. 209.060 m) as marked on the site map, PLAN H-332-17-1, provided by MTO and shown on Drawing 1 in Appendix B. The existing salt sheds are located approximately 2 m west of the existing sand dome. The sand stockpile is about 28 m in diameter and is located approximately 8 m east of the sand dome, while the gravel stockpile is located about 25 m northeast of the sand dome. The site plan is as shown on Drawing 1 in Appendix B.

### 1.2.2 Geological Setting

According to Bedrock Geology of Ontario Map 2544 (Ministry of Northern Development and Mines, Ontario), the bedrock underlying the site is from the Mesoproterozoic geologic era (approximately 0.57 to 1.6 billion years old) and falls under Central Gneiss Belt which consists of migmatitic rocks and gneisses of uncertain protolith commonly layered biotite gneisses and migmatites and locally includes quartzofeldspathic gneisses, orthogneisses, and paragneisses.

According to Surficial Geology Map by the Province of Ontario's Ministry of Northern Development, Mines and Forestry (MNDMF), the surficial deposit in this area is a discontinuous layer of drift Precambrian deposit.

## 1.3 Investigation Procedures

### 1.3.1 Field Work

The current field investigation was carried out during March 16 and March 17, 2015. The field program consisted of drilling four (4) sampled boreholes (BH M15-1, BH M15-2, BH M15-3, and BH M15-4) and four (4) dynamic cone penetration test (DCPT) adjacent to each sampled boreholes. The boreholes were initially planned to be drilled at the each corner of the proposed building. However, the borehole locations were dictated by accessibility to these locations at the site at the time of drilling. The existing sand dome obstructed the drilling of BH M15-4, so that borehole was drilled at the best accessible location adjacent to the initially proposed location of that borehole. Drawing 1 in Appendix B shows the locations of four boreholes. The depths of the boreholes were around 10 m.

The borehole locations (referenced to the MTM NAD83 coordinate system) and their ground surface elevations were surveyed by **exp** personnel, with reference to the benchmark at the southwest of the sand dome (MTO BM Elevation 209.060 m), as shown in the site map provided by MTO (PLAN H-332-17-1).

The boreholes were advanced using a track-mounted CME 55 drill rig, equipped with continuous flight hollow stem augers/diamond drill with NW casing. All borehole drilling/sampling were operated by a specialist drilling contractor, LandCore Drilling Co. Ltd.

During the drilling operation, soil samples were obtained using a 51 mm outside diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586), at intervals shown on the attached borehole logs (Appendix C). The original field (uncorrected) SPT "N" values were recorded on the borehole logs as recommended in the Canadian Foundation Engineering Manual (CFEM, pg. 40), and used to provide an assessment of in-situ consistency of cohesive soils or relative density of non-cohesive soils. All boreholes were encountered with sand heaving at a depth ranging from 2.1 m to 3.0 m. In this case, wash boring was utilized to facilitate taking representative samples at designated elevation with reasonable accuracy. A dynamic cone penetration test (DCPT) was performed in the vicinity of each borehole to assess relative density of the soil.

At each borehole, the groundwater level measurements were carried out in the open auger holes before starting of wash boring. The measured groundwater levels were recorded in borehole log sheets in Appendix C. The boreholes were decommissioned by bentonite/cement mixtures in accordance with the Ministry of the Environment Regulation 903, as amended by Regulation 128/03 (the well regulation under the Ontario Water Resources Act).

The fieldwork was supervised by a member of **exp's** engineering staff who directed the drilling and sampling operation, logged borehole data in accordance with MTO and/or ASTM standards for soils classification, and retrieved soil samples for subsequent laboratory testing and identification.

All of the recovered soil samples were placed in labelled moisture-proof bags and returned to **exp's** Brampton laboratories for additional visual, textual and olfactory examination, and sampling for lab testing.

### **1.3.2 Laboratory Testing**

All samples returned to the laboratory were subjected to visual examination and classification. The laboratory testing program included the determination of natural moisture content and particle size distribution for approximately 25% of the collected soil samples. All of the laboratory tests were carried out in accordance with MTO and/or ASTM standards as appropriate.

The laboratory test results are provided on the attached borehole log sheets in Appendix C. The results of the grain size analyses are presented graphically in Appendix D.

### **1.3.3 Previous Investigation**

The foundation report from April 1962 related to this site and named "Foundation Investigation Report for Proposed D.H.O. Patrol Yard at Hwy 17, 1.5 Miles West of McKerrow, District of Sudbury, District No.17" (GEOCRE 41I00-020) is provided by MTO. The locations of the historical boreholes drilled in April 1962 are shown on Drawing 1 in Appendix B and the soil stratigraphy assessed during that investigation is shown on the attached drawing in Appendix E.

## **1.4 Subsurface Conditions**

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are presented on the borehole log sheets in Appendix C. Laboratory test results are provided in Appendix D. The "Explanation of Terms Used in Report" preceding the borehole logs in Appendix C forms an integral part of and should be read in conjunction with this report.

A borehole location plan and stratigraphic section along the proposed storage structure alignments are provided in Appendix B. It should be noted that the stratigraphic boundaries indicated on the borehole logs and stratigraphic section are inferred from non-continuous sampling, observations of drilling progress and results of Standard Penetration Tests. These boundaries typically represent transitions from one soil type to another and should not be regarded as exact planes of geological change. Further, subsurface conditions may vary between and beyond the borehole locations.

In general, the stratigraphic sequence at the proposed structure site consists of top silty sand fill/ gravelly sand fill to silt, underlain by sand deposits and followed by silty sand deposits. A brief summary of the soil and groundwater conditions encountered in the boreholes is provided below.

### 1.4.1 Fill: Gravelly Sand/Silty Sand

Gravelly sand/silty sand fill was encountered below 25 mm thick layer of asphalt in BH M15-1 and BH M15-4 and at ground surface in BH M15-2. The thickness of gravelly sand/ silty sand fill layer ranged from 0.8 m to 1.5 m extending from Elev. 208.4 m to 206.5 m.

This layer consists of fine gravel, fine to coarse sand, few to some silt. The material is blackish grey to brown in color and moist. The SPT "N" values within this layer is about 30 blows per 300 mm of penetration, suggesting compact relative density of this layer.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture content:

- 5.3% to 30.8%

Grain size distribution:

- 30% gravel;
- 63% sand; and
- 7% silt and clay.

The results of moisture content and grain size distribution tests are provided on the record of the borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 1 in Appendix D.

### 1.4.2 Silt

A layer of silt was encountered at the ground surface in BH M15-3 and below gravelly sand fill layer in BH M15-2 and BH M15-4. The thickness of silt layer ranged from 0.7 m to 1.5 m extending from Elev. 207.8 m to 206.8 m. .

This layer consists of silt, trace gravel, trace to few sand, and trace to few clay. At BH M15-3, the silt layer contains trace of organics. It is grey and brown in color and moist. At BH M15-4 the silt layer was frozen. The SPT "N" values within this layer ranged from 21 and 55 blows per 300 mm of penetration, suggesting compact to very dense relative density of this layer, but more typically compact to dense condition.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture content:



- 17.4% to 23.4%

Grain size distribution:

- 1% gravel
- 4% to 7% sand;
- 89% to 91% silt; and
- 2% to 6% clay.

The results of moisture content and grain size distribution tests are provided on the record of the borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 2 in Appendix D.

### 1.4.3 Sand

A layer of sand was encountered below silty sand fill layer in BH M15-1 and below silt layer in BH M15-2, BH M15-3 and BH M15-4. The thickness of sand layer ranged from 3.8 m to 8.3 m extending from Elev. 207.1 m to 198.3 m. Sampling of BH M15-1 and BH M15-4 were terminated within this layer.

This layer consists of medium to fine sand, trace gravel, and trace to little silt. It is reddish to greyish brown in color and wet. The SPT "N" values within this layer typically ranged from 2 to 18 blows per 300 mm of penetration. One SPT "N" value of 34 blows per 300 mm penetration was encountered in BH M15-3. SPT "N" values measured within this layer suggests that the sand layer is very loose to compact in relative density.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture content:

- 20.2% to 30.8%

Grain size distribution:

- 1% gravel
- 85% to 99% sand;
- 1% to 15% silt and clay

The results of moisture content and grain size distribution tests are provided on the record of the borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 3 in Appendix D.

#### 1.4.4 Silty Sand

A layer of silty sand was encountered below sand layer in BH M15-2 and BH M15-3. The thickness of silty sand layer ranged from 3.7 m to 5.2 m extending from Elev. 203.3 m to 198.1 m. Sampling of BH M15-2 and BH M15-3 were terminated within this layer.

This layer consists of sand and trace to some silt. It is greyish brown in color, wet. The SPT "N" values within this layer ranged from 2 to 10 blows per 300 mm of penetration, suggesting very loose to compact relative density of this layer.

Laboratory testing performed on selected samples consisted of moisture content and grain size distribution tests. The test results are as follows:

Moisture content:

- 22.3% to 29.2%

Grain size distribution:

- 66% to 99% sand;
- 1% to 34% silt and clay

The results of moisture content and grain size distribution tests are provided on the record of the borehole sheets in Appendix C. The results of the grain size distribution tests are also provided on Figure 4 in Appendix D.

### 1.5 Groundwater Conditions

Information regarding groundwater levels at the site was obtained measuring the water levels in the open auger holes before starting wash boring to advance the boreholes. The groundwater levels measured in the boreholes are shown on Table 1.1 and borehole logs. Water levels measured in open holes might not be stabilized due to short term observation.

Table 1.1 Groundwater levels at the site

Borehole No.	Date of Drilling	Groundwater Level	
		Depth, (m)	Elevation, (m)
BH-1	03/16/2015	1.2	206.85
BH-2	03/16/2015	1.2	207.1
BH-3	03/16/2015	1.2	206.6
BH-4	03/16/2015	0.9	207.47

## 2 DISCUSSIONS AND ENGINEERING RECOMMENATIONS

### 2.1 General

This section of the report provides geotechnical design recommendations for the proposed MTO Patrol Yard structure located at McKerrow Patrol Yard, Hwy 17, Township of Baldwin, Sudbury area. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current investigation at the site and presented in **Part I-Foundation Investigation Report**. The interpretation and recommendations provided are intended solely to permit designers to assess foundation alternatives, and design the proposed structure. Comments on construction are only provided to highlight issues that could affect the design. Contractors bidding on the works should make their own assessments of the factual data and how it might affect construction means and methods, scheduling and the like.

Based on information included in the TOR and correspondence with MTO it is understood that the existing storage structures will be replaced with a new material storage which will be similar to that at the Cartier Patrol Yard, Cartier Township, in the Sudbury area. The proposed building will consist of a conventional building for storage of road sand/salt, and will allow for inside loading and dumping. Based on the Patrol Yard Site Plan drawing (Plan H-332-17-1) the proposed sand/salt building will have a footprint of about 24.4 m x 42.7 m in plan dimensions. In addition, based on design drawings of the MTO Cartier Patrol Yard (Drawings No. 10-632-1 to 10-632-5 by James Knight & Associates) provided by MTO, it is understood that the proposed structure will have a maximum height of about 11.0 m to the bottom of the trusses (Drawing No. 10-623-4) and it will be encompassed with a 3.8 m high, cast-in-place concrete walls around the perimeter. The building will have a steel roof and timber wall cladding, and an asphalt floor slab within the plan area of the storage. The assumed finished top of the asphalt floor will be approximately at Elev. 208.2 m to tie-in to the existing exterior paved areas.

This report addresses the geotechnical design of the foundation for the proposed structure by providing geotechnical design parameters at the Ultimate Limit State (ULS) and Serviceability Limit States (SLS) as well as other geotechnical parameters that may be required in accordance with the latest edition of the *Canadian Highway Bridge Design Code (CHBDC)* (CAN/CSA-S6-14), the *Guideline for Professional Engineers Providing Geotechnical Engineering Service* (1992), the *Canadian Foundation Engineering Manual (CFEM)* (2006), the *provisions in the TOR* and good practice. It also provides discussion about the structure foundation type, stability and settlement analyses, frost protection, construction considerations and dewatering during construction, as requested in the TOR from MTO letter dated March 2, 2015.

As further requested in the TOR, the settlement and stability analyses were completed for a scenario in which the new structure would be loaded to its full allowable capacity. This scenario consists of winter sand/salt stacked to the maximum allowable height of the concrete wall with a stockpile area covering the entire footprint of the building.

## 2.2 Geotechnical Design Considerations for Foundations

The geotechnical investigation and its findings pertaining to the subsurface soil characteristics have been covered in Part I - Foundation Investigation Report 1 which contains details of the field and laboratory aspects of the investigation. In general, the subsurface conditions encountered at the site below the asphalt layer in the area of the existing dome consist of up to 1.5 m of compact gravely to silty sand fill underlain by a non-cohesive deposit of loose to compact sand found up to 9.8 m depth (measured 'N' values between 4 and 18; average 'N' value is 8 blows per 300 mm of penetration). The subsurface conditions at the site in the area without the asphalt on the ground surface consist of up to 1.5 m of compact silt underlain by an approximately 4 m thick, very loose to compact, layer of sand (measured 'N' values between 2 and 34; average 'N' value is 9 blows per 300 mm of penetration) followed by a deposit of very loose to compact silty sand to the depth of 9.8 m (measured 'N' values between 2 and 10; average 'N' value is 5 blows per 300 mm of penetration). The average 'N' value of the deposit measured at the site during the previous investigation in 1962 was 25 blows per 300 mm of penetration (see Appendix E). The dynamic cone penetration tests carried out during this investigation gave 'N' values between 10 and 30 blows/300 mm of penetration. The bedrock or very dense material was not encountered within the borehole termination depth (i.e., to 9.8 m below ground surface) during this investigation. The results of dynamic cone penetration tests carried out in the previous investigation indicated the very dense material ('N' value > 50) at the depth of approximately 12 m (see Appendix E). The unstabilized groundwater levels were measured at elevations between Elev. 206.8 m and 207.5 m.

### 2.2.1 Evaluation of Foundation Alternatives

Considering that bedrock or very dense material was not encountered within the borehole termination depth during this investigation, as well as the high cost of pile foundations and the structure's operating life it is unlikely that deep foundations can be considered practical for this patrol yard structure. It appears that shallow foundations are more practicable. However, an evaluation of these two foundation alternatives is included in this report. Advantages and disadvantages of shallow foundations such as strip/spread footings and deep foundations such as driven steel H-piles are presented in Table 2.1.

Given the subsurface conditions at the site the impact on settlement at the foundations of the structure will be influenced by the operating/stockpiling practices. It is our understanding that the structure will accommodate stockpiles of both road sand and salt at strategic locations within the structure. Based on the information mentioned in Section 2.1, the maximum loading condition is likely to be sand/salt stockpiled to at least the level of the concrete wall over the full footprint. Mounding in the centre at the angle of repose is also a possibility.

These types of structures generally have service lives of about 20 years. Typically, in settings of poor soil conditions, the approach would be to mitigate potential distress for a shallow foundation supported on it rather than employ expensive deep foundations for building support. Mitigation to create stable foundation soils can include vibrocompaction to densify loose sands, preloading of the footprint area before construction, structure support on engineered fill and/or stockpiling constrains in order to enhance serviceability.

Table 2.1 Evaluation of foundation alternatives

Options	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
<b>Shallow Foundation</b> (Strip/Spread Footings on Native Loose Sand Material)	1*	<ul style="list-style-type: none"> <li>▪ Straightforward construction</li> </ul>	<ul style="list-style-type: none"> <li>▪ Fairly low geotechnical resistance available</li> <li>▪ Foundation on engineered fill may be necessary</li> <li>▪ Depending on conditions, some stockpiling constraints may be necessary</li> </ul>	<ul style="list-style-type: none"> <li>▪ Significantly lower relative cost compare to piles</li> </ul>	<ul style="list-style-type: none"> <li>▪ Risk of differential settlements due to loading patterns in the past and during operations</li> <li>▪ Possible constraints on a storage volume</li> </ul>
<b>Deep Foundation</b> (Driven Steel H-Piles )	2	<ul style="list-style-type: none"> <li>▪ Straightforward construction</li> </ul>	<ul style="list-style-type: none"> <li>▪ Not typical for this type of structure</li> <li>▪ Could be very long piles, not warranted for this type of structure</li> </ul>	<ul style="list-style-type: none"> <li>▪ Higher relative costs compared with shallow foundations</li> <li>▪ Unlikely to be economically feasible at the site</li> </ul>	<ul style="list-style-type: none"> <li>▪ Deep firm ground</li> <li>▪ Not viable due to cost</li> </ul>

\* If geotechnical resistance is adequate, otherwise vibrocompaction and/or preloading and/or founding on engineered fill and/or stockpiling constraints may be necessary.

Shallow foundations for the sand/salt storage structure should consist of strip/spread footings which typically for this kind of structure have a width of 3 m (i.e. the Cartier Patrol Yard project). The footings could be founded on/within the native sand deposit, or on free draining engineered fill, such as Granular 'A' or Granular 'B', Type I or Type II (OPSS.PROV 1010).

The feasibility of shallow foundations depends on whether the structure can be accommodated in ground conditions with the axial resistance and settlement conditions described below. If the geotechnical resistances provided below for strip/spread footings are not sufficient for the design of the structure and a deep foundation option is required, additional boreholes extending to greater depths and potentially to refusal would be required. Providing the refusal is between 10 m and 15 m below the footing, an application of helical piles can be also considered.

## 2.2.2 Footing Elevation

Based on the results of the geotechnical investigation and a requirement for adequate protection against frost penetration in the project area (i.e. a minimum 1.9 m below the lowest surrounding area, see Section 2.2.5), the following founding elevations of strip/spread footings are recommended:

Table 2.2 Recommendations for footing elevation

Soil at Founding Level	Foundation Elevation (m)	Depth Below Existing Grade
Engineered Fill over Native Sand	205.9	1.9 m to 2.5 m (requires excavation to Elev. 203.9 m and up to 2.0 m of engineered fill)
Native Sand	205.9	1.9 m to 2.5 m

## 2.2.3 Geotechnical Resistances

In the context of the CHBDC, a satisfactory foundation design would require, in terms of Limit States Design, the factored geotechnical resistance of its foundation to withstand and not exceed the imposed Ultimate Limit State loads - (ULS) Design Approach, and its ability to deform acceptably under the Service Limit State loads - (SLS) Design Approach. These associated loads are typically known as unfactored and factored loads, respectively. Therefore, strip/spread footings placed on the properly prepared subgrade at the design elevation given in Table 2.2, should be designed based on the factored resistances at ULS and geotechnical reactions at SLS for 25 mm of settlement given in Table 2.3 below. The footing width of 3 m is assumed. Settlement of the footings under the loading from the stockpiles inside the structure which will occur after its construction is considered and discussed in Section 2.6.2. In determining the settlement characteristics of the proposed building (tolerable total and differential settlement), the unfactored loads are required to be provided by the Structural or Design Engineer.

**Table 2.3 Geotechnical resistance at ULS and geotechnical reaction at SLS for a 3 m wide footing**

Soil at Founding Level	Width of Footing (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa) (for 25 mm settlement)
Engineered Fill over Native Sand	3	600	180
Native Loose Sand	3	350	120

Since the ULS resistance and the settlement depend on the footing size and depth of embedment, the geotechnical resistances given in Table 2.3 should be reviewed if the selected footing width or founding elevations differ from those given in the table. Similarly, if an inclined load is applied instead of a vertical load, which are used in these calculations, the values given in Table 2.3 has to be reviewed to take into account those inclinations.

Prior to placing footings, the exposed native subgrade should be inspected according with OPSS 902. A Qualified Geotechnical Engineer should check that the design foundation elevation is achieved and all unsuitable soils including fill, organics and those soils with the USCS classification of CH, OH, MH, OL or PT have been removed.

## 2.2.4 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the subgrade and concrete should be calculated in accordance with Section 6.7.5 of the CHBDC. The unfactored values of the coefficient of friction,  $\tan \delta$ , between the base of cast-in-place concrete footing and the granular subgrade soils below the frost level are presented in Table 2.4.

**Table 2.4 Recommendations for coefficient of friction**

Soil at Founding Level	Coefficient of Friction, $\tan \delta^*$
Engineered Fill over Native Sand	0.6
Native Loose Sand	0.35

\*- based on NAVFAC 1986, Table 1, pg. 7.2-63

A factor of 0.8 should be applied in calculation of the horizontal resistance in accordance with CHBDC.

In a case of the unbalanced lateral earth pressures caused by sand/salt stockpiles being piled against the perimeter walls, these walls should be designed based on the following geotechnical parameters assuming a triangular lateral earth pressure distribution:

- Unit weight of sand/salt stockpile material =  $20 \text{ kN/m}^3$
- Friction angle of sand/salt stockpile material =  $30^\circ$
- Lateral earth pressure coefficient ( $K_0$ ) = 0.5

## 2.2.5 Frost Protection

According to Ontario Provincial Standard Drawing (OPSD – 3090.101), the frost depth in Township of Baldwin is about 1.9 m. Consequently, all footings exposed to seasonal freezing conditions should be protected from frost action by at least 1.9 m of soil cover or equivalent insulation.

## 2.3 Earthquake Considerations

Recommendations for the geotechnical aspects to determine the earthquake loading are presented below.

### Subsoil Conditions:

The subsoil and groundwater information at this site have been examined in relation to Section 4.1.8.4 A of the Ontario Building Code (OBC, 2012). The subsoil generally consists of silt, sand and silty sand layers. It is expected that the foundations will be founded in the sand layer underlain by the silty sand layer. The reported N-values for the soil below the founding level ranged from 2 to 16, with an average value about 7.

### Corrected N-Values $N_{60}$ :

The Average Standard Penetration Resistance shown in Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2012 refers to  $N_{60}$  which is defined as “Average Standard Penetration Resistance for the top 30 m, corrected to a rod energy efficiency of 60% of the theoretical maximum”. It should be noted that the drillers in the Sudbury area do not have their rod energy efficiencies measured and therefore, computed  $N_{60}$  values are not available for this site. In our opinion, the reported N-values could be considered as an approximate equivalent to the normalized  $N_{60}$  values as noted in the OBC 2012 for the purpose of establishing the site classification.

### Depth of Boreholes:

Table 4.1.8.4.A. Site Classification for Seismic Site Response in OBC 2012 indicated that the average properties in the top 30 m are to be used to determine the site classification. The four (4) boreholes advanced for building construction at this site were approximately 9.8 m deep.

### Site Classification:

Based on the above assumptions and interpretations, and the soil conditions, the Site Class for this site is estimated to be “D” as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012.

These parameters should be reviewed by the Structural Engineer.

## 2.4 Liquefaction Considerations

The 9.8 m below the ground surface the site mainly consists of sand with SPT N-values ranging from 2 to 16 blows per 300 mm penetration. The water level after completion of drilling is at about 0.9 m to 1.2 m depth. According to measured SPT's values, the subsoil could potentially be susceptible to liquefaction. Accordingly, liquefaction analyses have been performed using the Seed's approach, which is



recommended by the CFEM (4<sup>th</sup> Edition 2006; Chapter 6, pg.101). This approach defines a factor of safety against liquefaction as the ratio of the induced cyclic stress ratio over the cyclic resistance ratio. The calculated factor of safety for the site subsoil is generally more than 1.0. As a result, liquefaction is not likely to occur in the upper soils at the project site for the earthquake having 10% probability of exceedance in a 50-year period.

## 2.5 Perimeter Wall and Floor Construction

The perimeter wall of the proposed structure may be constructed as a cantilever retaining wall with an extended heel toward the inside of the structure and founded on native soils or engineered fill. Structural steel bars should be provided in the footings and in the walls. The asphalt floor will be designed inside the structure. The construction of spread footing and subgrade for the asphalt floor may be carried out in accordance with the following recommendations:

Prior to construction, all obviously unsuitable material should be fully removed from the entire underfooting and underfloor area (see Section 2.7). Following rough grading, the exposed subgrade should be proofrolled with a roller under the full-time supervision of a qualified geotechnical personnel. Any soft spots detected during proofrolling should be sub-excavated and replaced with approved materials compacted to at least 98 % of the Standard Proctor Maximum Dry Density (SPMDD). The prepared subgrade should be covered with at least 200 mm compacted OPS Granular A, crowned slightly in the central area.

Around the perimeter of the building the ground surface should be sloped on a positive grade away from the structure to promote surface water run-off and reduce groundwater infiltration adjacent to the foundations. Permanent perimeter drains are not required if the interior base is set at least 200 mm above the exterior grade and the grade is sloped away from the structure. However, a permanent subfloor drainage system may be required to collect salt-bearing water. In order to minimize contamination into the native soils and subsequently into the groundwater, a barrier such as a compacted low-permeability clay liner or geomembrane should be installed below the sand/salt storage area. In practice the use of geomembrane shows advantage over the compacted clay liner in terms of improved performance of the barrier. The geomembrane should be installed on a minimum 75 mm thick sand layer and covered with a 300 mm thick layer of sand fill on top of the geomembrane in order to protect it from the overlying pavement structure. In addition, asphalt used for the floor of the structure should be sealed to minimize infiltration, since it is somewhat permeable. However, sulphate resistant concrete should be used for structure foundations.

## 2.6 Stability and Settlement Analyses

### 2.6.1 Stability

To assess the global stability of the storage structure and to check that a minimum Factor of Safety of 1.3 will be achieved for the maximum height sand/salt stockpile of approximately 10.9 m, a series of slope stability analyses were performed. The static slope stability analyses were performed using the

Morgenstern-Price method developed on the basis of limit equilibrium. The SLOPE/W computer program developed by GeoSlope International was employed for computation.

Stability assessments were performed for two cases: (i) an approximately 10.9 m high stockpile (assumed slopes of 1.7H:1V) restrained with both sides by concrete side walls as shown on Figure F1, and (ii) an approximately 10.9 m high stockpile with the restrained back side with the concrete wall and an unrestrained front side with assumed slope of 1.5H:1V as shown on Figure F2. It should be noted that the side stability of the stockpile will be governed by the angle of repose of the stockpile material, which can be different than those assumed in these analyses. The stratigraphy and groundwater condition at the site were developed based on the results of the geotechnical investigation presented in Part I - Foundation Investigation Report.

Given the above, effective stress analyses for a long term stability assessment were performed taking into consideration the subsoil conditions encountered directly beneath and adjacent the proposed structure. The asphalt floor was not included in the analyses, but it is expected that it will provide additional stability support to the system.

Tabulated below in Table 2.5 are the soil parameters used for the slope stability analyses. The soil parameters were generally estimated based on the results of field and laboratory investigation.

*Table 2.5 Soil properties used in slope stability analyses*

Material Type	Effective Stress Parameters		
	$\phi$ (degrees)	c (kPa)	$\gamma$ (kN/m <sup>3</sup> )
Engineered Fill	35	0	22
Loose Sand	28	0	18
Loose Silty Sand	28	0	18
Sand/Salt Stockpile Material	30	0	20

The graphical results of these analyses can be seen in Appendix F. As shown on Figure F1, the results of stability analyses for an approximately 10.9 m high sand/salt stockpile restrained with concrete walls on both sides suggest that the factor of safety of 1.3 can be obtained for a deep-seated failure surface. The same results (i.e. min FOS =1.3) were obtained for the approximately 10.9 m high stockpile with the restrained back side and an unrestrained front slope, but for the stockpiled sand sliding along the membrane, as shown on Figure F2.

## 2.6.2 Settlement

To evaluate the maximum settlement and differential settlement values bellow the sand/salt stockpile loading in the proposed storage building, a 3D computer program; Settle3D (Rocscience) was employed.

Considering the cohesionless nature of the soil material encountered at the site it is anticipated that only an immediate settlement would be occurred under the loading. The elastic moduli of deformation for the encountered soil layers used in the settlement model are evaluated based on the results of the SPT and DCPT as per CHBDC. The parameters are listed in Table 2.6.

*Table 2.6 Soil properties used in settlement analyses*

Material Type	$\gamma$ (kN/m <sup>3</sup> )	E (MPa)	Poisson's Ratio
Engineered Fill	22	120	0.3
Loose Sand	18	25	0.3
Loose Silty Sand	18	25	0.3
Sand/Salt Stockpile Material	20	-	-

The geometry of the stockpile was assumed based on its maximum allowable capacity; a maximum height of approximately 10.9 m at the center (with side slopes of 1.7H:1V) and 3.8 m along the sides at the concrete wall. The settlements under the maximum capacity of stockpile were computed for two cases: (i) the structure is founded directly on the loose native sand, and (ii) the loose native sand is replaced by a 2 m thick engineered fill. The models for these two cases are illustrated on Figures G1 and G2 included in Appendix G.

The results of the settlement analyses for the case with the native sand soil and the case with the engineered fill are plotted on Figure G3 and G4 (Appendix G), respectively. The estimated settlement under the stockpile at the center and at the edge of the stockpile (i.e. location of footings) is presented in Table 2.7.

*Table 2.7 Results of settlement analyses for the proposed structure*

Foundation Soil Type	Estimated Settlement (mm)		
	At Center	At Edge	Differential
Loose Sand	87	46	41
Engineered Fill	69	37	32

As it is mentioned, the calculated settlement is considered immediate after the stockpile loading. However the loading and consequent settlement would be occurred after the footings have been constructed. Therefore, the footing for this structure has to be design under the full allowable stockpile loading. The geometry of stockpile under the full allowable loading including its maximum height is

recommended above. It is also recommended that the designer includes detailed procedures in the contract drawings and note.

If the footprint area is preloaded by a gravel/sand stockpile prior of construction, the post-construction settlement can be significantly reduced. The settlement analyses for different height of the stockpile preloading were performed and the results are presented in Table 2.8 and attached Figures G5 – G7, appendix G. The results show that the immediate settlement of approximately 37 mm, 50 mm and 60 mm at the center could be achieved by placing a 5 m, 7 m, and 9 m high stockpile, respectively. The settlement of 16 mm to 20 mm can be produced at the proposed location of the storage footings. Therefore, these analyses demonstrate that preloading can significantly reduce the postconstruction settlement. It is anticipated that these predicted immediate settlements will take place as the load is applied or within a time period of about 14 days.

Table 2.8 Results of settlement analyses for preloading

Height of Stockpile Preloading (m)	Estimated Settlement at Centre (mm)	Estimated Settlement at Location of Proposed Footing (mm)
5	37	16
7	50	18
9	60	20

## 2.7 Site Preparation and Engineered Fill Construction

As mentioned previously, the area within the limits of the building should be stripped and cleared of surface vegetation, topsoil and debris prior to construction. Any soils containing excessive organics or loose/disturbed materials are not suitable for the subgrade of building foundations, floor slabs or engineered fill. Therefore, areas with those soils should be excavated and replaced with engineered fill comprised of Granular A or Granular B, Type I or Type II. Considering the high groundwater table at the site the preferred engineered fill should be Granular B, Type II. However, in order to prevent migration of fine soil particles a geotextile can be used underneath Granular B, Type II.

Engineered fill could be placed after stripping all topsoil, organic matter, fill and other compressible, weak and deleterious materials within an area extending at least 1.0 meters beyond the outside edge of the founding level of any footings. After stripping, the entire area should be heavily proof-rolled inspected and approved by a Geotechnical Engineer. Engineered fill should be placed in accordance with OPSS 501 and SP SP105S21. The fill material should be placed in thin layers not exceeding approximately 300 mm when loose. Oversize particles larger than 120 mm should be discarded, and each fill layer should be uniformly compacted with heavy compactors, suitable for the type of fill used. The engineered fill below the footing and floor slab should be compacted to 100% of its SPMDD, while within

outside/exterior paved areas, the fill should be compacted to 98% of its SPMDD.

Full-time geotechnical inspection and quality control (by means of frequent field density and laboratory testing) should be provided by the Geotechnical Engineer. Every lift should be evaluated by a sufficient number of tests to ensure that the level of compaction is constantly achieved and the compaction procedure is applied.

Considering the loose condition of the native foundation soil and an uneven loading pattern of the proposed footprint area (i.e. only approximately 1/3 of the area is occupied by the existing storage), a ground improvement technique such as vibrocompaction to densify loose soils and/or preloading is recommended to be applied at the site prior to construction. The combined action of vibration and water saturation by jetting can rearrange loose sand grains into a more compact state. Vibrocompaction can be performed with specially-designed vibrating probes. The probe will be first inserted into the ground by both jetting and vibration. After the probe reaches the required depth of compaction, granular material, usually sand, will be added from the ground surface to fill the void space created by the vibrator and a compacted radial zone of granular material will be created.

## **2.8 Excavation and Groundwater Control**

For the construction of the proposed structure, excavations at least about 1.9 m depth will be required. The excavations are expected to encounter mostly sandy materials and below the groundwater level.

All excavations should be carried out in accordance with the latest version of the Occupational Health and Safety Act. For the purpose of the act, the existing materials are considered as Type 3 soils above the groundwater table and Type 4 soils below the groundwater table. Temporary excavations (i.e. those that are open only for a short period) above the groundwater table may be made with side slopes not steeper than about 1H:1V, while the temporary slopes below the groundwater table have to be formed at 3H:1V unless a suitable dewatering system is installed to lower the water level below the base of the excavation.

Considering the subsurface conditions at the site (i.e. sandy soils and the groundwater table approximately 0.9 m to 1.2 m below the ground surface) and the depth of the excavations, it is expected that excavations will encounter groundwater seepage. Therefore groundwater control, such as the use of the perimeter trenches and sumps within excavations, will be required in order to allow excavation in dry. Perimeter ditches should be incorporated into the surface water drainage plan to promote run-off away from the structures. An example of Non-standard Special Provision (NSSP) concerning dewatering of the native soils during excavation and foundation construction is attached in Appendix

It should be noted that the water levels in this area may fluctuate depending on the time of year. It is recommended that excavations for the footings be carried out in late summer when the water levels are anticipated to be lower.

May 22, 2015

### 3 CLOSURE

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

This Foundation Investigation and Design Report has been prepared by Silvana Micic, Ph.D., P.Eng. and Nimesh Tamrakar, M.Eng. and reviewed by TaeChul Kim, M.E.Sc., P.Eng. and Stan E. Gonsalves, M.Eng., P.Eng., Designated MTO Foundation Contact. The field investigation was conducted by Nimesh Tamrakar, M.Eng.

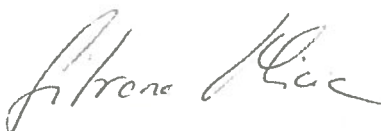
We trust that these comments provide you with sufficient information to proceed with design. Should you have any questions, please do not hesitate to contact this office.

Yours truly,

**exp Services Inc.**



Nimesh Tamrakar, M.Eng.  
Technical Specialist



Silvana Micic, Ph.D, P.Eng.  
Senior Geotechnical Engineer



Stan E. Gonsalves, M.Eng., P.Eng.  
Principal Engineer  
Designated MTO Foundation Contact

Encl.



## **Appendix A – Photographs**





Photo 1. Facing east from location of BH M15-2



Photo 2. Facing north from location of BH M15-2





Photo 3. Facing south from location of BH M15-2



Photo 4. Facing northeast from location of BH M15-1



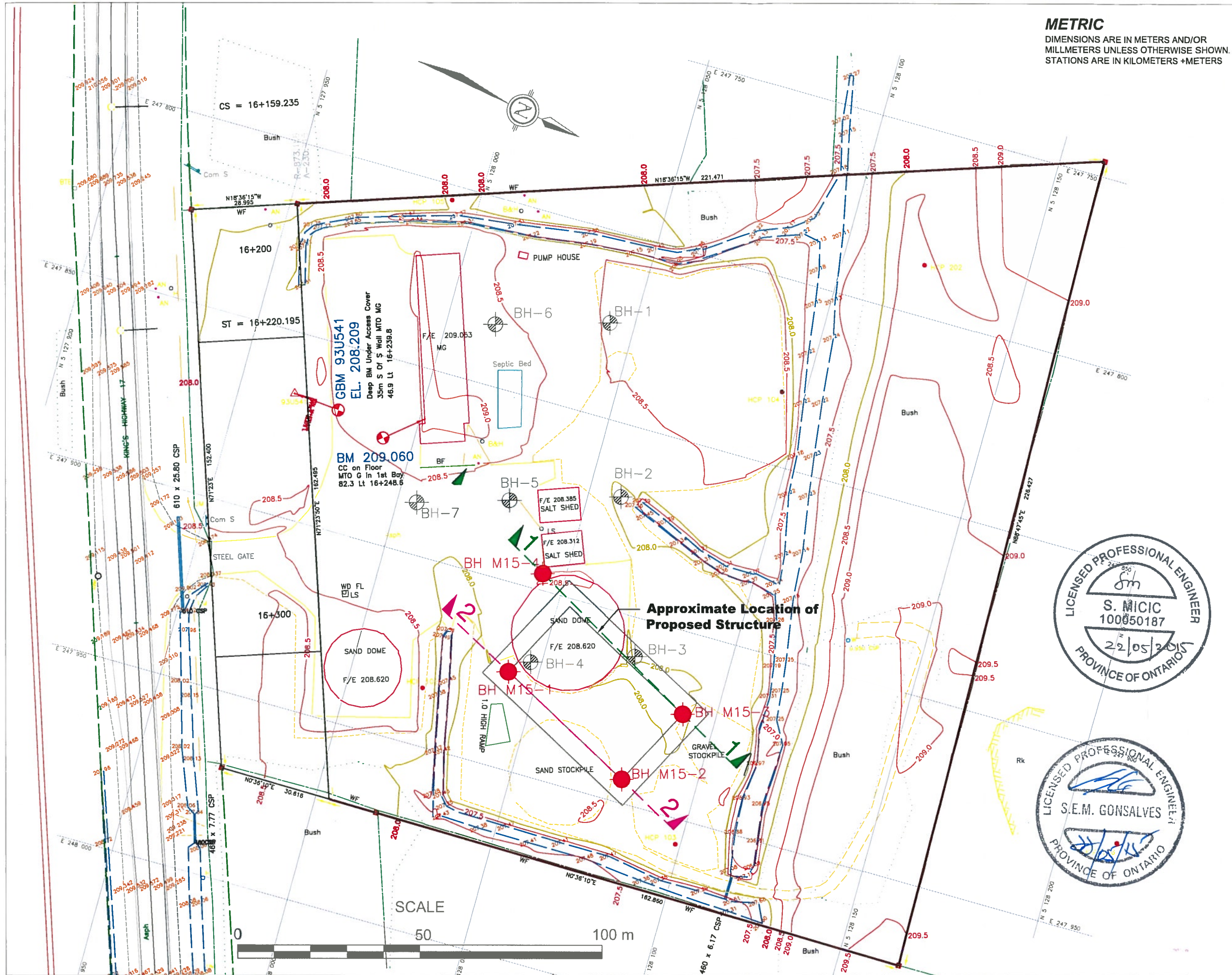
Photo 5. Facing west from location of BH M15-1



Photo 6. Facing northeast from location of BM

## **Appendix B – Drawings**





Agreement No. 5013-E-0008  
Assignment No. 7  
WO 2015-11006

**SAND AND SALT STORAGE  
STRUCTURES**  
Mckerrrow Patrol Yard  
Site Plan

SHEET  
1

exp

exp Services Inc.

KEY PLAN

LEGEND

BM (Bench Mark)

Borehole Locations(March, 2015)

Approximate Historical Borehole Locations (April, 1962)

BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
BM	209.060	5128000	247865
BH M15-1	208.06	5128042	247924
BH M15-2	208.31	5128080	247944
BH M15-3	207.82	5128091	247922
BH M15-4	208.38	5128044	247896

NOTE

Locations of the historical boreholes have been taken from the MTO Drawing No. 62-F-28 A.  
This drawing is for site information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.  
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in the report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

DATE	BY	DESCRIPTION
2015.05.22	SM	FINAL SUBMISSION
2015.04.30	SM	SUBMISSION FOR MTO REVIEW

SCALE	SEE SCALE BAR	PROJECT NO. ADM-00028245-H0
SUBM'D SM	CHECKED SM	DATE 2015.05.21
DRAWN NT	CHECKED SG	APPROVED SG DWG. 01



METRIC  
DIMENSIONS ARE IN METERS AND/OR  
MILLIMETERS UNLESS OTHERWISE SHOWN.  
STATIONS ARE IN KILOMETERS +METERS

Agreement No. 5013-E-0008  
Assignment No. 7  
WO 2015-11006

SAND AND SALT STORAGE  
STRUCTURES

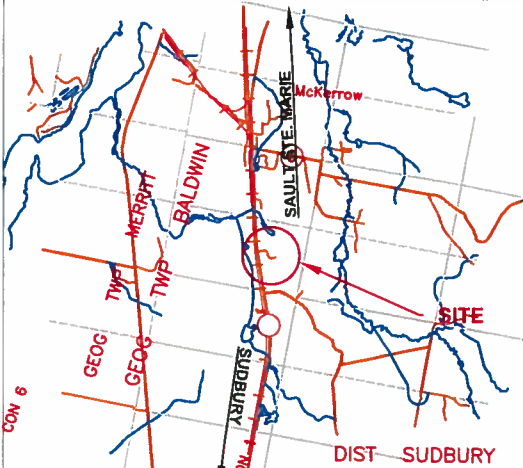
Mckerrow Patrol Yard  
Soil Strata

SHEET  
2



exp Services Inc.

KEY PLAN



LEGEND

- Borehole Locations (March, 2015)
- Approximate Historical Borehole Locations (April, 1962)
- Standard Penetration Test (Blows/0.3 m)
- Groundwater Level Measured in the Open Hole

SOIL STRATA SYMBOLS

- FILL
- SAND
- SILT
- SILTY SAND

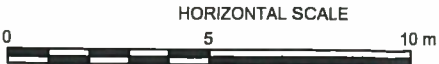
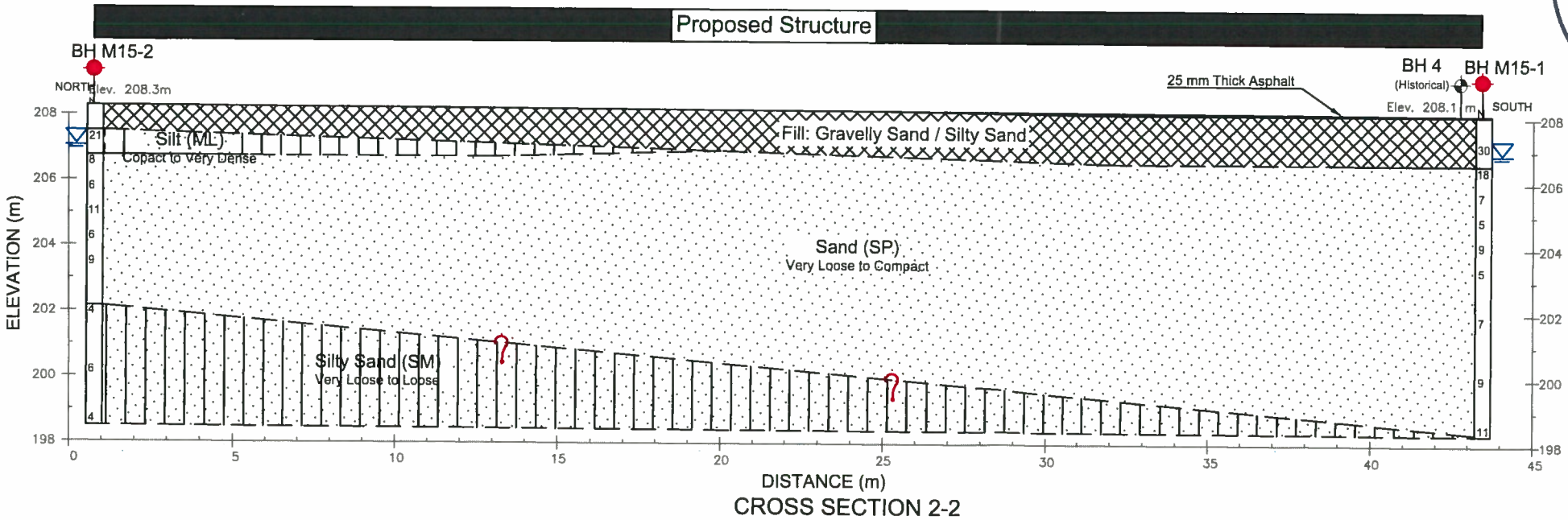
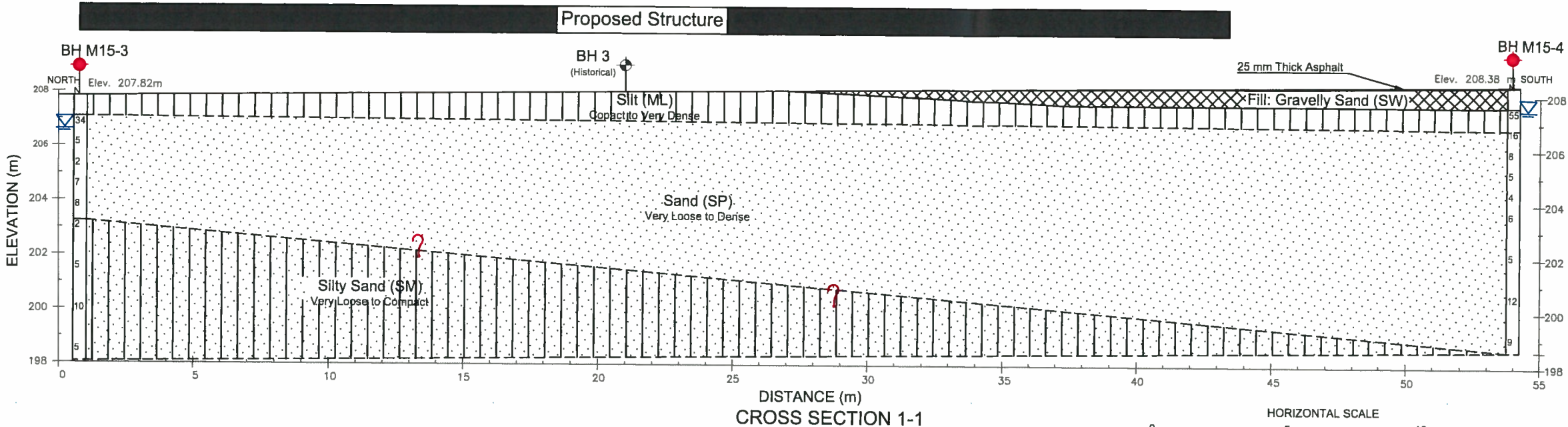
BH No.	APPROX. ELEV.	MTM CO-ORDINATES	
		NORTH	EAST
BH M15-1	208.06	5128042	247924
BH M15-2	208.31	5128080	247944
BH M15-3	207.82	5128091	247922
BH M15-4	208.38	5128044	247896

NOTE

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contract Documents.

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in the report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

DATE	BY	DESCRIPTION
2015.05.22	SM	FINAL SUBMISSION
2015.04.30	SM	SUBMISSION FOR MTO REVIEW
GEORES NO. 411-332		
SCALE	SEE SCALE BAR	PROJECT NO. ADM-00028245-H0
SUBM'D	SM	CHECKED SM
DRAWN	NT	CHECKED SG
		DATE 2015.05.21
		APPROVED SG
		DWG. 02



## **Appendix C – Borehole Logs**

# Explanation of Terms Used on Borehole Records

## SOIL DESCRIPTION

Terminology describing common soil genesis:

*Topsoil:* mixture of soil and humus capable of supporting good vegetative growth.

*Peat:* fibrous fragments of visible and invisible decayed organic matter.

*Fill:* where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc.; none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

*Till:* the term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Terminology describing soil structure:

*Desiccated:* having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.

*Stratified:* alternating layers of varying material or color with the layers greater than 6 mm thick.

*Laminated:* alternating layers of varying material or color with the layers less than 6 mm thick.

*Fissured:* material breaks along plane of fracture.

*Varved:* composed of regular alternating layers of silt and clay.

*Slickensided:* fracture planes appear polished or glossy, sometimes striated.

*Blocky:* cohesive soil that can be broken down into small angular lumps which resist further breakdown.

*Lensed:* inclusion of small pockets of different soil, such as small lenses of sand scattered through a mass of clay; not thickness.

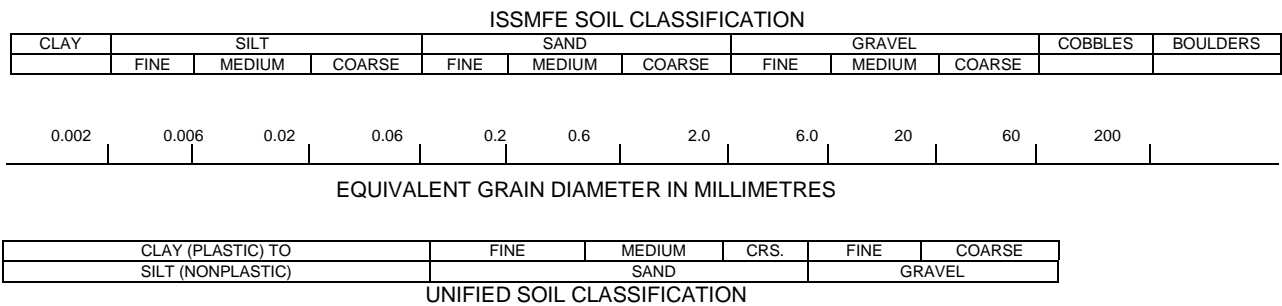
*Seam:* a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

*Homogeneous:* same color and appearance throughout.

*Well Graded:* having wide range in grain sized and substantial amounts of all predominantly on grain size.

*Uniformly Graded:* predominantly on grain size.

All soil sample descriptions included in this report follow the ASTM D2487-11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System). The system divides soils into three major categories: (1) coarse grained, (2) fine-grained, and (3) highly organic. The soil is then subdivided based on either gradation or plasticity characteristics. The system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification. The classification excludes particles larger than 76 mm. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually in accordance with ASTM D2488-09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems. Others may use different classification systems; one such system is the ISSMFE Soil Classification.



Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present and as described below in accordance with Note 16 in ASTM D2488-09a:

Table a: Percent or Proportion of Soil, Pp

	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	$5 \leq Pp \leq 10\%$
Little	$15 \leq Pp \leq 25\%$
Some	$30 \leq Pp \leq 45\%$
Mostly	$50 \leq Pp \leq 100\%$

The standard terminology to describe cohesionless soils includes the compactness as determined by the Standard Penetration Test 'N' value:

Table b: Apparent Density of Cohesionless Soil

	'N' Value (blows/0.3 m)
Very Loose	$N < 5$
Loose	$5 \leq N < 10$
Compact	$10 \leq N < 30$
Dense	$30 \leq N < 50$
Very Dense	$50 \leq N$



The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests or similar field and laboratory analysis, Standard Penetration Test 'N' values can also be used to provide an approximate indication of the consistency and shear strength of fine grained, cohesive soils:

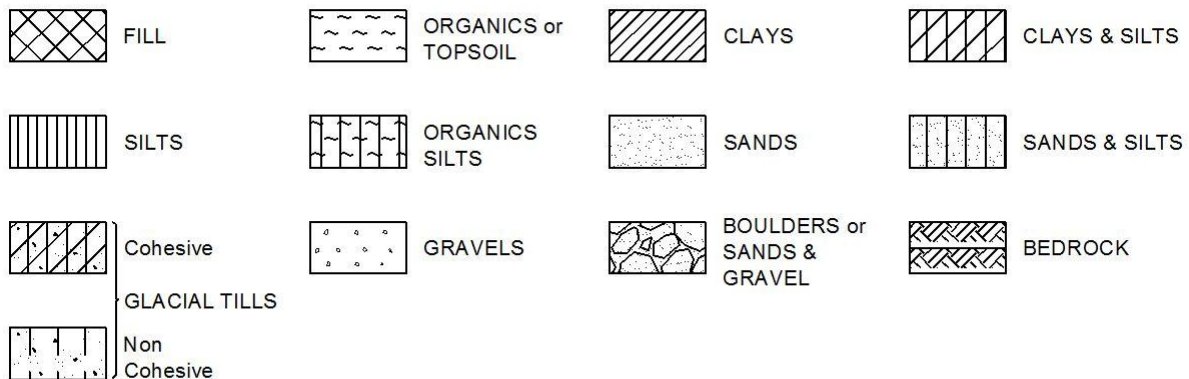
Table c: Consistency of Cohesive Soil

Consistency	Vane Shear Measurement (kPa)	'N' Value
Very Soft	<12.5	<2
Soft	12.5-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Note: 'N' Value - The Standard Penetration Test records the number of blows of a 140 pound (64kg) hammer falling 30 inches (760mm), required to drive a 2 inch (50.8mm) O.D. split spoon sampler 1 foot (305mm). For split spoon samples where full penetration is not achieved, the number of blows is reported over the sampler penetration in meters (e.g. 50/0.15).

## STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



## WATER LEVEL MEASUREMENT



Open Borehole or Test Pit



Monitoring Well, Piezometer or Standpipe

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

SS	Split spoon sample (obtained from the Standard Penetration Test)
WS	Wash sample
BS	Bulk sample
TW	Thin wall sample or Shelby tube
PS	Piston sample
AS	Auger sample
VT	Vane test
GS	Grab sample
HQ, NQ, etc.	Rock core samples obtained with the use of standard size diamond drilling bits

### STRESS AND STRAIN

$u_w$	kPa	Pore water pressure
$r_u$	1	Pore pressure ratio
$\sigma$	kPa	Total normal stress
$\sigma'$	kPa	Effective normal stress
$\tau$	kPa	Shear stress
$\sigma_1, \sigma_2, \sigma_3$	kPa	Principal stresses
$\varepsilon$	%	Linear strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	Principal strains
E	kPa	Modulus of linear deformation
G	kPa	Modulus of shear deformation
$\mu$	1	Coefficient of friction

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	Coefficient of volume change
$c_c$	1	Compression index
$c_s$	1	Swelling index
$c_r$	1	Recompression index
$c_v$	m <sup>2</sup> /s	Coefficient of consolidation
H	m	Drainage path
$T_v$	1	Time factor
U	%	Degree of consolidation
$\sigma'_{v0}$	kPa	Effective overburden pressure
$\sigma'_p$	kPa	Preconsolidation pressure
$\tau_f$	kPa	Shear strength
$c'$	kPa	Effective cohesion intercept
$\phi'$	—°	Effective angle of internal friction
$c_u$	kPa	Apparent cohesion intercept
$\phi_u$	—°	Apparent angle of internal friction
$\tau_R$	kPa	Residual shear strength
$\tau_r$	kPa	Remoulded shear strength
$S_t$	1	Sensitivity = $c_u/\tau_r$

### PHYSICAL PROPERTIES OF SOIL

$P_s$	kg/m <sup>3</sup>	Density of solid particles
$\gamma_s$	kN/m <sup>3</sup>	Unit weight of solid particles
$\rho_w$	kg/m <sup>3</sup>	Density of water
$\gamma_w$	kN/m <sup>3</sup>	Unit weight of water
$\rho$	kg/m <sup>3</sup>	Density of soil
$\gamma$	kN/m <sup>3</sup>	Unit weight of soil
$\rho_d$	kg/m <sup>3</sup>	Density of dry soil
$\gamma_d$	kN/m <sup>3</sup>	Unit weight of dry soil
$\rho_{sat}$	kg/m <sup>3</sup>	Density of saturated soil
$\gamma_{sat}$	kN/m <sup>3</sup>	Unit weight of saturated soil
$\rho'$	kg/m <sup>3</sup>	Density of submerged soil
$\gamma'$	kN/m <sup>3</sup>	Unit weight of submerged soil
$e$	1, %	Void ratio
$n$	1, %	Porosity
$w$	1, %	Water content
$S_r$	%	Degree of saturation
$W_L$	%	Liquid limit
$W_P$	%	Plastic limit
$W_s$	%	Shrinkage limit
$I_p$	%	Plasticity index = $(W_L - W_P)$
$I_L$	%	Liquidity index = $(W - W_P)/I_p$
$I_C$	%	Consistency index = $(W_L - W)/I_p$
$e_{max}$	1, %	Void ratio in loosest state
$e_{min}$	1, %	Void ratio in densest state
$I_D$	1	Density index = $(e_{max} - e)/(e_{max} - e_{min})$
D	mm	Grain diameter
$D_n$	mm	N percent - diameter
$C_u$	1	Uniformity coefficient
h	m	Hydraulic head or potential
q	m <sup>3</sup> /s	Rate of discharge
v	m/s	Discharge velocity
i	1	Hydraulic gradient
k	m/s	Hydraulic conductivity
j	kN/m <sup>3</sup>	Seepage force

## 1 OF 1

METRIC

ORIGINATED BY NT

COMPILED BY ET

CHECKED BY SM

+ 3, × 3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

Brampton, Ontario

## RECORD OF BOREHOLE No BH M15-2

1 OF 1

METRIC

W. P. WO 2015-11006

LOCATION Mckerrow Patrol Yard, MTM Z12 E247944 N5128080

ORIGINATED BY NT

DIST Sudbury, Hwy 17

BOREHOLE TYPE CME-55, Hollow stem auger/ Diamond drill, Cased hole (below 1.52m)

COMPILED BY ET

DATUM BM ELEV 209.06m MTM Z12 E247865 N5128000

DATE 2015/03/16 - 2015/03/16

CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa						
								○ UNCONFINED	+ FIELD VANE	×	QUICK TRIAXIAL	LAB VANE		
208.3	Ground Surface							20	40	60	80	100		
	FILL: GRAVELLY SAND(SW) few silt, grey, moist		1	AS1										
207.6 0.8	SILT (ML) trace sand, trace clay, grey, moist, compact		2	SS2	21									0 5 91 4
206.8 1.5	SAND (SP) medium to fine sand, trace gravel, trace silt & clay, reddish to greyish brown, wet, very loose to compact		3	SS3	8									
			4	SS4	6									1 95 (4)
			5	SS5	11									
			6	SS6	6									
			7	SS7	9									
202.2 6.1	SILTY SAND (SM) greyish brown, wet, very loose to loose		8	SS8	4									
	- becoming reddish brown sand		9	SS9	6									
198.6 9.8	END OF SAMPLING		10	SS10	4									0 77 (23)
197.6 10.7	END OF BOREHOLE													
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Wash boring technique was used to advanced borehole below 1.52m depth 4. Groundwater level was measured in the open auger hole													

OPG\_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 7(MCKERROW)-BH LOGS.GPJ ONTARIO MOT.GDT 4/24/15

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

## 1 OF 1

METRIC

ORIGINATED BY NT

COMPILED BY ET

CHECKED BY SM

+ 3, × 3: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

Brampton, Ontario

## RECORD OF BOREHOLE No BH M15-4

1 OF 1

METRIC

W. P. WO 2015-11006

LOCATION Mckerrow Patrol Yard, MTM Z12 E247896 N5128044

ORIGINATED BY NT

DIST Sudbury, Hwy 17

BOREHOLE TYPE CME-55, Hollow stem auger/ Diamond drill, Cased hole (below 1.52m)

COMPILED BY ET

DATUM BM ELEV 209.06m MTM Z12 E247865 N5128000

DATE 2015/03/17 - 2015/03/17

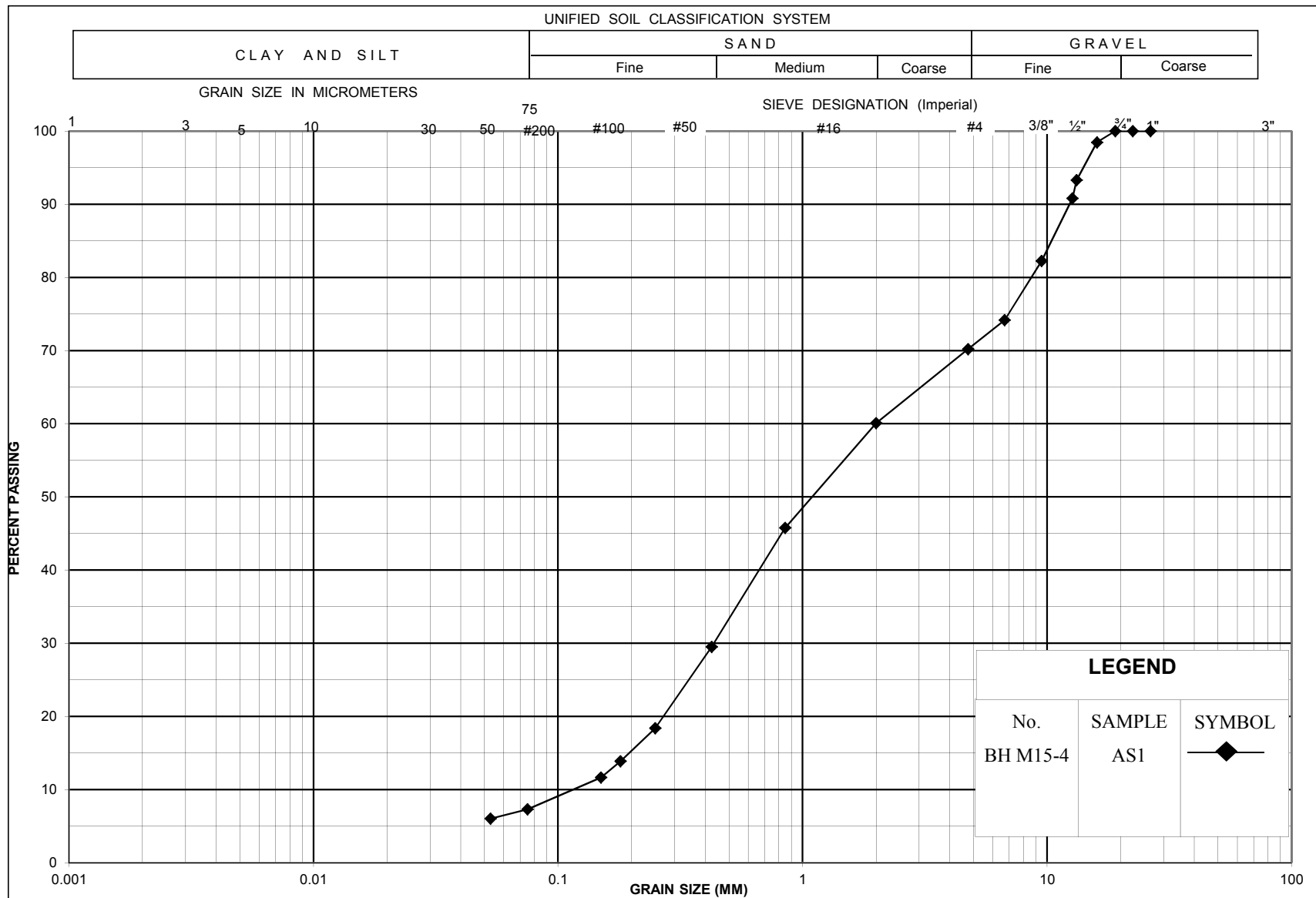
CHECKED BY SM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH: Cu, KPa							WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE								
						×	QUICK TRIAXIAL	LAB VANE									
208.4	Ground Surface					▽		20	40	60	80	100	10	20	30	GR SA SI CL	
206.9	ASPHALT 25 mm thickness FILL: GRAVELLY SAND(SW) few silt, brown, moist		1	AS1			208							○			30 63 (7)
207.6	SILT (ML) few sand, trace clay, brown, frozen, very dense		2	SS2	55		207							○			0 7 91 2
206.9	SAND (SP) medium to fine sand, few to little silt and clay, wet, loose to compact  - becoming little silt		3	SS3	16		206							○			
			4	SS4	8		205							○			
			5	SS5	5		204							○			0 85 (15)
			6	SS6	4		203							○			
			7	SS7	6		202							○			
			8	SS8	5		201							○			
198.6	END OF SAMPLING		9	SS9	12		200							○			
197.7			10	SS10	9	199							○			0 92 (8)	
197.7	END OF BOREHOLE																
10.7	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Wash boring technique was used to advanced borehole below 1.52m depth 4. Groundwater level was measured in the open auger hole																

OPG\_EXP RECORD OF BOREHOLE 5013-E-0008 ASSIG. 7(MCKERROW)-BH LOGS.GPJ ONTARIO MOT.GDT 4/24/15

+ 3, x 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

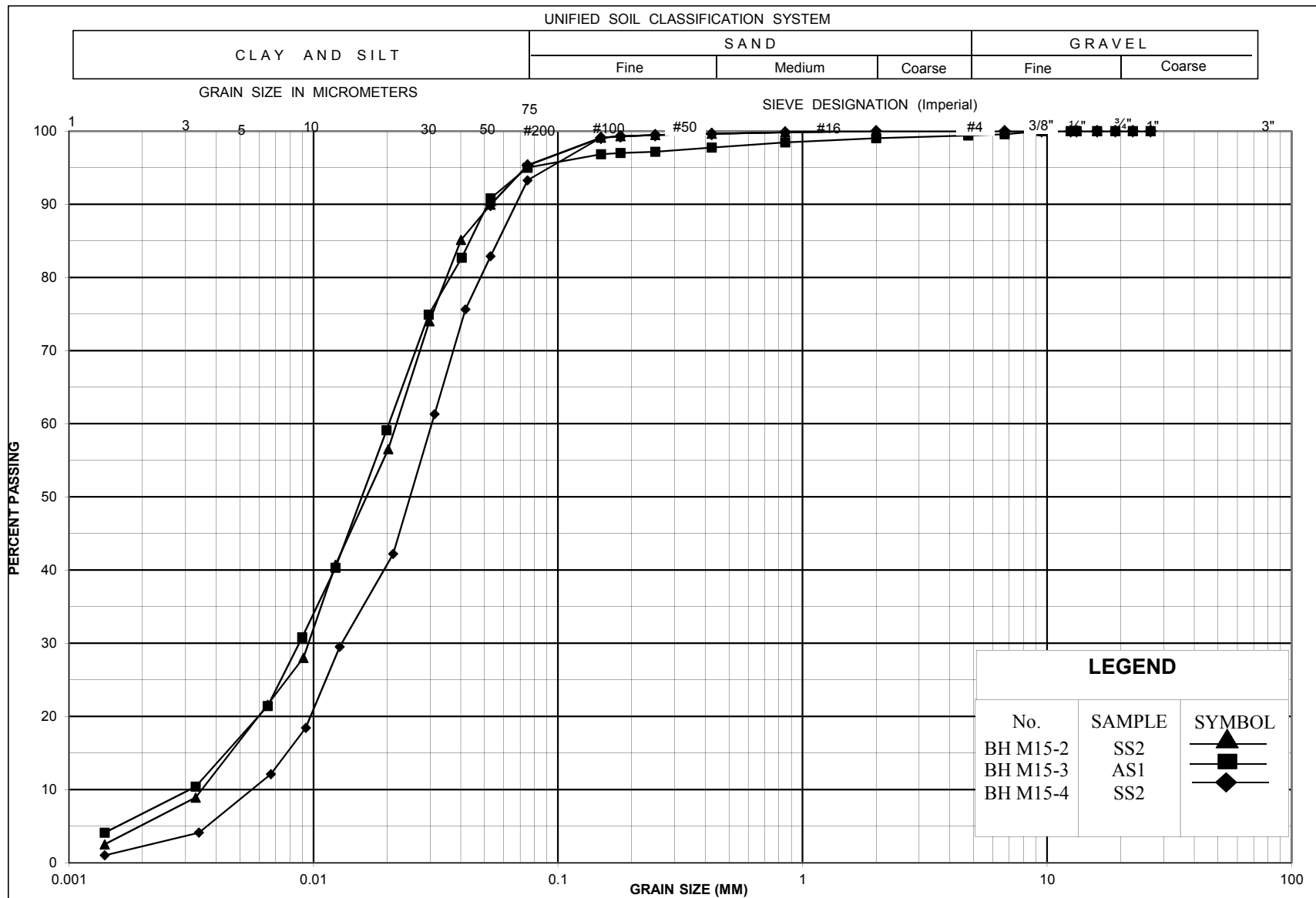
## **Appendix D – Laboratory Data**

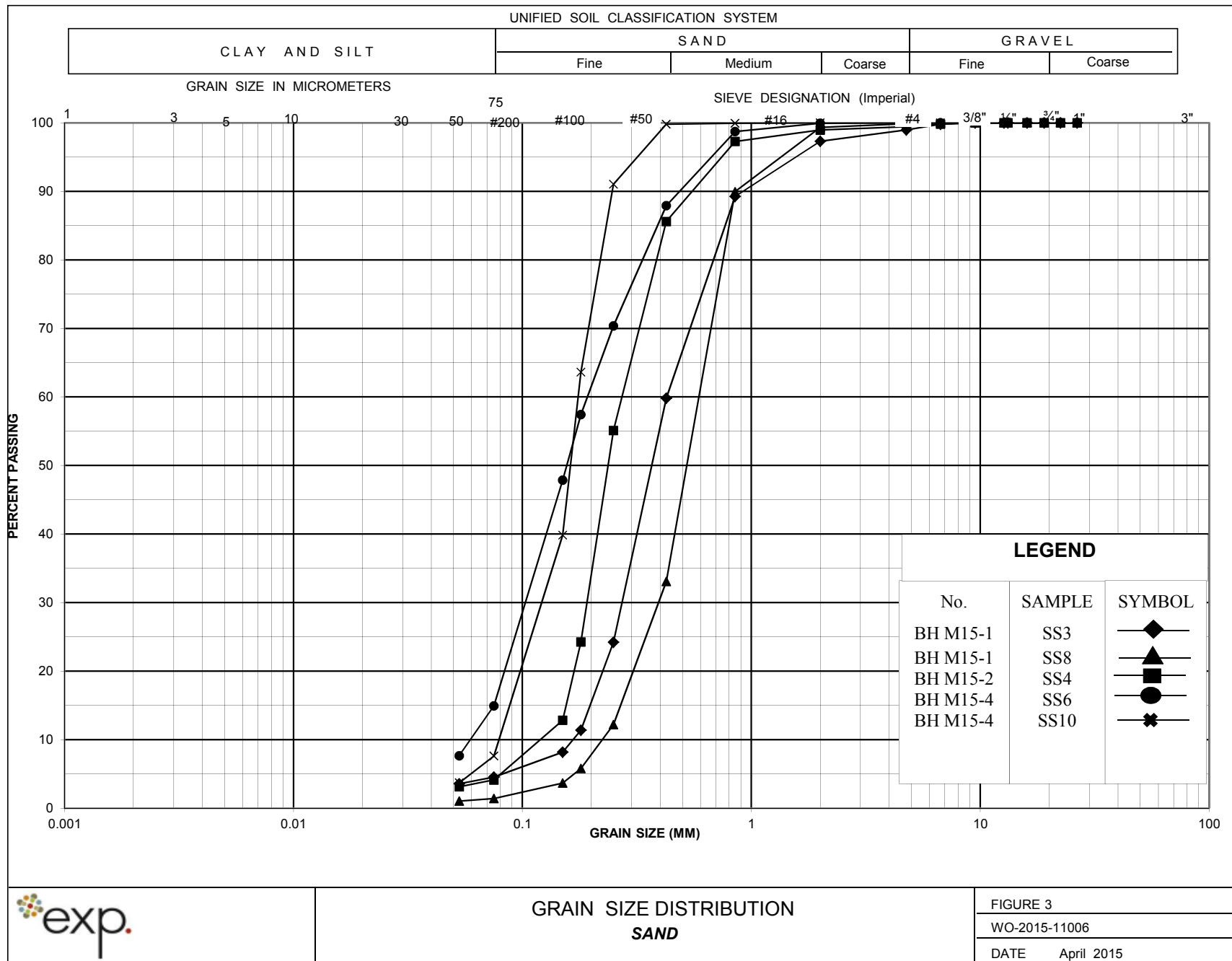


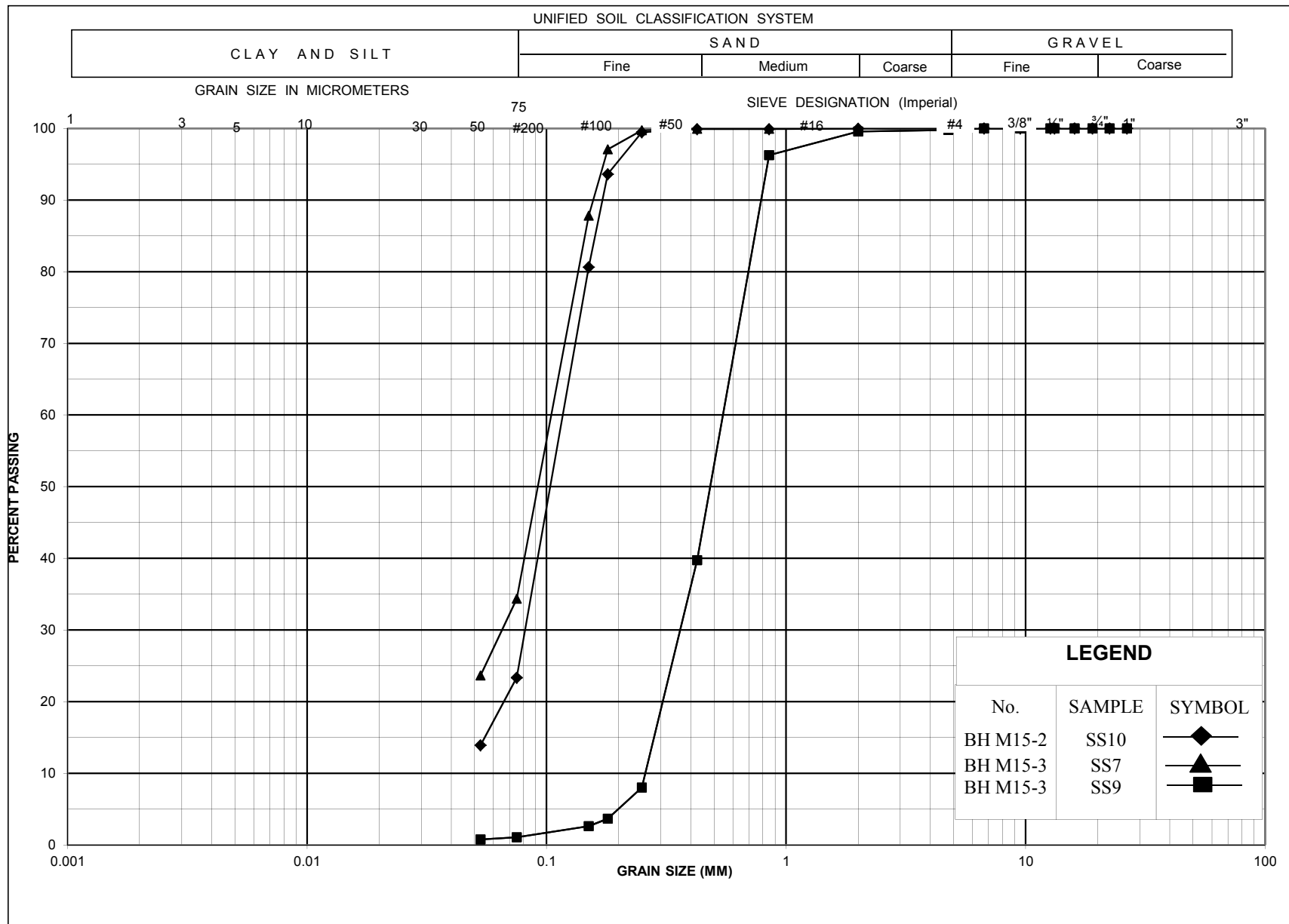
**GRAIN SIZE DISTRIBUTION**  
**FILL: GRAVELLY SAND/ SILTY SAND**

FIGURE 1  
 WO-2015-11006  
 DATE April 2015









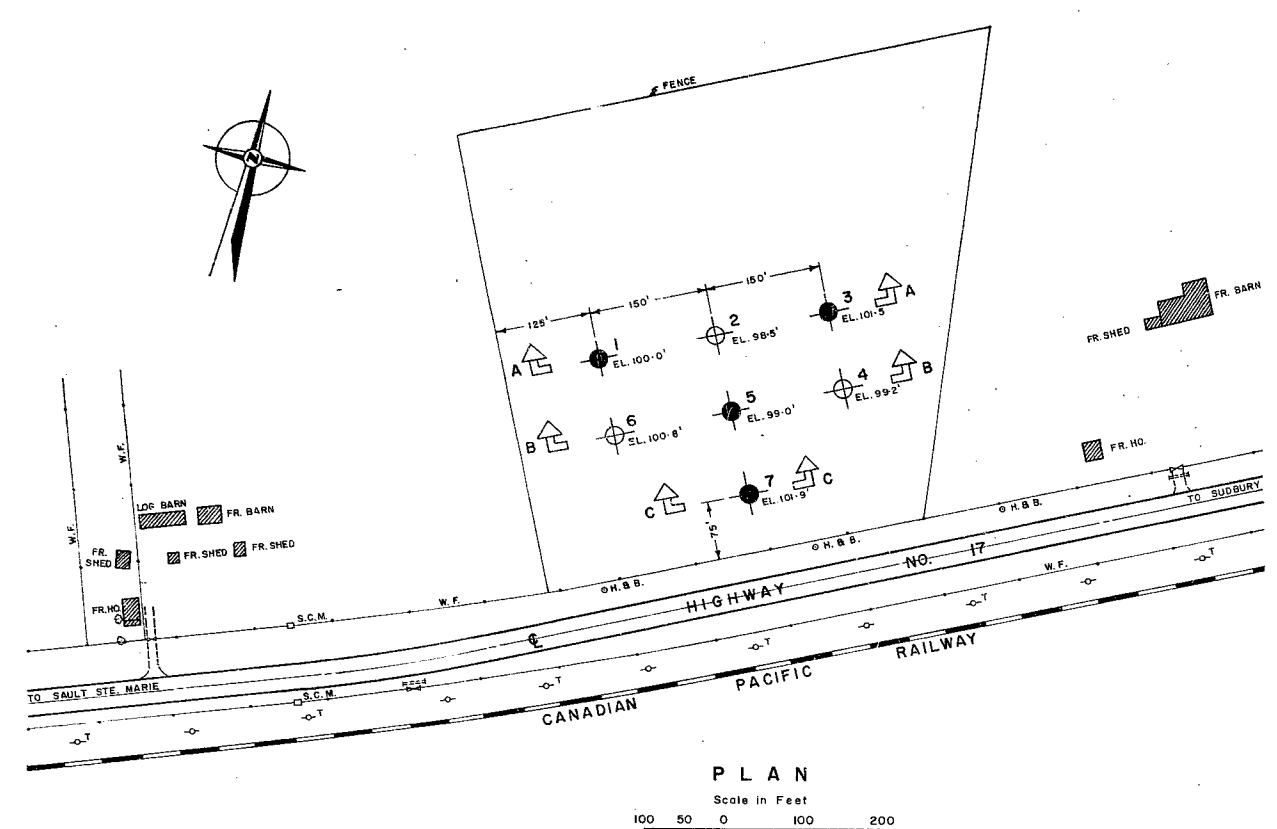
**GRAIN SIZE DISTRIBUTION**  
**SILTY SAND**

FIGURE 4

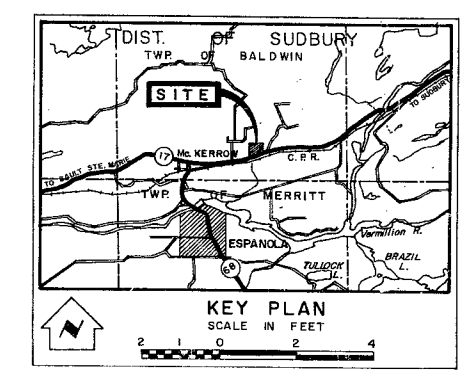
WO-2015-11006

DATE April 2015

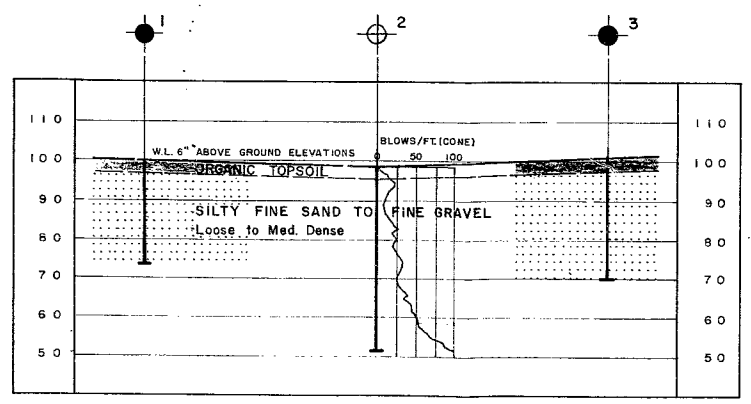
## **Appendix E – Record of Historical Geotechnical Data**



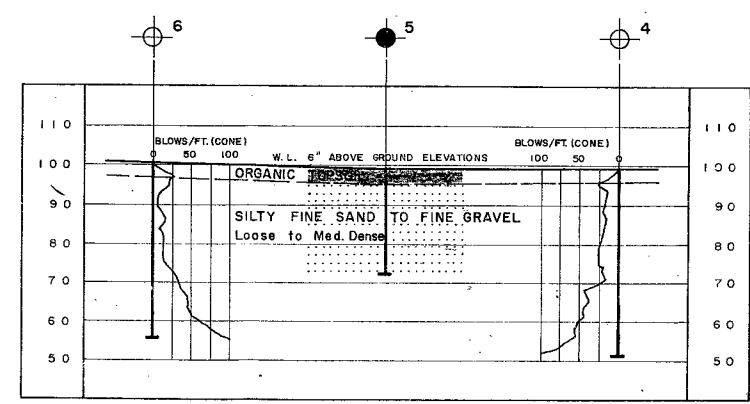
**PLAN**  
Scale in Feet  
100 50 0 100 200



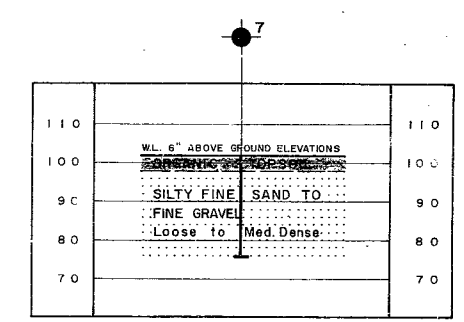
- LEGEND**
- BORE HOLE
  - ⊕ CONE PENETRATION HOLE



**SECTION A-A**  
Scale in Feet  
50 25 0 50 100  
Horizontal  
20 10 0 20 40  
Vertical



**SECTION B-B**  
Scale in Feet  
50 25 0 50 100  
Horizontal  
20 10 0 20 40  
Vertical



**SECTION C-C**  
Scale in Feet  
50 25 0 50 100  
Horizontal  
20 10 0 20 40  
Vertical

DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH SECTION

**McKerrow Garage Site**

ORIGINATED W. KULMATICAS	DISTRICT NO. 17	DATE 17 APRIL 1962
DRAWN D. MUMFORD	W.P. NO.	JOB NO. 62-F-26
CHECKED <i>[Signature]</i>	SCALE	DRAWING NO.
APPROVED <i>[Signature]</i>	AS SHOWN	62-F-26A

## **Appendix F – Results of Stability Analyses**

May 22, 2015

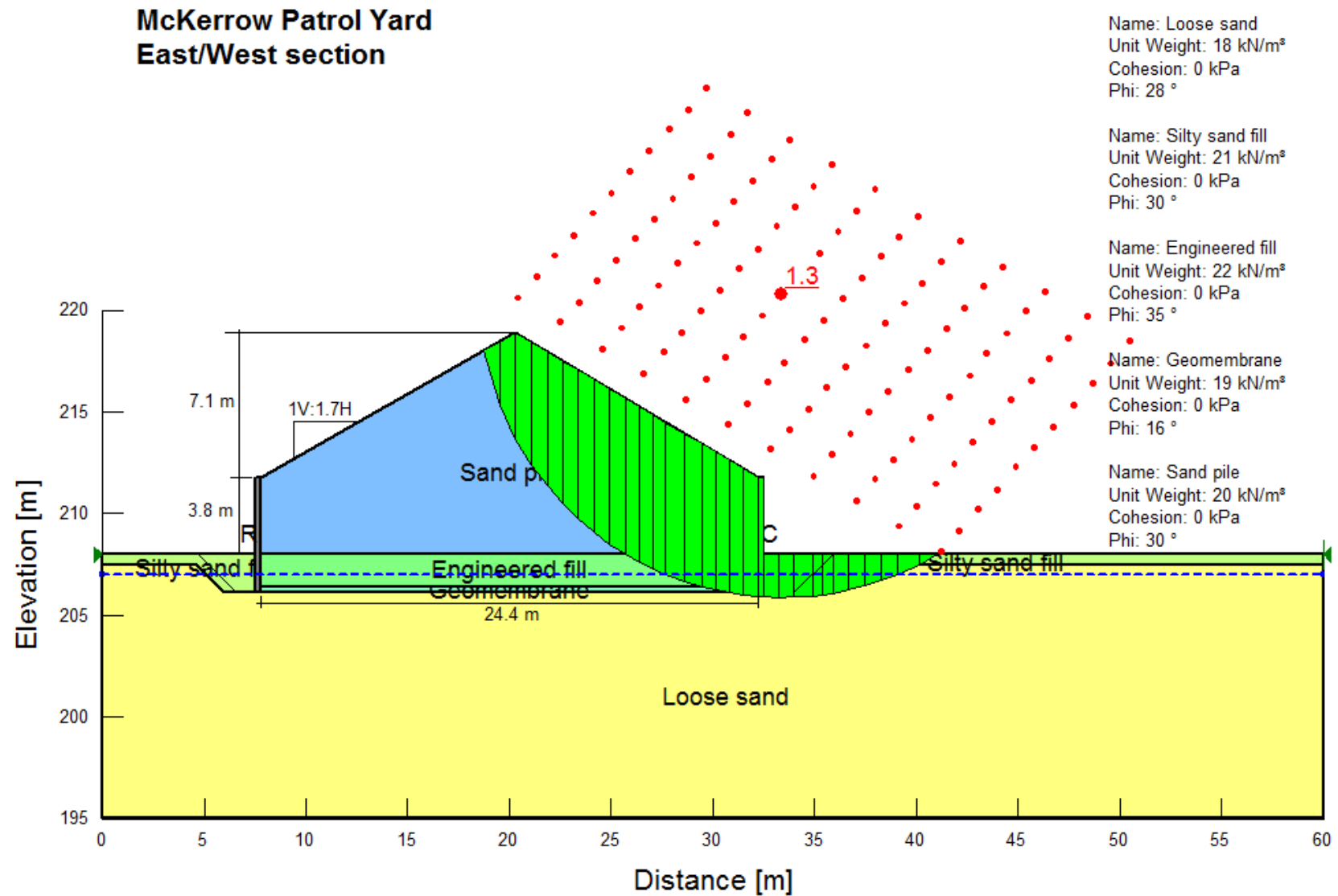


Figure F1. Results of global stability analyses with full allowable capacity – east-west section

May 22, 2015

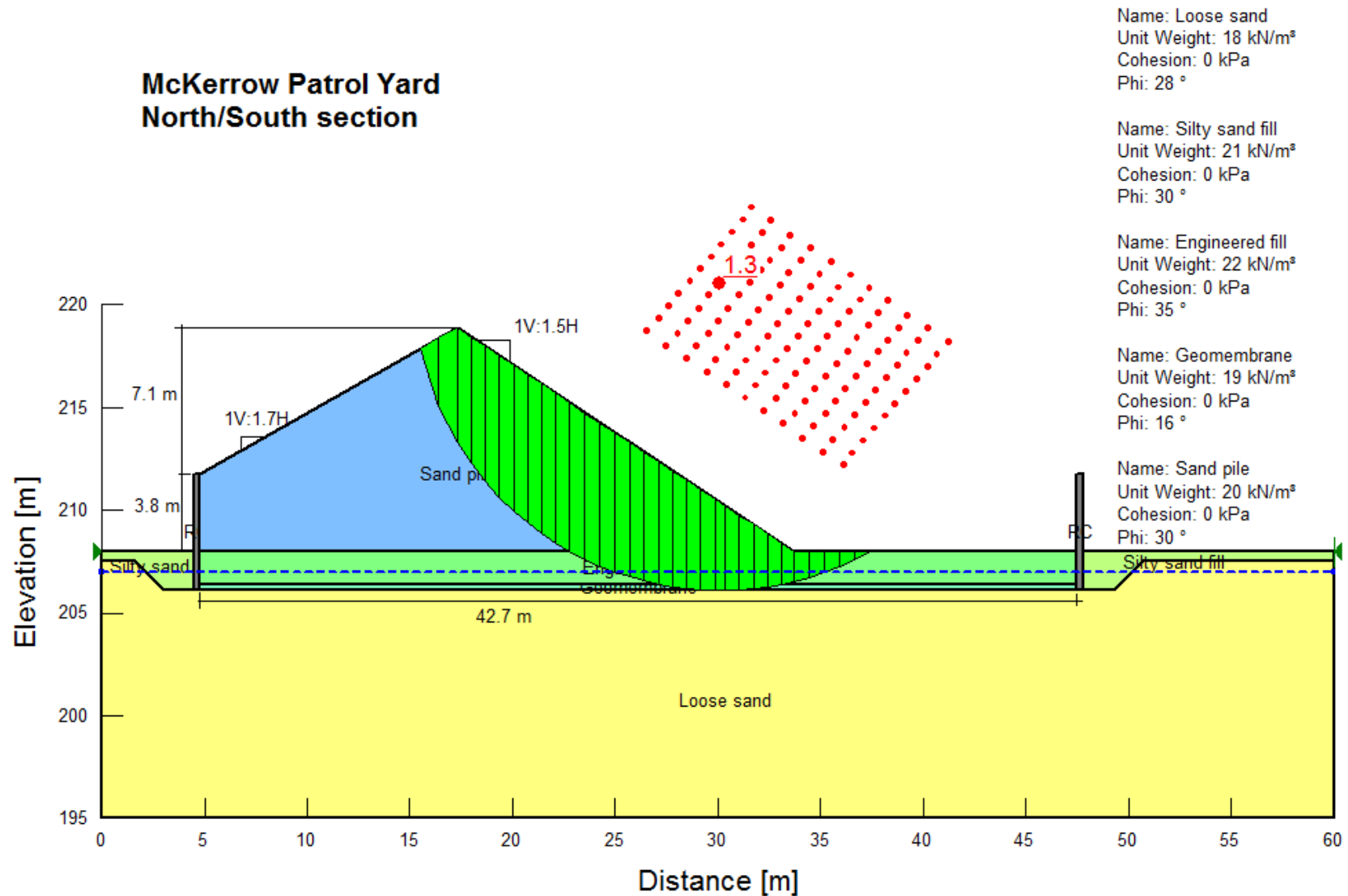


Figure F2. Results of global stability analyses with full allowable capacity – north-south section



## **Appendix G – Results of Settlement Analyses**

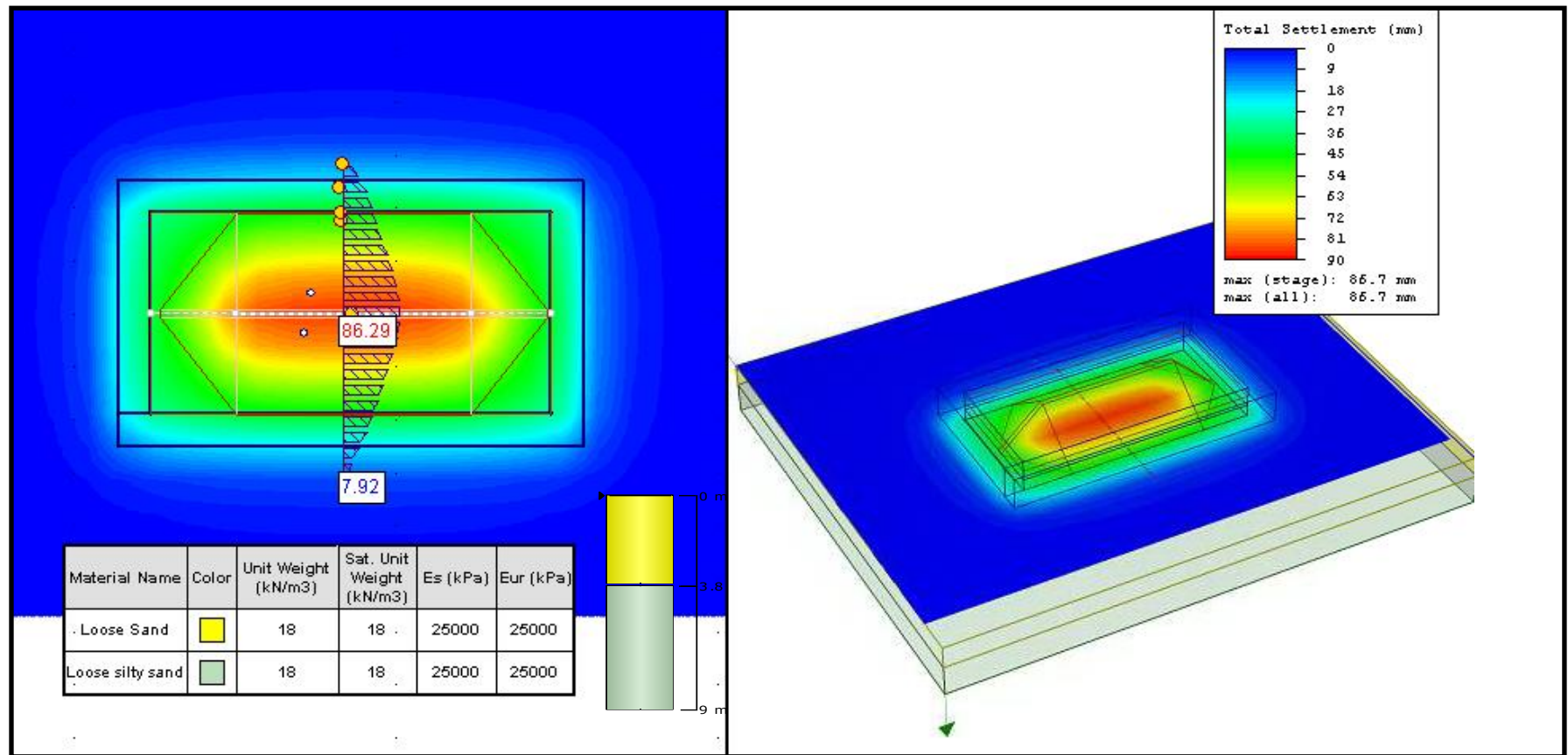


Figure G1. Settle 3D result for footing directly putting on loose sand

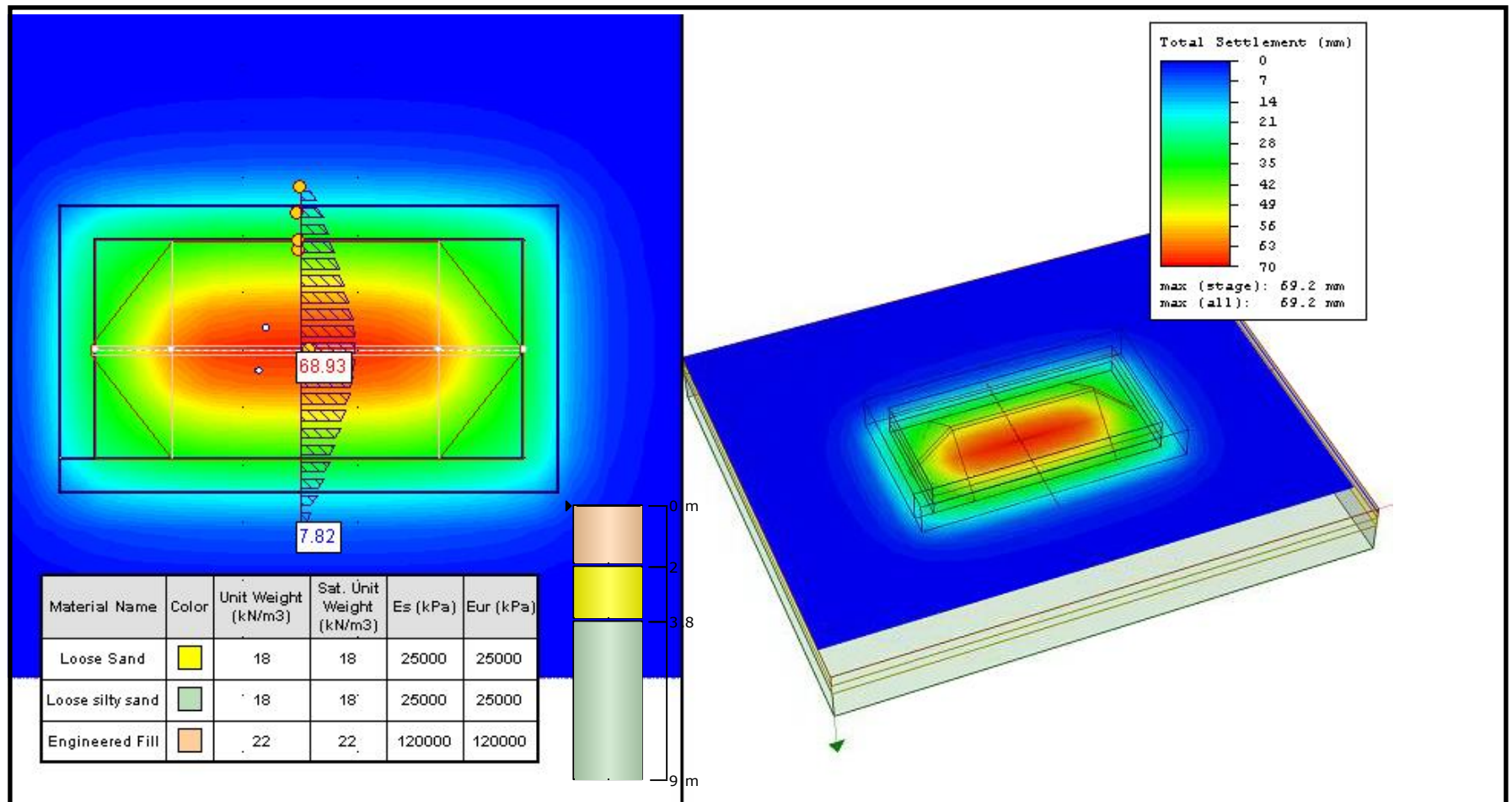


Figure G2. Settle 3D result for footing on 2 m of engineered fill

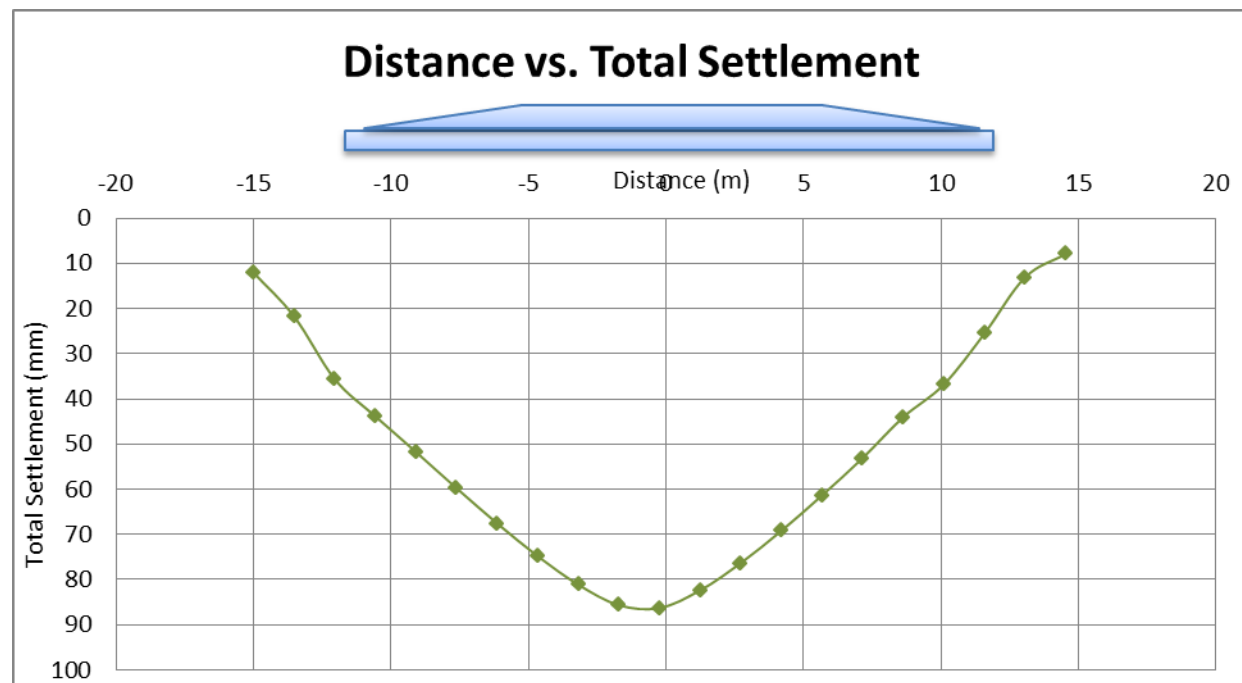


Figure G3. Settlement result for footing directly on the loose sand



Figure G4. Settlement result for footing on 2m of engineered fill

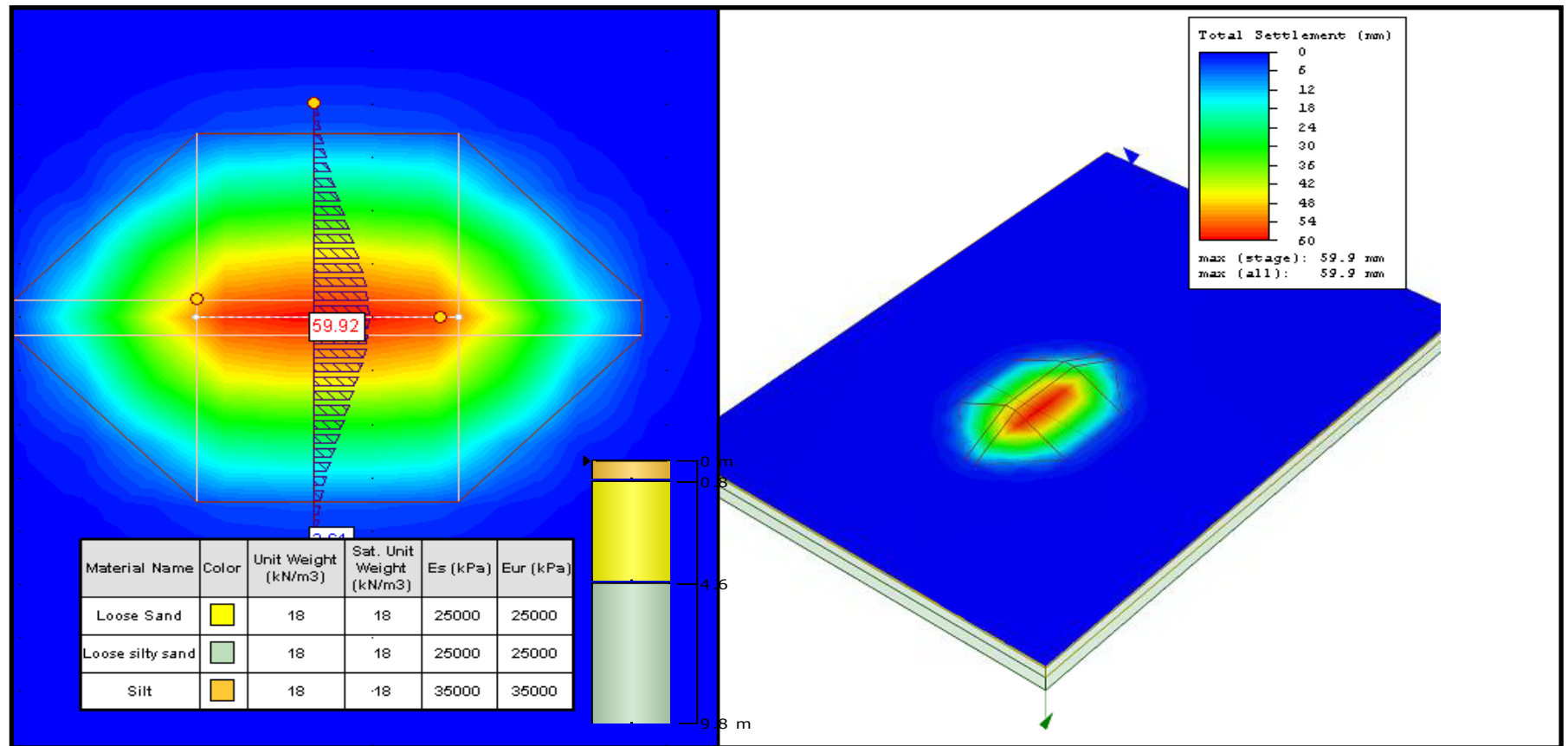


Figure G5. Settlement with a 9m high stockpile preloading

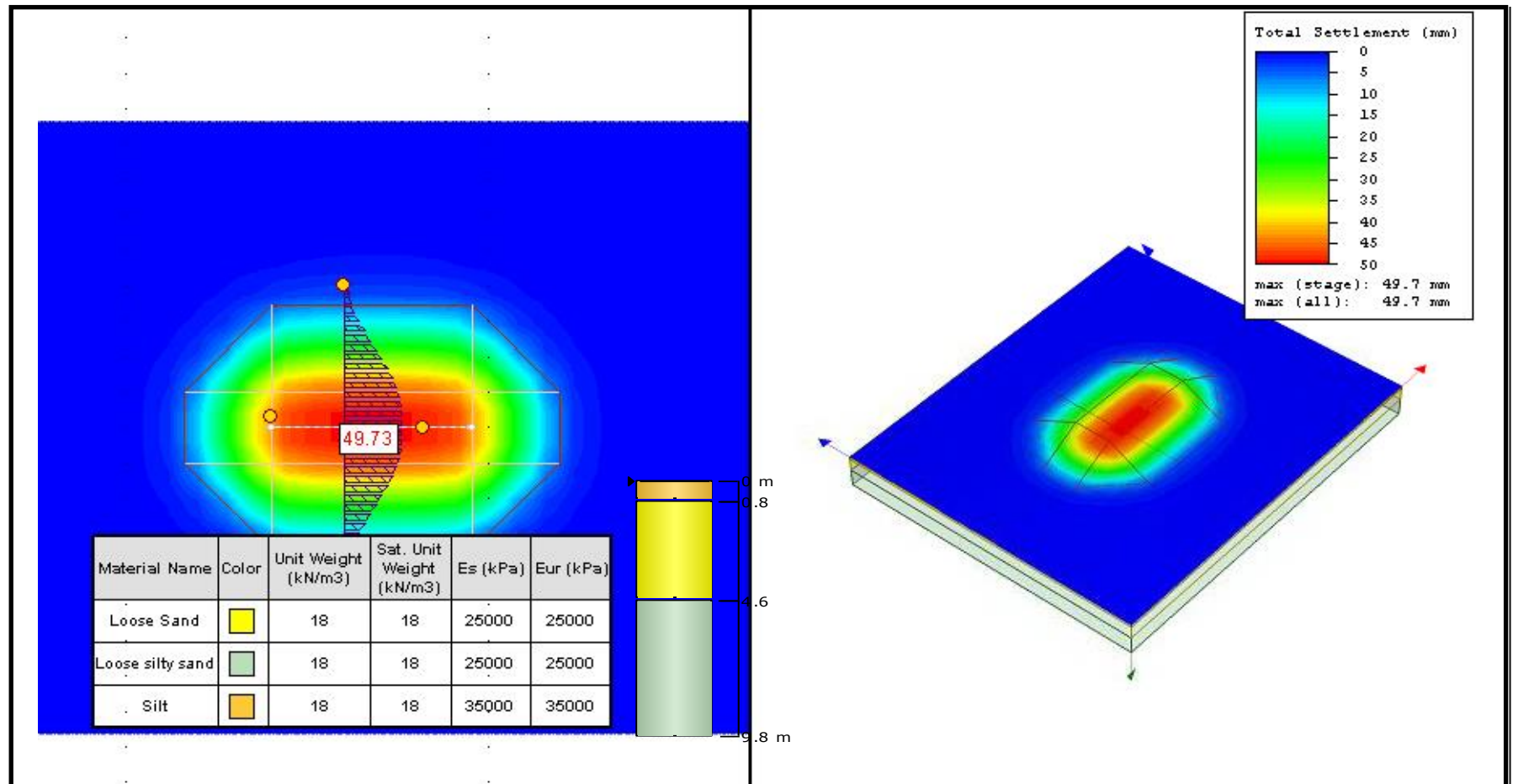


Figure G6. Settlement with a 7m high stockpile preloading

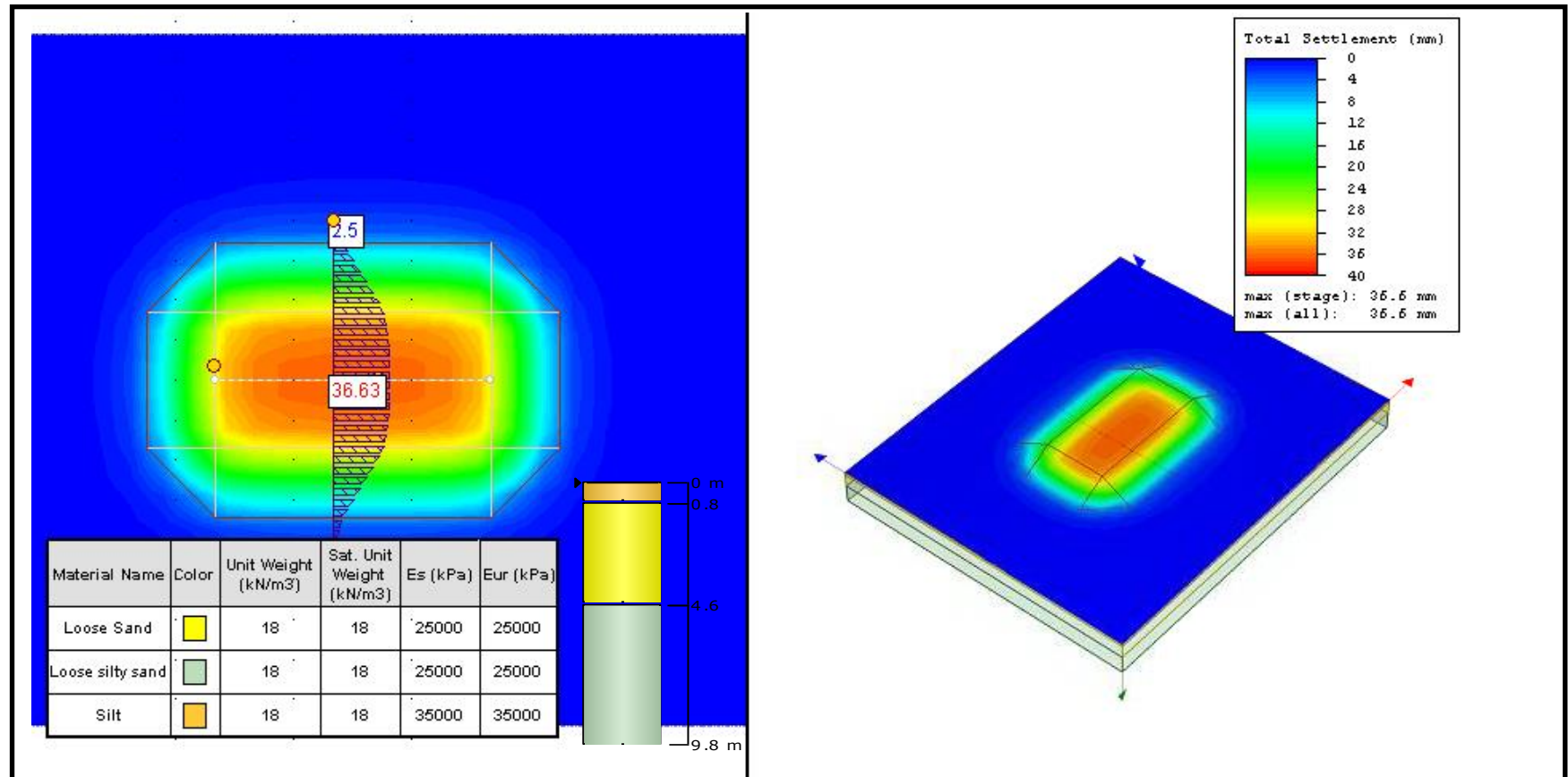


Figure G7. Settlement with a 5m high stockpile preloading



## **Appendix H – Non-Standard Special Provision**

## **NSSP FOR DEWATERING FOR EXCAVATIONS**

### **Scope of Work**

The Contractor should be aware that the groundwater table is approximately 0.9 m to 1.2 m below the ground surface, as well as that the overburden soils at the site consist of silty and gravelly sand fill and native silty sand materials which are water-bearing soils. Therefore foundation elements requiring construction below the groundwater level must be carried out in the dry. The excavation shall be kept stable during the excavation and construction.

It should be noted that water levels within the area are known to fluctuate. Therefore, it is recommended that excavation for the foundations or any other structure element be performed in late summer when the water levels are anticipated to be lower.

### **Basis of Payment**

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials required to the work.